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Project/File: 310003448 Green Steel
Sensitivity Analysis

Date: 2 February 2026

Reference: Sensitivity Analysis

1 Introduction

National Green Steel Limited (the client) have engaged Stantec New Zealand (Stantec) to undertake Sensitivity Analysis to address the following from the *Joint Witness Statement (JWS) in Relation to: Monofills, Earthworks and Related Geotechnical Issues*, dated 22 January 2026:

Liner selection

b) Whether groundwater concentration assessments are adequately conservative, particularly with respect to conceptualisation of groundwater flow area.

Matters of agreement

All parties agree that, as soon as practicable, sensitivity analysis of the groundwater model and suitability of dilution factor, including potential environmental effects on boundary taking into account 99th percentile NEMP 3.0 ecological criteria should be undertaken.

This memo presents a summary of the Sensitivity Analysis undertaken by Stantec for the proposed Southwest Monofill (Figure 1). The analysis focuses on the calculations presented in Sections 6.4 and 6.5 of the following report:

"Monitoring Plan and Evaluation of Surface and Groundwater Effects, Green Steel Monofill, Hampton Downs, 61 Hampton Downs Road, Hampton Downs, Waikato". Earthtech Consulting Limited, dated 9 June 2025, Ref: R4424-6 (Earthtech, 2025a).

Figure 1 shows the location of the two proposed monofills. Calculations for assessment of effects undertaken by Earthtech (2025a) focussed on the Southwest Monofill. The proposed location of this monofill is considered to be the more sensitive environment given the proximity to the Waipapa Stream which runs along the southwestern boundary.

Reference: Green Steel Hydrogeological AEE

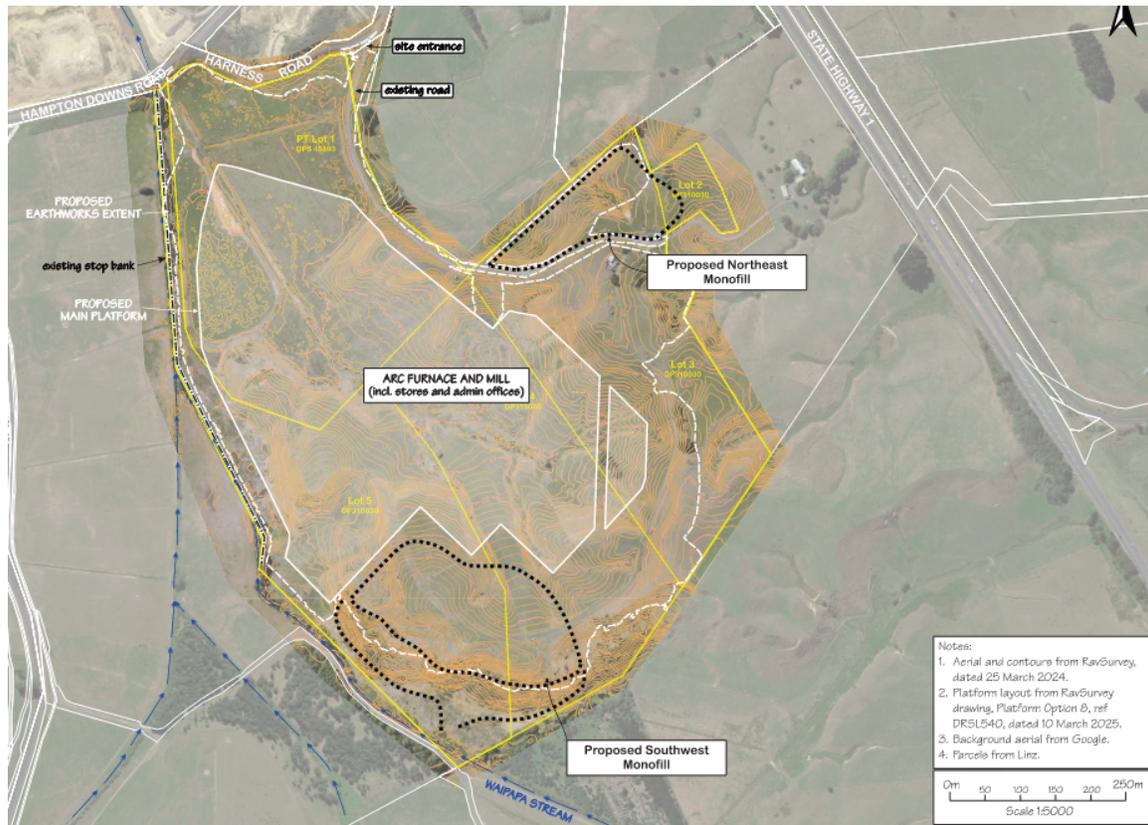


Figure 1 Monofill location plan (Earthtech, 2025)

2 Environmental Setting and Conceptual Site Model

The regional mapped geology shows three main units beneath the site; Holocene alluvium deposits, rhyolitic terrace deposits and the Amokura Formation (alternating siltstone/sandstone), which is a sub unit of the Waitemata Group (GNS Science, 2026). Ground investigations for the site confirm this geological setting. Four auger borelogs MF1, MF2, MF3 and MF54 (Figure 2) provided information on the subsurface conditions close to Southwest Monofill. MF1 and MF2 at lower elevations intercepted the Holocene alluvium, with saturated alluvium material intercepted at >1.4 m below ground level (bgl). MF3 and MF4 at higher elevations encountered alluvium underlain by volcanic ash. The two bores at higher elevations did not encounter saturated sediment. MF4 had the highest elevation and encountered Amokura sandstone material at 2.2 m bgl, approximately 10 m relative level (RL).

Over the remainder of the site, the geotechnical investigation (Earthtech, 2024) interpreted the alluvium/Amokura Formation at approximately 4 mRL for most of the site, rising in elevation towards the Southwest Monofill, then falling away in elevation toward the southern boundary as observed in MF1 to

Reference: Green Steel Hydrogeological AEE

MF4. The weathering depth for the Amokura Formation has been interpreted as 3 m to 10 m (bgl) (Earthtech, 2024).

