10 June 2022

Project No. 70097470.600/A.0

Richard Terry
Engineering Manager - Landfill
Veolia Environmental Services
Email Submission: richard.terry@veolia.com

BLUE HAZE LANDFILL SITE: STABILITY RISK ASSESSMENT: SCHEDULE 5 RESPONSE

Dear Richard

Further to your request of providing responses to the Schedule 5 comments received from the Environment Agency (EA) on the Stability Risk Assessment (SRA) report for the Blue Haze Landfill Site, we have prepared our formal response below.

EA Comment No.8

Provide an updated Stability Risk Assessment for the new cap, particularly on the steepest flanks and side slopes to:

a. Describe the construction method and sequence of the new cap.

b. Assess the effect of vehicle loading on stability of the capping system.

c. Assess the effect of build-up of gas pressure underneath the cap.

Reason: It is not clear that the construction method of the new cap, effect of vehicle loading on its stability, and the effects of gas pressure on the new cap have been taken into account. These are not mentioned in the risk screening part of the SRA, nor considered in the subsequent analysis sections.

WSP Golder Response

Capping Construction Method/Sequence

We envisage that the new Geosynthetic Clay Liner (GCL) cap will be constructed in accordance with the following sequence:

- Placement of waste regulating layer;
- Installation of Geosynthetic Clay Liner;
- Placement of 600 mm thick lower protection soil layer; and
- Placement of 900 mm thick final restoration soils.

A capping CQA Plan detailing the construction method of the new cap will be submitted to the EA for approval prior to the construction work. This will require the restoration soils to be placed from the toe of the slope upwards. To minimum the effect of plant/vehicle loading on underlying GCL and slope stability, placing soils from the upper side of the slope down gradient must be always avoided.

Vehicle Loading Analysis

Whilst we do not consider the effect of vehicle loading significant if good construction practice as indicated above is followed, further capping slope stability analysis has been carried out to include the potential adverse effect of plant/vehicle loading. The most critical capping slope geometry (Section B, pre-settlement profile) same as the one adopted in the original SRA has been analysed.

The effect of vehicle/plant loading has been assessed using the method proposed by Koerner & Daniel (1997)[1]. A weight of a typical CAT D5H LGP of 201 kN has been applied in the analysis to simulate the action of pushing soil upwards from the toe of the slope. Table 1 presents the results of the vehicle loading analysis and the detailed calculations are included in Attachment 1.

Table 1: Summary of Vehicle Loading Analysis

Analysis Scenario	Factor of Safety against Cover Soil Sliding	Factor of Safety against GCL Rupture
Without vehicle loading	2.75	Infinite
With vehicle loading	2.65	Infinite

As can be seen in the table, the baseline case without vehicle loading gives a factor of safety of 2.75 against cover soil sliding. The factor of safety against GCL rupture is infinite indicating no tension being mobilised with GCL. These factors of safety are consistent with the results of the GCL cap analysis under the dry conditions (i.e. PSR = 0) in the original SRA. It is noted that the cover soils should always be placed under the dry conditions during capping construction and therefore a PSR value of 0 is considered appropriate for the vehicle loading analysis.

When a vehicle loading is applied to the analysis, the factor of safety against cover soil sliding reduces slightly to 2.65 and the factor of safety against GCL rupture remains infinite. The stability of the GCL cap is therefore considered satisfactory under the effect of vehicle loading during construction.

Gas Pressure Analysis

Given the site has a full operational landfill gas extraction system, we do not anticipate significant gas pressure build-up which could adversely affect the stability of the GCL cap. However, further capping slope stability analysis has been carried out to include the potential adverse effect of the gas pressure build-up underneath the GCL cap.

The effect of gas pressure build-up has been assessed using the method proposed by Jones & Dixon (1998)[2]. A typical gas pressure of 5 kPa for a landfill site without gas extraction has been considered in the analysis. Table 2 presents the results of the gas pressure analysis, and the detailed calculations are included in Attachment 2.

[1] Koerner R.M. & Daniel D.E. (1997). Final Covers for Solid Waste Landfills and Abandoned Dumps, Thomas Telford, London.

[2] Jones D.R.V & Dixon N (1998). The Stability of Geosynthetic Landfill Lining System, pp 99-117, Geotechnical Engineering of Landfills. Thomas Telford, London.

Table 2: Summary of Gas Pressure Analysis

Description		Factor of Safety against Cover Soil Sliding	Factor of Safety against GCL Rupture
Section B: 23 m high and 1v:6h slope Gas Pressure = 5 kPa	PSR = 0	2.23	Infinite
	PSR = 0.25	1.87	Infinite
	PSR = 0.5	1.53	Infinite

As can be seen in the table, with a gas pressure of 5 kPa, the factor of safety against cover soil sliding calculated for the dry conditions (i.e. PSR = 0) is 2.23. When PSR values of 0.25 and 0.5 are applied, the factors of safety reduce to 1.87 and 1.53. These values are all above the minimum required 1.3 and therefore considered satisfactory. The factors of safety against GCL rupture are all calculated as infinite indicating there will be no tension developed within GCL.

It is noted that the factor of safety will further reduce and eventually become less than 1.3 if higher PSR values are applied to the analysis. Whilst the likelihood of gas pressure build-up is low due to the presence of an operational gas extraction system on site, it is still considered prudent to implement an effective surface water and drainage system on site to keep the restoration soil in relatively dry conditions. This should be considered in the detailed capping design stage. In addition, site-specific interface share strength should be carried out in the construction stage to verify the share strength values adopted in the above analyses.

EA Comment No.9

Provide assessment of the effect from additional loading on the landfill in-waste leachate monitoring and management infrastructure, including the basal drainage blanket pipework, and that for gas management.

Reason: The effects of additional loading on the pollution control infrastructure and the potential additional deformation of the wells and pipework especially on the flanks of the landfill have not been assessed. The SRA Draft Schedule 5(2) – 25/04/22 5 requires additional details on the effect of the proposals on the landfill in-waste leachate monitoring and management infrastructure, including the basal drainage blanket pipework, and for gas management.

WSP Golder Response

As requested, an assessment of the effect from the additional waste loading on the basal leachate drainage blanket pipework has been carried out and the pipework deflection analysis are presented below. As for the gas management system, we anticipate that the gas extraction wells to be extended above the new waste then reconnect to the system with renewed pipework above the new cap. As such, we do not envisage the increased waste depth will have any effect on the gas pipework and therefore no further analysis is considered necessary for the gas management system.

The leachate pipework deflection analysis has been carried out in accordance with the approach proposed in Qian *et al.* (2003)[3]. The input parameters for the pipework are based on the information from the construction records of the previous landfill cells. Given the uncertainties of the compaction quality of the pipe surrounding material, two scenarios considering both 85% and 95% standard proctor densities are analysed.

[3] Qian X., Koerner R.M. and Gray D.H. (2002). Geotechnical Aspects of Landfill Design and Construction. Prentice Hall.

Table 3 presents a summary of the pipework deflection analysis for the existing waste height of approximately 28 m. The detailed analysis is presented in Attachment 3. As can be seen in the table, the pipe deflection ratio calculated for both pipes are 1.2% for 85% standard proctor density and 0.4% for 95% standard proctor density. These values are less than the maximum allowable deflection ratio of 2.7% for polyethylene pipes with a SDR value of 11 in accordance with Qian *et al.* (2003). This is considered satisfactory.

Table 3: Leachate Pipework Deflection Results – Existing Waste

Description	85% Standard Proctor Density		95% Standard Proctor Density	
	(mm)	(%)	(mm)	(%)
Leachate pipe with an internal diameter of 180 mm	2.16	1.2	0.64	0.4
Leachate pipe with an internal diameter of 355 mm	4.27	1.2	1.26	0.4

Table 4 presents a summary of the pipework deflection analysis for the increased waste height of approximately 33 m. The detailed analysis is presented in Attachment 3. As can be seen in the table, the pipe deflection ratio calculated for both pipes increase slightly to 1.4% for 85% standard proctor density and 0.4% for 95% standard proctor density. These values are still less than the maximum allowable deflection ratio of 2.7%. This is therefore considered satisfactory.

Table 4: Leachate Pipework Deflection Results – Additional Waste

Description	85% Standard Proctor Density		95% Standard Proctor Density	
	(mm)	(%)	(mm)	(%)
Leachate pipe with an internal diameter of 180 mm	2.51	1.4	0.74	0.4
Leachate pipe with an internal diameter of 355 mm	4.96	1.4	1.46	0.4

EA Comment No.10

Provide the cross-section B and show the numerical model lines of the section on the plan views also, to demonstrate the worst-case slope and conditions have been assessed.

Reason: For the site conceptual model representation, Cross section B is not presented, and numerical model lines of section need to be shown on plan views too, to demonstrate the worst-case slope and conditions have been assessed.

WSP Golder Response

As requested, a full cross section B has now been provided in Attachment 4. It is noted that the pre-settlement contours (i.e. blue line in the cross section) have been used to derived the most critical capping slope geometry adopted in capping slope stability analyses.

EA Comment No.11

Provide justification on how the interface parameter values for the capping materials in Table 4 have been derived.

Reason: It is not clear how the interface parameter values for the capping materials in Table 4 have been derived. These need to be justified. Note these will be verified during the CQA process.

WSP Golder Response

As indicated in the capping analysis sheets, the interface parameters used in the analysis have been derived from a summary of the technical literature on interface shear strengths reported by Jones & Dixon (1998)[4] in conjunction with WSP Golder's in-house experience.

We confirm that these adopted values shall be verified by site-specific interface shear strength testing during the construction stage via the CQA process.

Closing

We trust that you will find our response to your queries satisfactory; however, if you have any further queries, please do not hesitate to contact us.

Yours sincerely

WSP Golder

Dr B Zhang
Associate Director

Dr DRV Jones
Commercial Director

Author: WY Htike/BZ/DRVJ/ab

Attachments: Attachment 1: Vehicle Loading Analysis
Attachment 2: Gas Pressure Analysis
Attachment 3: Pipework Deflection Analysis
Attachment 4: Cross Section B

[4] Jones D.R.V & Dixon N (1998). Shear Strength Properties of Geomembrane/Geotextile Interfaces. Geotextile and Geomembranes, Vol 16, pp 45-71.