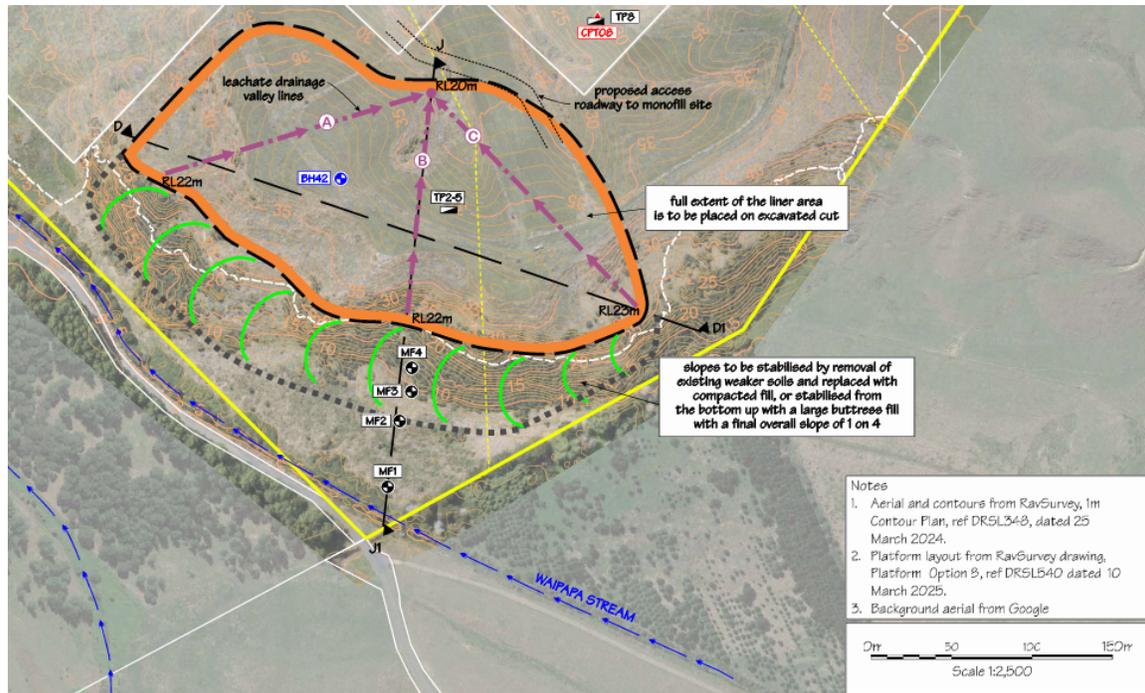


Figure 2 Southwest Monofill with hand auger locations (Earthtech, 2025)

The regional aquifer is the Waitematā Sandstone Aquifer, which is a fractured aquifer comprised of interbedded sandstone and siltstone. Groundwater flow is mostly horizontal through fractures and the sandstone beds with the mudstone sequences acting as aquitards. A transmissivity range of 6 – 62 m²/day has been estimated for the aquifer (Pattle Delamore Partners Ltd, 2012) with an estimated hydraulic conductivity value of 2.72 x 10⁻² m/day (Viljevac et al., 2002). Two exploration bores at the site (BH54 and BH42) intercepted groundwater associated with the fractured aquifer between 8 m and 33 m below ground surface, which correspond to 6 m and 9 mRL. These groundwater levels are associated with fractured zones intercepted at >57 m depth (-15 mRL). Estimates from site specific tests for transmissivity and storativity gave values of 12 m²/day and 7 x 10⁻⁴ respectively (Earthtech, 2025b)

We could not find any published information regarding the shallow unconfined watertable. Site investigations indicate that across the wider site, groundwater was found from 0.5 to 3 m bgl in what is interpreted as the shallow watertable.

To the south of Southwest Monofill groundwater was encountered within the Holocene alluvium at a depth of approximately 4.5 mRL. The Waipapa Stream was estimated to be at approximately 4 mRL, indicating that locally the shallow aquifer is contributing to the stream. Expected hydraulic conductivity (K) for the unconfined groundwater within silty sand/sandy silt ranges between 1x10⁻⁷ m/s to 5x10⁻³ m/s (Freeze and Cherry, 1979).

Reference: Green Steel Hydrogeological AEE

Additionally, seeps have also been observed at/within the 'horseshoe' area north of the Southwest Monofill between 25 and 30 mRL indicating perched aquifers in the near surface units at higher elevations.

2.1 Conceptual Site Model

We have reviewed the hydrogeological information for the Southwest Monofill and have updated the conceptual model (red annotation) as presented in the Earthtech (2025a) report. This is summarised below and presented in Figure 3.

From the observed geology we have assumed that the underlying Amokura Formation is elevated in the location of the Southwest Monofill and there is less alluvium coverage compared to that observed in augers MF1 to MF3. The saturated alluvium (sandy silt) observed in MF1 and MF2 is representative of the unconfined aquifer at depth of approximately 5 mRL, 7 m below the proposed liner depth of 12 mRL. We have assumed that this aquifer will be found at a similar depth across the site (within the alluvium, residual soils and weathered Amokura Formation materials) with slight variations in depth in response to geological and topographic changes. Based on site geology, we have assumed the unconfined groundwater aquifer will be situated near the interface of the alluvium and Amokura Formation (Earthtech, 2024) across the extent of the whole site. Leachate discharge will be to the unconfined aquifer rather than to the fractured Waitemata Sandstone Aquifer, which is at depth (>57 m) and under semiconfined/confined conditions. It is inferred that the unconfined groundwater beneath the monofill will follow topography and will flow southwest towards Waipapa Stream. Hydrogeological parameters will be those associated with an unconfined aquifer within sandy silt.