ATTACHMENT 1

Vehicle Loading Analysis

INTRODUCTION

The effect of vehicle loading on the stability of the capping system has been assessed for the final steepest capping geometry.

STABILITY

The effect of plant loading has been assessed using the method proposed by Koerner & Daniel (1997). A weight of a typical CAT D6H LGP of 201kN has been used in the analysis pushing soil upwards from the toe of the slope.

INTEGRITY

The integrity of the geosynthetic clay liner has been assessed by considering the shear strength developed above and below the geosynthetic clay liner, and comparing this to the material strength.

GEOSYNTHETICS

Analyses has been carried out assuming the capping layer comprises a a Geosynthetic Clay Liner (GCL) with 1.5 m of restoration soils.

The parameters used in the analysis have been derived from a summary of the technical literature on interface shear strengths reported by Jones & Dixon (1998). Based on this and our experience of geosynthetic interfaces, a conservative assessment of the interface shear strength parameters is:

- Cover soils / GCL α_p' = 0 kPa δ_p' = 24 Deg.
- GCL / Blinding layer α_p' = 0 kPa δ_p' = 24 Deg.

These values should be confirmed by site-specific shear strength testing at the detailed design stage. In addition, the values given above are all peak shear strengths.

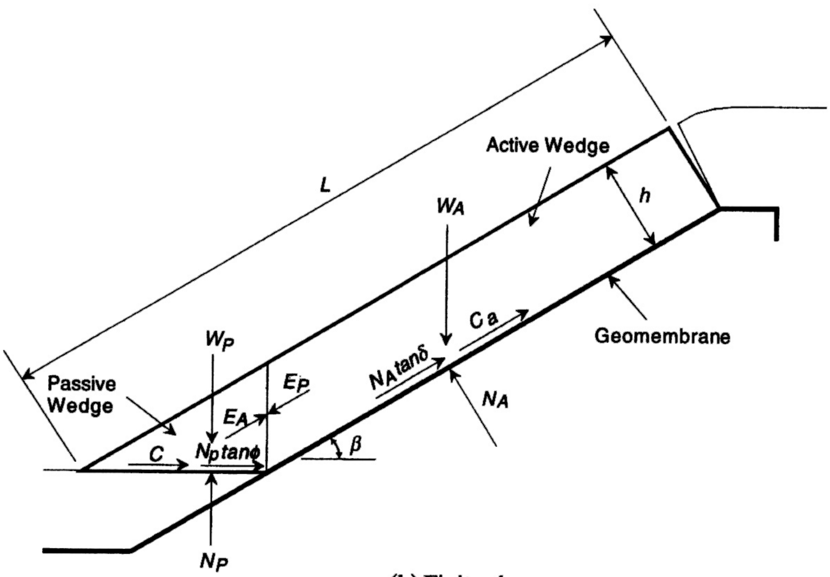
The tensile strength of the GCL has been taken from the GCL product (Naue Bentofix NSP 4300) used for the 2015 permanent capping works of the Site. A copy of the relevant section is given in the reference page.

Section **B** Without Vehicle Loading

Aim: To assess the effect of vehicle loading on stability of cover soils and integrity of geosynthetic

Approach: Use the approach proposed by Koerner & Daniels, 1997.

Geometry:



(b) Finite slope

Input Parameters

Cover soil unit weight, γ	18	kN/m^3
Cover soil internal shear strength, ϕ	22	Deg.
Cover soil cohesion, c	0	kPa
Thickness of cover soil, h	1.5	m
Height of slope, H	23	m
Slope angle, β	9.5	Deg.

Geosynthetic interface shear strengths:

Cover soil/GCL friction angle, δ_1	24	Deg.
Cover soil/GCL cohesion intercept, α_1	0	kPa
GCL/Blinding Layer friction angle, δ_2	24	Deg.
GCL/Blinding Layer cohesion intercept, α_2	0	kPa

I (Influence Factor)	0	
Wb (Buldozer Weight) (CAT D6H LGP)	201	kN
w (Track Length)	3.2	m
b (Track Width)	0.91	m
Force per unit area	34.5124	kPa
Equivalent Force/ unit width	0	kN/m
acceleration of plant	2	m/s^2
acceleration due to gravity	9.81	m/s^2
Dynamic Force per unit width	0	
Effective Equipment Force normal to failure Plane	0	
Cohesive Force Along Failure plane of Passive Wedg	0	

Geosynthetic tensile strengths:

GCL	12	kN/m

1. Stability of Cover Soil

Calculated Parameters

Length of slope, L	139.354	m
Weight of active wedge, W_A	3638.15	kN
Weight of passive wedge, W_P	124.398	kN
Pore pressure perp. to slope, U_n	0	
Pore pressure in interwedge surface, U_h	0	
Force normal to active wedge, N_A	3588.26	kN
a	592.233	
b	-1665.99	
c	106.533	

Factor of Safety against cover soil sliding 2.75

2. Integrity of Geosynthetics

(i) GCL

Mobilised shear stress at upper interface	601.335	kN
Shear strength at lower interface	1652.22	kN
Tension developed in the GCL	0	kN
Tensile strength of the GCL	12	kN

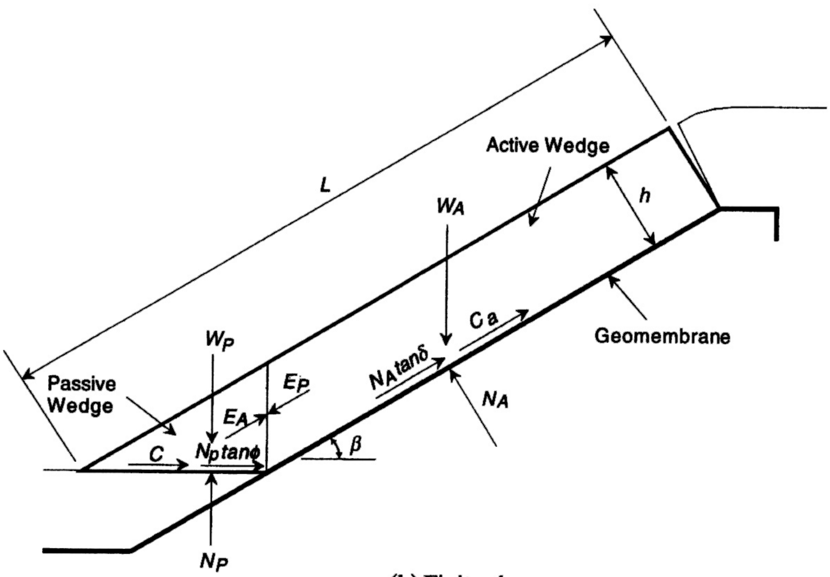
Factor of Safety against rupture Infinite

Section **B** With Vehicle Loading

Aim: To assess the effect of vehicle loading on stability of cover soils and integrity of geosynthetic

Approach: Use the approach proposed by Koerner & Daniels, 1997.

Geometry:



(b) Finite slope

Input Parameters

Cover soil unit weight, γ	18	kN/m^3
Cover soil internal shear strength, ϕ	22	Deg.
Cover soil cohesion, c	0	kPa
Thickness of cover soil, h	1.5	m
Height of slope, H	23	m
Slope angle, β	9.5	Deg.

Geosynthetic interface shear strengths:

Cover soil/GCL friction angle, δ_1	24	Deg.
Cover soil/GCL cohesion intercept, α_1	0	kPa
GCL/Blinding Layer friction angle, δ_2	24	Deg.
GCL/Blinding Layer cohesion intercept, α_2	0	kPa

PROJECT	Blue Haze Landfill SRA Schedule 5 Response			
Job No. 70097470		Made By: WYH	Date: 07/06/2022	
Ref. Attachment 1		Checked: BZ	Sheet: 5	
		Reviewed DRVJ	of: 6	

I (Influence Factor)		1	
Wb (Buldozer Weight)	(CAT D6H LGP)	201	kN
w (Track Length)		3.2	m
b (Track Width)		0.91	m
Force per unit area		34.5124	kPa
Equivalent Force/ unit width		110.44	kN/m
acceleration of plant		2	m/s^2
acceleration due to gravity		9.81	m/s^2
Dynamic Force per unit width		22.5157	
Effective Equipment Force normal to failure Plane		108.925	
Cohesive Force Along Failure plane of Passive Wedg		0	

Geosynthetic tensile strengths:

GCL	12	kN/m

1. Stability of Cover Soil

Calculated Parameters

Length of slope, L	139.354	m
Weight of active wedge, W_A	3638.15	kN
Weight of passive wedge, W_P	124.398	kN
Pore pressure perp. to slope, U_n	0	
Pore pressure in interwedge surface, U_h	0	
Force normal to active wedge, N_A	3588.26	kN
a	632.418	
b	-1716.53	
c	109.767	

Factor of Safety against cover soil sliding **2.65**

2. Integrity of Geosynthetics

(i) GCL

Mobilised shear stress at upper interface	623.783	kN
Shear strength at lower interface	1652.22	kN
Tension developed in the GCL	0	kN
Tensile strength of the GCL	12	kN

Factor of Safety against rupture **Infinite**

Fibre-reinforced Geosynthetic Clay Liner (GBR-C)

Bentofix® NSP 4300

0799-CPR-20

NAUE GmbH & Co. KG
Gewerbestrasse 2
32339 Espelkamp-Fiestel
Germany
Phone:+49 5743 41-0 Fax: +49 5743 41-240
E-Mail: info@naue.com Internet: www.naue.com

Bentofix® NSP 4300 is a shear strength transmitting geosynthetic clay barrier (GBR-C), continuously needle-punched through all components. A GBR-C is also known as geosynthetic clay liner (GCL) or bentonite mat. Additional bentonite powder is impregnated into a 500 mm overlapping area on both longitudinal sides of the cover layer. The 300 mm length longitudinal overlapping areas are marked on the carrier layer.