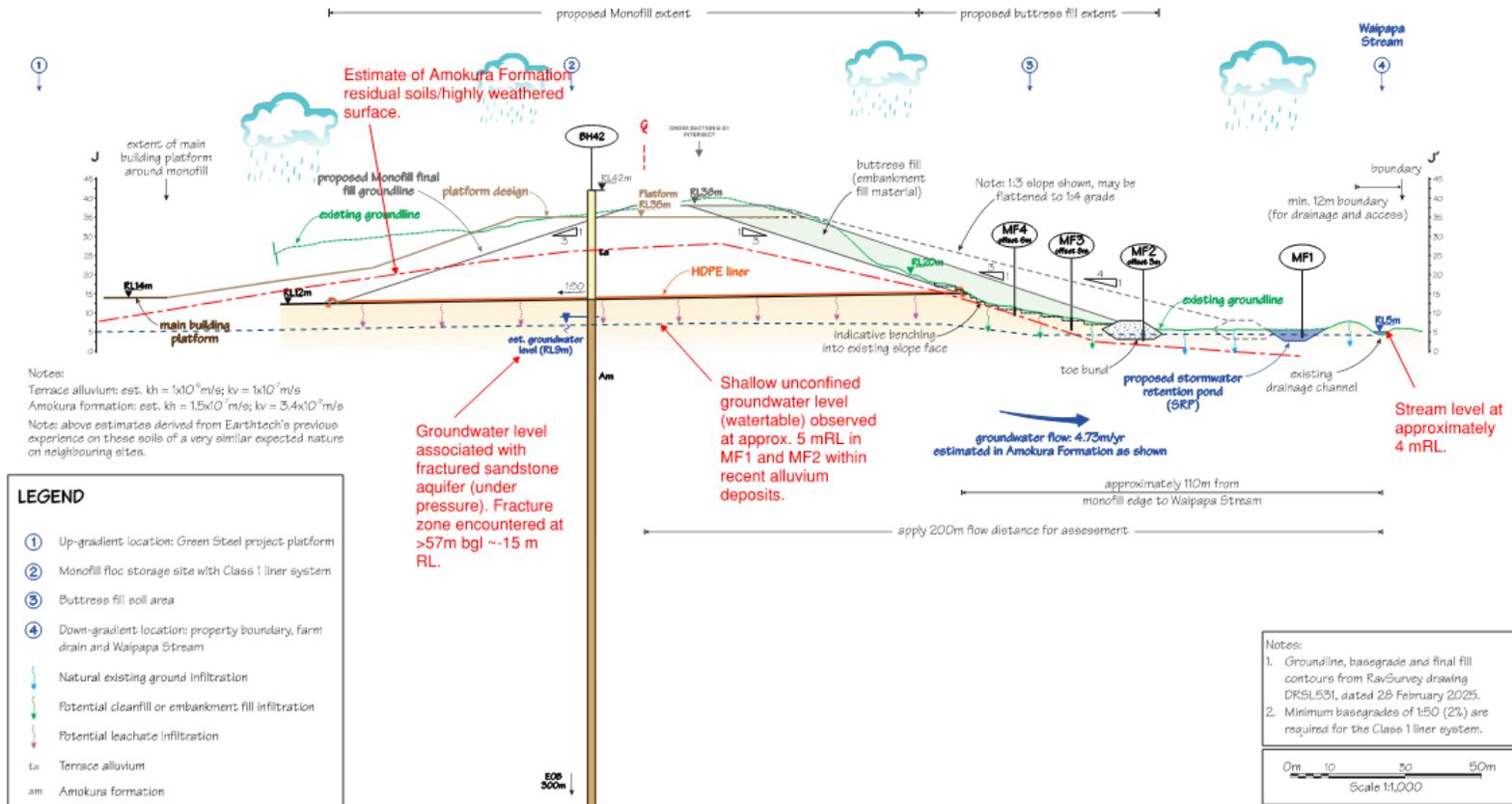


Figure 3 Conceptual Site Model, modified from Figure M6.2 (Earthtech, 2025). Stantec updates in red.

3 Sensitivity Analysis

3.1 Liner Leachate Rates

Stantec understands the calculation on page 15 (Section 6.4) is a simplified approach, which is based on the use of Darcy's equation. The estimated flow of leachate through the whole of the base of the monofill liner is calculated to be 4.464 litres/day. However, it would appear that the number of defects assumed in the calculation is one per 4,000 m², and not two defects per 4,000 m² as stated. Correcting this approximately doubles the estimated flow through the monofill base to 8.6 litres/day.

As a check of the calculation Stantec have used the method adopted by Qian et al¹ (from Giroud and Bonaparte) to estimate the flow through the base of the monofill. This is covered in Spreadsheet 1 (appended) and is summarised as follows:

- The results show that for good contact between an HDPE geomembrane and underlying geosynthetic clay liner (GCL), the leakage would amount to approximately 0.557 litres/day for the whole of the Southwest Monofill.
- If the contact is poor, then there is more leakage and it would increase to about 3.05 litres/day for the whole of the Southwest Monofill.
- The check shows a similar order of magnitude, though less in amount, under poor contact conditions. In that respect, Stantec consider the amount derived (i.e., assume it is corrected to 8.6 litres/day) to be a conservative estimate and it would be appropriate at detailed design stage to update the design calculations.

The difference in leakage rates between having good contact and poor contact between HDPE and underlying GCL (i.e., 0.557 litres/day versus 3.05 litres/day, or 5.5 times more for poor contact) stresses the importance of having good construction practices and quality control.

An underlying assumption in Earthtech's calculation, (as well as the check done by Stantec), is that a 300 mm head of leachate could develop on top of the liner. We have checked this in Spreadsheet 2 (appended) using three different methods. Indications are that the head could be as high as 291 mm at the point where the distance between leachate drains is largest. So, to assume a value of 300 mm for all circumstances is conservative.

¹ "Geotechnical Aspects of Landfill Design and Construction"; Qian, X., Koerner, R.M., Gray, D.H.; Prentice Hall; New Jersey; 2002; ISBN 0-13-012506-7.

Reference: Green Steel Hydrogeological AEE

3.2 Liner Leakage Effects on Groundwater

Stantec have reviewed the parameters used for the calculation of flow and dilutions (Section 6.5). As a check of the calculation inputs we have made the following changes and assumptions:

- We have doubled the estimated flow through the monofill base to 8.6 litres/day to account for approximately double the number of defects.
- The original calculations assume 50% of the leakage would be picked up in a subsoil drain, we have conservatively calculated 100% leachate leakage to groundwater (i.e. assumed no subsoil drain).
- The original calculations have assumed an aquifer with a depth of 50 m (and width of 330 m) in the flow calculation. We have conservatively changed this to 4 m depth (and kept the width at 330 m). This aquifer depth assumes the aquifer is restricted to the alluvium and residual soils/weathered material of the Amokura Formation (weathered layer thickness of the Amokura Formation ranged between 3 m and 10 m across the site (Earthtech, 2024)) and is unconnected to the deeper Waitematā Sandstone Aquifer.