Property	Test method*	Unit	Values
Geotextile layers:			
Cover layer (polypropylene nonwoven):			
Mass per unit area	EN ISO 9864	g/m²	220
Carrier layer (polypropylene woven):			
Mass per unit area	EN ISO 9864	g/m²	110
Bentonite layer (sodium bentonite powder):			
Mass per unit area	EN 14196 (ρ_{CLAY})	g/m²	4,000
Swell index	ASTM D5890	ml/2g	24
Fluid Loss	ASTM D5891	ml	≤ 18
Water content	DIN 18121 / ISO 11465 (5hrs, 105 °C)	%	approx. 10
Geosynthetic Clay Liner:			
Mass per unit area	EN 14196 ($\rho_{GBR\text{-}C}$)	g/m²	4,330
Thickness	EN ISO 9863-1	mm	6.0
Max. tensile strength, md/cmd**	EN ISO 10319 / ASTM D6768	kN/m	12.0 / 12.0
Elongation at break, md/cmd**	EN ISO 10319 / ASTM D6768	%	10.0 / 6.0
Peel strength	ASTM D6496	N/10 cm***	≥ 60
		N/m	≥ 360
Static puncture strength	EN ISO 12236 / ASTM D6241	N	2,000
Permeability / Hydraulic Conductivity (k_{10})	EN 16416 / ASTM D5887	m/s	2×10^{-11}
Index Flux (q_{10})	EN 16416 / ASTM D5887	(m³/m²)/s	4.5×10^{-9}
Roll dimensions:			
width x length, / diameter	-	m x m / m	5.0 x 50 / Ø 0.65

** = based on; **md = machine direction, cmd = cross machine direction; ***max. peak; ****subject to production line*

ATTACHMENT 2

Gas Pressure Analysis

INTRODUCTION

The effect of gas pressure on the stability of the capping system has been assessed for the final steepest capping geometry.

STABILITY

The effect of gas pressure has been assessed by reducing the normal force on the interface in the finite slope with the amount of normal force generated by gas pressure. The water pressure acting on the system have been modelled using a Parallel Submergence Ratio (PSR). PSR=0 for dry conditions, PSR=0.25 for 25% partially saturated conditions and PSR=0.50 for 50% partially saturated conditions.

INTEGRITY

The integrity of the geosynthetic clay liner has been assessed by considering the shear strength developed above and below the geosynthetic clay liner, and comparing this to the material strength.

GEOSYNTHETICS

Analyses has been carried out assuming the capping layer comprises a a Geosynthetic Clay Liner (GCL) with 1.5 m of restoration soils.

The parameters used in the analysis have been derived from a summary of the technical literature on interface shear strengths reported by Jones & Dixon (1998). Based on this and our experience of geosynthetic interfaces, a conservative assessment of the interface shear strength parameters is:

- Cover soils / GCL α_p' = 0 kPa δ_p' = 24 Deg.
- GCL / Blinding layer α_p' = 0 kPa δ_p' = 24 Deg.

These values should be confirmed by site-specific shear strength testing at the detailed design stage. In addition, the values given above are all peak shear strengths.

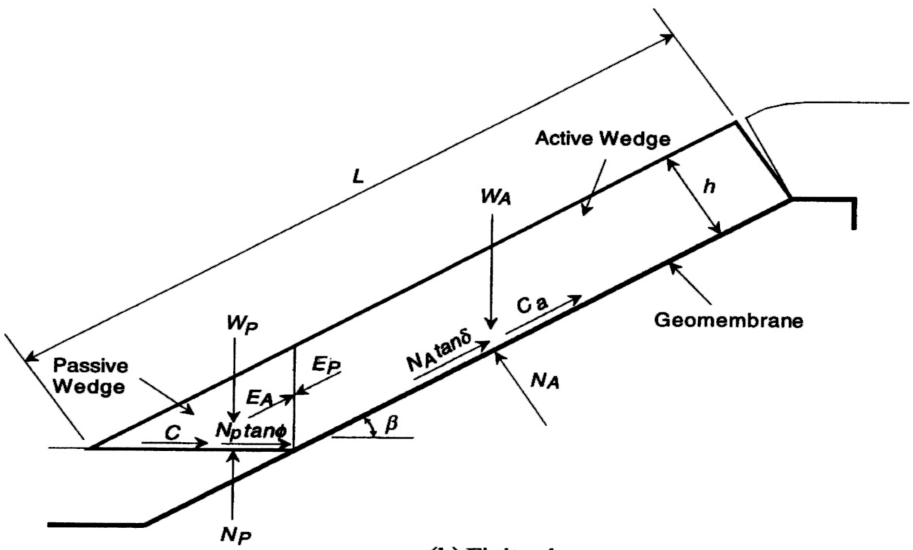
The tensile strength of the GCL has been taken from the GCL product (Naue Bentofix NSP 4300) used for the 2015 permanent capping works of the Site. A copy of the relevant section is given in the reference page.

Section **B** **PSR** = 0

Aim: To assess the effect of gas pressure on stability of cover soils and integrity of geosynthetics.

Approach: Use the approach proposed by Jones & Dixon (1998).