We have checked the hydraulic conductivity parameters (K) used in the Earthtech (2025a) calculations and find the values to be at the low end of the expected range for the observed soil type. Increasing the rate will result in higher flow and therefore higher dilution rates. Therefore, we consider the K values to be conservative and have left as is.

Our calculations with the updated dilution factors and daily loading rates are presented in Spreadsheet 3 (appended).

3.3 Dilution Calculations

As per the original Earthtech (2025a) calculations Stantec have used the 'first wash' TCLP concentrations from the Lysimeter trials for the loading rates. The dilution rates within the groundwater are presented in Spreadsheet 3. For a comparison against freshwater guidelines, in the absence of flow data we have assumed stream flow values of 0.5 L/s and 1 L/s. We have chosen low flow values based on the location in the catchment and size of the stream. We note that the stream is likely groundwater fed and given it has a well-formed channel it is likely perennial. Therefore, we consider these two flow scenarios are reasonably representative of low flow periods. The results of these calculations are presented in Spreadsheet 4 (appended).

4 Assessment of Environmental Effects

We have assessed the predicted dilution estimates against the Australian & New Zealand Guidelines for Fresh & Marine Water quality (ANZG, 2018), the PFAS National Environmental Management Plan 3.0 (NEPM, 2025) and the Water Services (Drinking Water Standards for New Zealand) Regulations 2022 (DWSNZ, 2022) as summarised in Table 1 (appended). Even in low flow conditions the predicted

Reference: Green Steel Hydrogeological AEE

determinand concentrations are within the ecological and drinking water guidelines. Additionally, we note that these calculations are conservative and do not take into account natural attenuation processes that will reduce contaminant concentrations prior to any possible discharges to Waipapa Stream. Therefore, the groundwater concentration assessments are considered adequately conservative, particularly with respect to conceptualisation of groundwater flow area.

5 Summary

In summary, Stantec have carried out a Sensitivity Analysis of the groundwater model and suitability of dilution factor presented in Section 6 of the Earthtech (2025a) report. We have reviewed the environmental site setting and hydrogeology and updated the conceptual site model. Where there is uncertainty around the conceptual model and/or calculation inputs, we have used the more conservative option, to assess the 'worst case' scenario for the site. We have assessed potential environmental effects on Waipapa Stream at the boundary by taking into account the ANZG guidelines for fresh water and the 99th percentile NEMP 3.0 ecological criteria.

Based on these results and given natural attenuation processes have not been accounted for, the effects of the predicted leachate leakage at the site border and Waipapa Stream will be within relevant guidelines for the protection of freshwater.

Additionally, it's worth noting that liner integrity surveys can be done using electric dipole methods, which can identify minor defects in the constructed liner even after the soil protection layer has been placed. This would provide significant certainty regarding the integrity of the monofill liner given that such a survey would enable the location of all defects, even down to pin-prick size, to be identified, and the defects remediated.

Yours Sincerely,

Stantec New Zealand

Reference: Green Steel Hydrogeological AEE

References

Earthtech, 2025a. Monitoring Plan and Evaluation of Surface and Groundwater Effects, Green Steel Monofill, Hampton Downs, 61 Hampton Downs Road, Hampton Downs, Waikato. Earthtech Consulting Limited, dated 9 June 2025, Ref: R4424-6.

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A. Freeze and J. A. Cherry, 1979, Groundwater, Prentice-Hall.

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Pattle Delamore Partners Ltd, 2012. Karaka Rural Urban Boundary Waitemata Aquifer Recharge Assessment.

Viljevac, Z., Murphy, G., Smail, A., Crowcroft, G., and Bowden, D. 2002. South Auckland Groundwater, Kaawa Aquifer Recharge Study and Management of The Volcanic And Kaawa Aquifers.

Table 1 Stream dilution predictions against drinking water standards and ecological guidelines

Leachate Quality Predictions*									
Leachate Quality Parameter	Units	Long Term Leaching Strength	High Strength Monofill Leachate	Stream dilution assuming a flow of 1L/s	Stream dilution assuming 0.5L.s flow	ANZG (2018)	NZ Drinking Water Standard	NEMP 3.0	
						DGV ² 95% Species Protection	2022	99% species protection	95% species protection
pH	-	>7.0 <7.8	7.0 to 7.1			-	-		
PFAS	µg/l	<0.1	0.7	0.000069	0.000139	-	0.63		
PFOS	µg/l							0.00023	0.13
PFOA	µg/l							19	220
Boron	mg/l	0.6	1.9	0.000189	0.000377	0.94	2.4		
Chromium (Cr)	mg/l	<0.1	1	0.000100	0.000199	0.001	0.05		
Copper (Cu)	mg/l	<0.1	0.3	0.000030	0.000060	0.0017	2		
Iron	mg/l	<0.1	0.5	0.000050	0.000100	-	-		
Lead (Pb)	mg/l	<0.1	0.3	0.000030	0.000060	0.0044	0.01		
Manganese (Mn)	mg/l	0.1	2	0.000198	0.000396	1.9	0.4		
Nickel (Ni)	mg/l	<0.1	0.4	0.000039	0.000079	0.011	0.08		
Zinc (Zn)	mg/l	<0.1	2.8	0.000278	0.000556	0.0096	1.5		

*based on lysimeter trials set up on 27/01/2021

- Notes:
1. Default Guideline Value Freshwater Guideline - 80% Species Protection
 2. Default Guideline Value Freshwater Guideline - 95% Species Protection
 3. Bold denotes exceedance of ANZG (2018) DGVs

SPREADSHEET 1
POTENTIAL LINER LEAKAGE

Source: Qian et al. "Geotechnical Aspects of Landfill Design and Construction", Pages 119 - 126

Leakage through three liner situations considered:
 - Compacted clay liner
 - Geomembrane liner
 - Composite geomembrane / compacted clay liner

Liner section of:
 - 1.5mm HDPE
 - 5.5mm GCL ($k < 2.5 \times 10^{-11}$ m/s)
 - 300 mm thick CCL ($k < 1 \times 10^{-10}$ m/s)

Assume:
 - 2 holes (defects) of 2mm dia. each, per 4,000 m² of liner
 - South-west monofill area is 45,160 m²
 - Head of leachate on liner is 300 mm
 - Defect is in the HDPE in contact with underlying GCL
 - Situation is as described in section 4.8.3 (starting on page 122)

Use equations 4.5.3 and 4.5.4:

- For good contact between HDPE and GCL

$$Q = 0.21 \times a^{0.1} \times h^{0.9} \times k_p^{0.74} \text{ where:}$$

Q = leachate leakage rate per hole in m³/s
 a = area of the hole/defect in m²
 h = head of leachate above the liner in m
 k_p = hydraulic conductivity of the low permeability soil (GCL in this case) in m/s

- For poor contact between HDPE and GCL

$$Q = 1.15 \times a^{0.1} \times h^{0.9} \times k_p^{0.74}$$

2	defects per liner area	4000	m ²
45160	m ²		

Hole dia. (mm)	a (in mm ²)	a (in m ²)	Q ²¹	h (in mm)	h (in m)	h ^{0.9}	k _p	k _p ^{0.74}	FOR GOOD CONTACT BETWEEN HDPE AND GCL		FOR POOR CONTACT BETWEEN HDPE AND GCL			
									Q (m ³ /s/hole)	Q (litres/day/liner area)	Q (m ³ /s/hole)	Q (litres/day/liner area)		
2	3.143	3.143E-06	0.282	300	0.3	0.338	2.5E-11	1.42717E-08	2.85651E-10	22.58	0.557	1.56428E-09	123.652	3.052

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TABLE 4.3 Calculated Flow Rates through Soil Layers with a Head of Water of 0.3 m (3.3 ft) above the Liner (USEPA, 1993)

Soil Type	Flow Rate (m ³ /day)
Soil Layer (A)	107.23
Geomembrane (B)	0.00
Composite Liner (C)	107.23

FIGURE 4.14 Soil Layer (A), Geomembrane Liner (B), and Composite Liner (C)

4.8.1 Flow Rate through Compacted Clay Liner

Flow rates through compacted clay liners are calculated using Darcy's law, which is the basic equation used to describe the flow of fluids through porous materials. Darcy's law states that:

$$Q = A \cdot i \cdot v$$

where:
 Q = flow rate through the liner, cm³/sec;
 A = hydraulic conductivity of the soil, cm/sec;
 i = hydraulic gradient; and
 v = area over which flow occurs, cm².

If the soil is saturated and there is no oil content, the hydraulic gradient is given by:

$$i = (h - D) / D$$

where:
 i = hydraulic gradient;
 h = leachate head over the liner (see Figure 4.15); and
 D = thickness of the soil layer.

For example, if 1 ft (0.3 m) of liquid is ponded on a 2-ft (0.61 m) thick soil liner that has a hydraulic conductivity of 1.0 × 10⁻¹⁰ cm/sec, the flow rate is 120 gal/acre/day (0.115 liter/m²/day). If the hydraulic conductivity is increased or decreased, the flow rate is changed proportionately (see Table 4.3).

FIGURE 4.15 Liquid Flow through a Composite Liner with poor contact between geomembrane and compacted clay liner.

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TABLE 4.4 Calculated Flow Rates through Soil Layers with a Head of Water of 0.3 m (3.3 ft) above the Liner (USEPA, 1993)

Hydraulic Conductivity (cm/sec)	Flow Rate (m ³ /day)	Flow Rate (litres/day)
1.0 × 10 ⁻¹⁰	1.20	1.12
1.0 × 10 ⁻¹¹	1.20	1.12
1.0 × 10 ⁻¹²	1.20	1.12
1.0 × 10 ⁻¹³	1.20	1.12

4.8.2 Flow Rate through Geomembrane Liner

The second liner depicted in Figure 4.14 is a geomembrane liner. It was assumed (USEPA, 1993) that the geomembrane has one or more circular holes (defects) in the liner, that the holes are sufficiently widely spaced that leakage through each hole occurs independently from the other holes, that the head of liquid ponded above the liner is constant, and that the soil that underlies the geomembrane has a relatively large hydraulic conductivity (i.e., the soil offers no resistance to flow through a hole in the geomembrane). In this case, flow rates through holes in geomembranes can be estimated using the Bernoulli equation assuming the size and shape of the holes are known. The Bernoulli equation is:

$$Q = C_d \cdot a \cdot \sqrt{2g \cdot h} \quad (4.5)$$

where:
 Q = flow rate through geomembrane, cm³/sec;
 C_d = flow coefficient with a value approximately 1.0 for a circular hole;
 a = area of a circular hole in geomembrane, cm²;
 g = acceleration due to gravity, 981 cm/sec²; and
 h = liquid head above the liner, cm.

For example, if there is a single hole with an area of 1 cm² and the head is 30 cm (1 ft), the calculated rate of flow is 3,000 gal/acre/day (12,500 liter/day). If there is one hole per acre (2.5 holes per hectare), then the flow rate is 3,000 gal/acre/day (12,500 liter/day). Flow rates for other orifices are calculated (USEPA, 1993) to Table 4.7. Ground and Bouquer (1989) report that with good quality control, one hole per acre (2.5 holes per hectare) is typical. With poor quality control, 30 holes per acre (75 holes per hectare) is possible. They also note that most orifices are small (< 0.1 cm), but that larger holes are occasionally observed. In calculating the rate of flow for "no holes" in Table 4.7, it was assumed (USEPA, 1993) that any flux of liquid was controlled by water vapor transmission that is 0.01 gal/acre/day (0.35 × 10⁻¹⁰ liter/m²/day) corresponds to a typical water vapor transmission rate for a 6-mil (15-μm) HDPE geomembrane. It is important to note that this information is based on the investigation of geomembrane installation prior to 1989. With wider widths, craters, inclusions, and construction quality assurance, these estimates are generally very high.

FIGURE 4.15 Liquid Flow through a Composite Liner with poor contact between geomembrane and compacted clay liner.