Geometry:



(b) Finite slope

Input Parameters

Cover soils unit weight (dry), γ_{dry}	18	kN/m^3
Cover soils unit weight (saturated), γ_{sat}	20	kN/m^3
Cover soils internal shear strength, ϕ	22	Deg.
Cover soils cohesion, c	0	kPa
Thickness of cover soils, h	1.5	m
Height of slope, H	23	m
Slope angle, β	9.5	Deg.
Geosynthetic interface shear strengths:		
Cover Soils/GCL friction angle, δ_1	24	Deg.
Cover Soils/GCL cohesion intercept, α_1	0	kPa
GCL/Blinding layer friction angle, δ_2	24	Deg.
GCL/Blinding cohesion intercept, α_2	0	kPa
Parallel submergence ratio, PSR	0	
Geosynthetic tensile strengths:		
GCL	12	kN/m
Gas pressure	5	kPa

1. Stability of Cover Soils

Calculated Parameters

Length of slope, L	139.3537	m
Thickness of water, h_w	0	m
Weight of active wedge, W_A	3638.153	kN
Weight of passive wedge, W_P	124.3979	kN
Pore pressure perp. to slope, U_n	0	kN
Pore pressure in interwedge surface, U_h	0	kN
Force normal to active wedge, N_A	3588.258	kN
Force normal to active wedge from gas pressure, N_G	696.7687	kN
Vert pp on passive wedge, U_V	0	kN
a	592.2334	
b	-1360.02	
c	85.84668	

Factor of Safety against cover soils sliding **2.23**

2. Integrity of Geosynthetics

(i) (GCL

Mobilised shear stress at upper interface	740.4195	kN
Shear strength at lower interface	1652.221	kN
Tension developed in the geosythetic	0	kN
Tensile strength of the geosythetic	12	kN

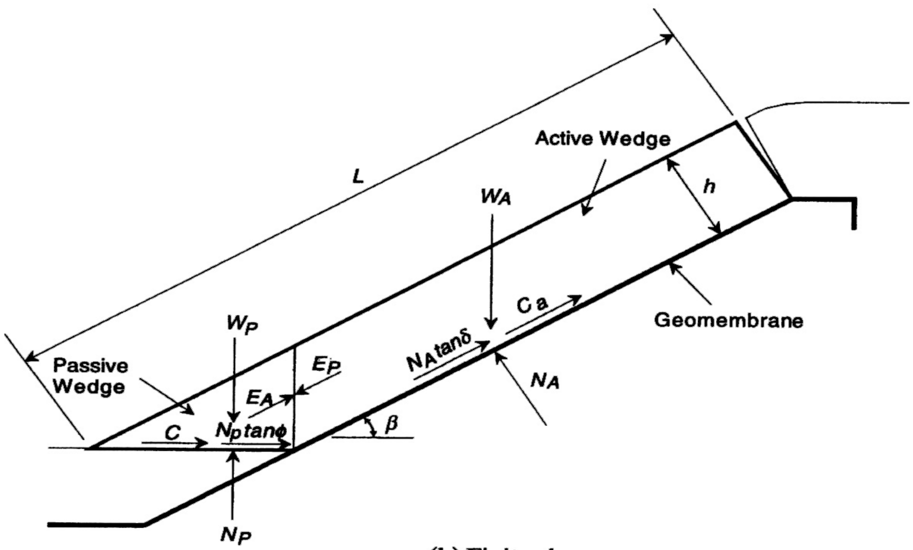
Factor of Safety against rupture **Infinite**

Section **B** **PSR** = 0.25

Aim: To assess the effect of gas pressure on stability of cover soils and integrity of geosynthetics.

Approach: Use the approach proposed by Jones & Dixon (1998).

Geometry:



(b) Finite slope

Input Parameters

Cover soils unit weight (dry), γ_{dry}	18	kN/m^3
Cover soils unit weight (saturated), γ_{sat}	20	kN/m^3
Cover soils internal shear strength, ϕ	22	Deg.
Cover soils cohesion, c	0	kPa
Thickness of cover soils, h	1.5	m
Height of slope, H	23	m
Slope angle, β	9.5	Deg.
Geosynthetic interface shear strengths:		
Cover Soils/GCL friction angle, δ_1	24	Deg.
Cover Soils/GCL cohesion intercept, α_1	0	kPa
GCL/Blinding layer friction angle, δ_2	24	Deg.
GCL/Blinding cohesion intercept, α_2	0	kPa
Parallel submergence ratio, PSR	0.25	
Geosynthetic tensile strengths:		
GCL	12	kN/m
Gas pressure	5	kPa

PROJECT	Blue Haze Landfill SRA Schedule 5 Response				
Job No.	70097470	Made By:	WYH	Date:	07/06/2022
Ref.	Attachment 2	Checked:	BZ	Sheet:	5
		Reviewed:	DRVJ	of:	8

1. Stability of Cover Soils

Calculated Parameters

Length of slope, L	139.3537	m
Thickness of water, h_w	0.375	m
Weight of active wedge, W_A	3741.804	kN
Weight of passive wedge, W_P	125.2618	kN
Pore pressure perp. to slope, U_n	511.1495	kN
Pore pressure in interwedge surface, U_h	0.703125	kN
Force normal to active wedge, N_A	3179.454	kN
Force normal to active wedge from gas pressure, N_G	696.7687	kN
Vert pp on passive wedge, U_V	4.201709	kN
a	609.1253	
b	-1180.251	
c	73.70953	

Factor of Safety against cover soils sliding **1.87**

2. Integrity of Geosynthetics

(i) GCL

Mobilised shear stress at upper interface	906.6247	kN
Shear strength at lower interface	1698.116	kN
Tension developed in the geosynthetic	0	kN
Tensile strength of the geosynthetic	12	kN