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TABLE 4.3 Calculated Flow Rates through a Geomembrane with a Water Head of 1 foot (0.3 m) above the Geomembrane (USEPA, 1993)

Soil Type	Flow Rate (m ³ /day)
Soil Layer (A)	107.23
Geomembrane (B)	0.00
Composite Liner (C)	107.23

FIGURE 4.14 Soil Layer (A), Geomembrane Liner (B), and Composite Liner (C)

4.8.3 Flow Rate through Composite Liner

The third type of liner depicted in Figure 4.14 is a composite geomembrane/compacted clay liner. A composite liner is a liner composed of a geomembrane layer and a layer of low-permeability soil placed in intimate contact with the geomembrane. Composite liners have been widely used in both hazardous and municipal solid waste facilities.

Leakage through a composite liner can result from flow through geomembrane defects or permeation through the geomembrane. In the case of a geomembrane defect, the rate of leakage through a composite liner is significantly less than the rate of leakage through a similar defect in a geomembrane placed on a high-permeability soil like sand or gravel (Gronow and Bado-Trochob, 1992).

The contained liquid in the geomembrane side of the composite liner is headless. If there is a defect in the geomembrane, the liquid flows first through the geomembrane defect, laterally some nominal distance between the geomembrane and the low-permeability soil, and finally, into and through the low-permeability soil layer, as shown in Figure 4.15. Flow in the space between the geomembrane and the soil, if there is such a space, is called interflow, and the area covered by the interflow flow is called the weep area. The flow in the soil is assumed to be vertical and h is the radius of the weep area.

The quality of the contact between the two components of a composite liner (i.e., the geomembrane and the low-permeability soil) is one of the key factors governing the rate of flow through the composite liner, because it governs the radius of the weep area (Figure 4.15). Good and poor contact conditions have been characterized by Bouquer et al. (1989) as follows:

(a) Good contact conditions correspond to a geomembrane installed with as few waves or wrinkles as possible, on top of a low-permeability soil layer that has been adequately compacted and free of air voids and inclusions.

(b) Poor contact conditions correspond to a geomembrane that has been installed with a certain number and size of wrinkles and/or defects on a low-permeability soil that has not been well compacted and is not smooth.

Other factors affecting the rate of flow (Gronow and Bado-Trochob, 1992) through a composite liner with a hole in a defect, the hydraulic conductivity of the low-permeability soil underlying the geomembrane, and the head of liquid on top of

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FIGURE 4.15 Liquid flow through a composite liner.

4.8.3 Flow Rate through Composite Liner

The third type of liner depicted in Figure 4.14 is a composite geomembrane/compacted clay liner. A composite liner is a liner composed of a geomembrane layer and a layer of low-permeability soil placed in intimate contact with the geomembrane. Composite liners have been widely used in both hazardous and municipal solid waste facilities.

Leakage through a composite liner can result from flow through geomembrane defects or permeation through the geomembrane. In the case of a geomembrane defect, the rate of leakage through a composite liner is significantly less than the rate of leakage through a similar defect in a geomembrane placed on a high-permeability soil like sand or gravel (Gronow and Bado-Trochob, 1992).

The contained liquid in the geomembrane side of the composite liner is headless. If there is a defect in the geomembrane, the liquid flows first through the geomembrane defect, laterally some nominal distance between the geomembrane and the low-permeability soil, and finally, into and through the low-permeability soil layer, as shown in Figure 4.15. Flow in the space between the geomembrane and the soil, if there is such a space, is called interflow, and the area covered by the interflow flow is called the weep area. The flow in the soil is assumed to be vertical and h is the radius of the weep area.

The quality of the contact between the two components of a composite liner (i.e., the geomembrane and the low-permeability soil) is one of the key factors governing the rate of flow through the composite liner, because it governs the radius of the weep area (Figure 4.15). Good and poor contact conditions have been characterized by Bouquer et al. (1989) as follows:

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Other factors affecting the rate of flow (Gronow and Bado-Trochob, 1992) through a composite liner with a hole in a defect, the hydraulic conductivity of the low-permeability soil underlying the geomembrane, and the head of liquid on top of

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TABLE 4.3 Calculated Flow Rates for Composite Liners with a Head of Water of 1 foot (0.3 m) above the Liner (USEPA, 1993)

Soil Type	Flow Rate (m ³ /day)
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Composite Liner (C)	107.23

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Leakage through a composite liner can result from flow through geomembrane defects or permeation through the geomembrane. In the case of a geomembrane defect, the rate of leakage through a composite liner is significantly less than the rate of leakage through a similar defect in a geomembrane placed on a high-permeability soil like sand or gravel (Gronow and Bado-Trochob, 1992).

The contained liquid in the geomembrane side of the composite liner is headless. If there is a defect in the geomembrane, the liquid flows first through the geomembrane defect, laterally some nominal distance between the geomembrane and the low-permeability soil, and finally, into and through the low-permeability soil layer, as shown in Figure 4.15. Flow in the space between the geomembrane and the soil, if there is such a space, is called interflow, and the area covered by the interflow flow is called the weep area. The flow in the soil is assumed to be vertical and h is the radius of the weep area.

The quality of the contact between the two components of a composite liner (i.e., the geomembrane and the low-permeability soil) is one of the key factors governing the rate of flow through the composite liner, because it governs the radius of the weep area (Figure 4.15). Good and poor contact conditions have been characterized by Bouquer et al. (1989) as follows:

(a) Good contact conditions correspond to a geomembrane installed with as few waves or wrinkles as possible, on top of a low-permeability soil layer that has been adequately compacted and free of air voids and inclusions.

(b) Poor contact conditions correspond to a geomembrane that has been installed with a certain number and size of wrinkles and/or defects on a low-permeability soil that has not been well compacted and is not smooth.

Other factors affecting the rate of flow (Gronow and Bado-Trochob, 1992) through a composite liner with a hole in a defect, the hydraulic conductivity of the low-permeability soil underlying the geomembrane, and the head of liquid on top of

Section 4.8 Assessment of Leakage Through Liners 125

TABLE 4.4 Calculated Flow Rates through Soil Layers, Geomembrane Liners, and Composite Liners with Poor Contact (USEPA, 1993)

Type of Liner	Assumed Values of Key Parameters	Flow Rate (m ³ /day)
Compacted Soil	$k_p = 1 \times 10^{-10}$ cm/sec	1.200
Geomembrane	$k_p = 1 \times 10^{-10}$ cm/sec	0.000
Composite Liner	$k_p = 1 \times 10^{-10}$ cm/sec	1.200

FIGURE 4.15 Liquid flow through a composite liner.