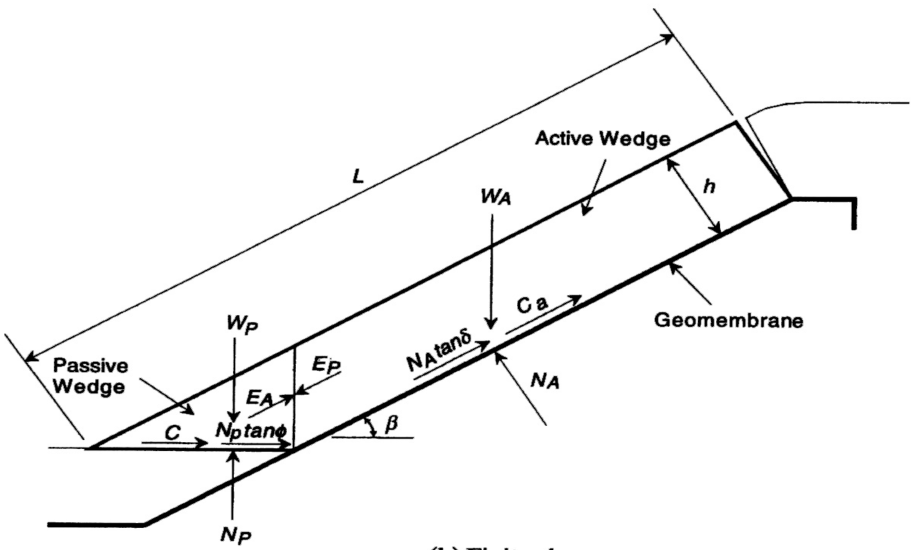
Factor of Safety against rupture **Infinite**

Section **B** **PSR** = 0.5

Aim: To assess the effect of gas pressure on stability of cover soils and integrity of geosynthetics**.**

Approach: Use the approach proposed by Jones & Dixon (1998).

Geometry:



(b) Finite slope

Input Parameters

Cover soils unit weight (dry), γ_{dry}	18	kN/m^3
Cover soils unit weight (saturated), γ_{sat}	20	kN/m^3
Cover soils internal shear strength, ϕ	22	Deg.
Cover soils cohesion, c	0	kPa
Thickness of cover soils, h	1.5	m
Height of slope, H	23	m
Slope angle, β	9.5	Deg.
Geosynthetic interface shear strengths:		
Cover Soils/GCL friction angle, δ_1	24	Deg.
Cover Soils/GCL cohesion intercept, α_1	0	kPa
GCL/Blinding layer friction angle, δ_2	24	Deg.
GCL/Blinding cohesion intercept, α_2	0	kPa
Parallel submergence ratio, PSR	0.5	
Geosynthetic tensile strengths:		
GCL	12	kN/m
Gas pressure	5	kPa

1. Stability of Cover Soils

Calculated Parameters

Length of slope, L	139.3537	m
Thickness of water, h_w	0.75	m
Weight of active wedge, W_A	3843.728	kN
Weight of passive wedge, W_P	127.8534	kN
Pore pressure perp. to slope, U_n	1013.779	kN
Pore pressure in interwedge surface, U_h	2.8125	kN
Force normal to active wedge, N_A	2777.699	kN
Force normal to active wedge from gas pressure, N_G	696.7687	kN
Vert pp on passive wedge, U_V	16.80684	kN
a	625.7743	
b	-1000.768	
c	61.78164	

Factor of Safety against cover soils sliding **1.53**

2. Integrity of Geosynthetics

(i) (GCL

Mobilised shear stress at upper interface	1136.218	kN
Shear strength at lower interface	1744.011	kN
Tension developed in the geosythetic	0	kN
Tensile strength of the geosythetic	12	kN

Factor of Safety against rupture **Infinite**

Fibre-reinforced Geosynthetic Clay Liner (GBR-C)

Bentofix® NSP 4300

NAUE

0799-CPR-20

NAUE GmbH & Co. KG
Gewerbestrasse 2
32339 Espelkamp-Fiestel
Germany
Phone:+49 5743 41-0 Fax: :+49 5743 41-240
E-Mail: info@naue.com Internet: www.naue.com

Bentofix® NSP 4300 is a shear strength transmitting geosynthetic clay barrier (GBR-C), continuously needle-punched through all components. A GBR-C is also known as geosynthetic clay liner (GCL) or bentonite mat. Additional bentonite powder is impregnated into a 500 mm overlapping area on both longitudinal sides of the cover layer. The 300 mm length longitudinal overlapping areas are marked on the carrier layer.