4.8.3 Flow Rate through Composite Liner

The third type of liner depicted in Figure 4.14 is a composite geomembrane/compacted clay liner. A composite liner is a liner composed of a geomembrane layer and a layer of low-permeability soil placed in intimate contact with the geomembrane. Composite liners have been widely used in both hazardous and municipal solid waste facilities.

Leakage through a composite liner can result from flow through geomembrane defects or permeation through the geomembrane. In the case of a geomembrane defect, the rate of leakage through a composite liner is significantly less than the rate of leakage through a similar defect in a geomembrane placed on a high-permeability soil like sand or gravel (Gronow and Bado-Trochob, 1992).

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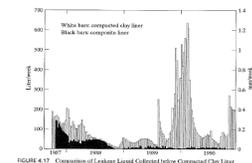
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ESTIMATE OF MAXIMUM LEACHATE HEAD ON LINER

Source: Qian et al: "Geotechnical Aspects of Landfill Design and Construction": Pages 275 - 289

Moore's 1980 Method

$$Y_{max} = L \cdot (r/k)^{1/2} \{ [k \cdot S^2 / r] + 1 - [k \cdot S / r] \cdot [S^2 + r/k]^{1/2} \}$$

AR	1,400	= Annual rainfall (mm/year)
L	85000	= horizontal drainage distance (mm)
r = AR/secs in Yr	4.43937E-05	= vertical inflow rate to drainage layer (mm/s)
k	1	= hydraulic conductivity of the drainage layer (mm/s)
a	1.1458	= slope angle of drainage layer, measured from horizontal in degrees (2% slope)
S	0.020008701	= slope of the drainage layer, S = tana
Y _{max}	291	= maximum liquid head on the landfill barrier (mm)

Moore's 1983 Method

$$Y_{max} = L \cdot [(r/k + S^2)^{1/2} - S]$$

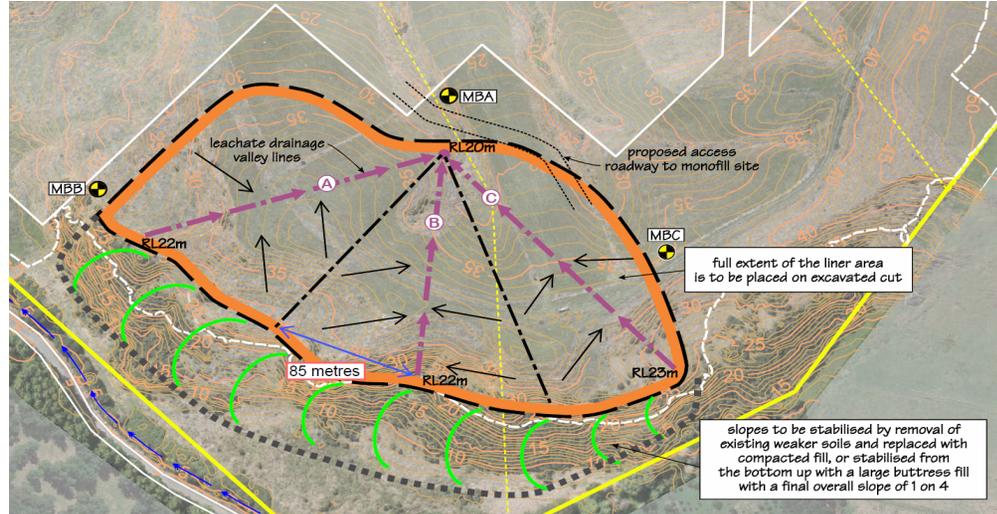
AR	1,400	= Annual rainfall (mm/year)
L	85000	= horizontal drainage distance (mm)
r = AR/secs in Yr	4.43937E-05	= vertical inflow rate to drainage layer (mm/s)
k	1	= hydraulic conductivity of the drainage layer (mm/s)
a	1.1458	= slope angle of drainage layer, measured from horizontal in degrees (2% slope)
S	0.020008701	= slope of the drainage layer, S = tana
Y _{max}	92	= maximum liquid head on the landfill barrier (mm)

Giroud's 1992 Method (Preferred Method)

$$Y_{max} = j \cdot L \cdot [4 \cdot r/k + S^2]^{1/2} - S] / (2 \cdot \cos a)$$

$$j = 1 - 0.12 \cdot \exp\{-[\log(1.6 \cdot r/k / S^2)]^{5/8}\}$$

AR	1,400	= Annual rainfall (mm/year)
L	85000	= horizontal drainage distance (mm)
r = AR/secs in Yr	4.43937E-05	= vertical inflow rate to drainage layer (mm/s)
k	1	= hydraulic conductivity of the drainage layer (mm/s)
a	1.1458	= slope angle of drainage layer, measured from horizontal in degrees (2% slope)
S	0.020008701	= slope of the drainage layer, S = tana
j	0.9637	= parameter in the formula
Y _{max}	155	= maximum liquid head on the landfill barrier (mm)



full extent of the liner area is to be placed on excavated cut

slopes to be stabilised by removal of existing weaker soils and replaced with compacted fill, or stabilised from the bottom up with a large buttress fill with a final overall slope of 1 on 4

Length	Slope	Max. Head (mm)
85	2%	291

< 300 Acceptable

SPREADSHEET 3
 UPDATED CALCULATION SHEET Based on Earthtech original calculation sheet dated 1 May 2025
Green Steel Monofill
Liner Leakage and Groundwater Dilution Effects Assessment
 Earthtech Assessment Calculations

INPUTS		Dilutions	
	Value	Earthtech	Sensitivity Checks
annual rainfall	1.4		
Days per yr	365	D	4
hr	24	W	330
m	60	Kv	3.4E-09
s	60	Kh	1.5E-07
		GW x-sect. area	1,320.0
		H	1.0
		L	200.0
		Hydraulic Gradient (I)	0.005
		Q/GW	85.5
		Q liner leakage	8.6
		DF (Dilution Factor)	10

Liner Leakage Rate Assumption Calculation:

Leachate Head on Liner (H)	300 mm
Total Liner Thickness (t_{leak})	1.5 mm HOPE
Total Liner Thickness (t_{leak})	5.5 mm GCL
Total Liner Thickness (t_{leak})	300 mm (2x 150mm CCLs)
Total Liner Thickness (t_{leak})	307 mm
Hydraulic Gradient (H/L)	54.8
Liner defects (Giroud):	
Liner area	45160 m ²
No Defects	22.58 <i>Corrected by Stantec.</i>
Say:	23
Defect Area	72.256631

GCL permeability 2.5E-11 m/s
 Therefore $Q_{leachate}$: 8.55571062 litres per day

Leachate to groundwater 100%

Therefore $Q_{leachate}$: **8.6 litres per day**
0.0019 m³/ha/day

PFAS	Boron (B)	Chromium	Copper	Iron	Lead
PFAS leachate 0.7 ug/l	Boron leachate 1.9 mg/L	chromium leachate 1 mg/L	copper leachate 0.3 mg/L	Iron leachate 0.5 mg/L	lead leachate 0.3 mg/L
PFAS GW 0.0 ug/l	Boron GW _(Dose 42) 0.072 mg/L	romium GW _(Dose 42) 0 mg/L	copper GW _(Dose 42) 0.0052 mg/L	Iron GW _(Dose 42) mg/L	lead GW _(Dose 42) 0.0024 mg/L
PFAS mass (through liner)					
PFAS _{leachate} 6.0 ug/day	B _{leachate} 16.3 mg/day	Cr _{leachate} 8.6 mg/day	Cu _{leachate} 2.6 mg/day	Fe _{leachate} 4.3 mg/day	Pb _{leachate} 2.6 mg/day
PFAS _{GW} 0.000 ug/day	B _{GW} 0.072 mg/day	Cr _{GW} 0.000 mg/day	Cu _{GW} 0.445 mg/day	Fe _{GW} 0.000 mg/day	Pb _{GW} 0.205 mg/day
PFAS concentration (at boundary)	B concentration (at boundary)	Cr concentration (at boundary)	Cu concentration (at boundary)	Fe concentration (at boundary)	Pb concentration (at boundary)
flux method 0.063651 ug/litre	flux method 0.173531 mg/litre	flux method 0.090929 mg/litre	flux method 0.032026 mg/litre	flux method 0.045465 mg/litre	flux method 0.029461 mg/litre
change at boundary 0.063651 ug/litre	change at boundary 0.101531 mg/litre	change at boundary 0.090929 mg/litre	change at boundary 0.026806 mg/litre	change at boundary 0.045465 mg/litre	change at boundary 0.027061 mg/litre

NOTES: (1) GW quality (below) is from the fractured aquifer system. No GW quality data is available for the shallow aquifer

Total Arsenic g/m^3	0.0016	0.0038	0.0016	0.0015
Total Boron g/m^3	0.072	0.24	0.07	0.34
Total Calcium g/m^3	9	4.9	12.2	6.6
Total Copper g/m^3	0.0052	0.0045	0.0035	0.00078
Total Iron g/m^3	6.2	5	4	0.58
Total Lead g/m^3	0.0024	0.0017	0.00142	0.00043
Total Magnesium g/m^3	2.5	2.1	2.3	0.89
Total Manganese g/m^3	0.09	0.059	0.061	0.0132
Total Potassium g/m^3	1.55	1.21	1.65	1.24
Total Sodium g/m^3	107	110	96	141
Total Zinc g/m^3	0.111	0.077	0.14	0.029
Chloride g/m^3	26	27	33	101
Nitrate-N g/m^3	-0.05	-0.05	-0.05	-0.05
Sulphate g/m^3	4.5	3.4	3.9	2.5

Manganese	Nickel	Zinc (Zn)
Manganese leachate 2 mg/L	nickel leachate 0.4 mg/L	Zinc leachate 2.5 mg/L
Manganese GW _(Dose 42) 0.09 mg/L	nickel GW _(Dose 42) #REF! mg/L	Zinc GW _(Dose 42) 0.111 mg/L
Mn _{leachate} 17.1 mg/day	Ni _{leachate} 3.4 mg/day	Zn _{leachate} 24.0 mg/day
Mn _{GW} 7.698 mg/day	Ni _{GW} #REF! mg/day	Zn _{GW} 9.494 mg/day
Mn concentration (at boundary)	Ni concentration (at boundary)	Zn concentration (at boundary)
flux method 0.263675 mg/litre	flux method #REF! mg/litre	flux method 0.355509 mg/litre
change at boundary 0.173675 mg/litre	change at boundary #REF! mg/litre	change at boundary 0.244509 mg/litre

SPREADSHEET 4

Dilution calculation for stream with flow of 1L/s

$$C_{\text{diluted}} = \frac{\text{mass per day}}{86,400 \text{ L/day}}$$

	Loading per day	Unit	Stream dilution	Unit
PFAS	6.0	ug/day	6.94444E-05	ug/L
Boron	16.3	mg/day	0.000188657	mg/L
Chromium (Cr)	8.6	mg/day	9.9537E-05	mg/L
Copper (Cu)	2.6	mg/day	3.00926E-05	mg/L
Iron	4.3	mg/day	4.97685E-05	mg/L
Lead (Pb)	2.6	mg/day	3.00926E-05	mg/L
Manganese (Mn)	17.1	mg/day	0.000197917	mg/L
Nickel (Ni)	3.4	mg/day	3.93519E-05	mg/L
Zinc (Zn)	24.0	mg/day	0.000277778	mg/L

Dilution calculation for stream with flow of 0.5L/s

	Loading per day	Unit	Stream dilution	Unit
PFAS	6.0	ug/day	0.000138889	ug/L
Boron	16.3	mg/day	0.000377315	mg/L
Chromium (Cr)	8.6	mg/day	0.000199074	mg/L
Copper (Cu)	2.6	mg/day	6.01852E-05	mg/L
Iron	4.3	mg/day	9.9537E-05	mg/L
Lead (Pb)	2.6	mg/day	6.01852E-05	mg/L
Manganese (Mn)	17.1	mg/day	0.000395833	mg/L
Nickel (Ni)	3.4	mg/day	7.87037E-05	mg/L
Zinc (Zn)	24.0	mg/day	0.000555556	mg/L