Property	Test method*	Unit	Values
Geotextile layers:			
Cover layer (polypropylene nonwoven):			
Mass per unit area	EN ISO 9864	g/m²	220
Carrier layer (polypropylene woven):			
Mass per unit area	EN ISO 9864	g/m²	110
Bentonite layer (sodium bentonite powder):			
Mass per unit area	EN 14196 (ρ_{CLAY})	g/m²	4,000
Swell index	ASTM D5890	ml/2g	24
Fluid Loss	ASTM D5891	ml	≤ 18
Water content	DIN 18121 / ISO 11465 (5hrs, 105 °C)	%	approx. 10
Geosynthetic Clay Liner:			
Mass per unit area	EN 14196 (ρ_{GBR-C})	g/m²	4,330
Thickness	EN ISO 9863-1	mm	6.0
Max. tensile strength, md/cmd**	EN ISO 10319 / ASTM D6768	kN/m	12.0 / 12.0
Elongation at break, md/cmd**	EN ISO 10319 / ASTM D6768	%	10.0 / 6.0
Peel strength	ASTM D6496	N/10 cm***	≥ 60
		N/m	≥ 360
Static puncture strength	EN ISO 12236 / ASTM D6241	N	2,000
Permeability / Hydraulic Conductivity (k_{10})	EN 16416 / ASTM D5887	m/s	2×10^{-11}
Index Flux (q_{10})	EN 16416 / ASTM D5887	(m³/m²)/s	4.5×10^{-9}
Roll dimensions:			
width x length, / diameter	-	m x m / m	5.0 x 50 / Ø 0.65

** = based on; **md = machine direction, cmd = cross machine direction; ***max. peak; ****subject to production line*

ATTACHMENT 3

Pipework Deflection Analysis

Leachate Pipework Strength Calculations

Aim: To assess strength of the leachate drainage pipe with a diameter of 180 mm under the existing waste loading.

Approach: To use the Modified Iowa formula to predict the long term deformation of the leachate drainage pipe.

References:

1. Environment Agency, R&D Technical Report P1-397/TR, Landfill Engineering: Leachate Drainage, Collection and Extraction Systems, September 2002.

2. Qian X., Koerner R.M., and Gray D.H., Geotechnical Aspects of Landfill Design and Construction. Prentice Hall, 2002.

The Modified Iowa Formulae can be used to predict the deformation of a pipeline at any stage in its life. The primary design limitation of long term deformation can be calculated using the following equation:

$$\delta_v = \frac{D_L K_X W_c}{(EI/r^3)+(0.061\ E')}$$

Equation 1.

Where:

W_c = Static Loading (simple prismatic loading is assumed)

= ((depth of existing waste·γ_{waste})+(restoration soil thickness·$\gamma_{restor\ soils}$))·OD of pipe

= ((28 m x 10 kN/m^3) + (1.5 m x 20 kN/m^3) +) x 0.18

= **55.8** kN/m

D_L = Deflection lag factor (dimensionless)

= **1.5** (assumed)

K_x = Bedding factor

= **0.103** (value assumed is as recommended by the Water Research Centre)

r = Mean radius of pipe

= **90** mm

t = Wall thickness of pipe

= **16.36** mm

I = Moment of inertia of pipe wall per unit length

= **365.1** mm^3

E = Modulus of elasticity of the pipe material (long term)

= **160,000** kPa

S_L = (EI/r^3) = Long-term stiffness of pipe

= **80.1** kPa

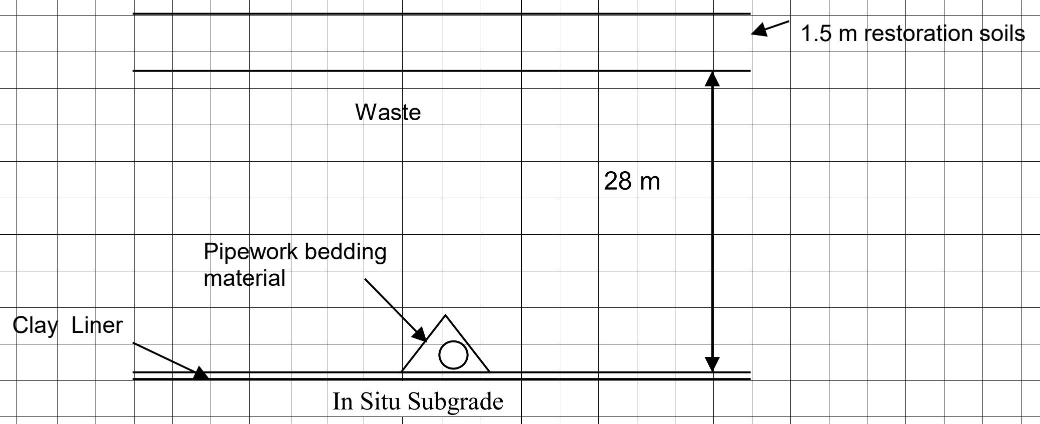
`

E' = Modulus of soil reaction under high stress conditions

= **64,000** kPa Gravel surround with little no fine compacted to 85% Standard Proctor density

= **220,000** kPa Gravel surround with little no fine compacted to 95% Standard Proctor density

Geometry:



Calculation:

From Equation (1), the pipe deflection assuming gravel surround compacted to 85% standard density is given by:

δ_v	=	**0.002**	m
	=	**2.16**	mm
	=	**1.2**	% of the nominal pipe inside diameter

The calculations indicate that under the current condition, assuming the gravel surround is compacted to 85% standard density, the leachate drainage pipe will deflect up to approximately 1.2% which is less than the maximum allowable deflection ratio of 2.7% for HDPE pipes with a SDR value of 11. This is considered satisfactory.

From Equation (1), the pipe deflection assuming gravel surround compacted to 95% standard density is given by:

δ_v	=	**0.001**	m
	=	**0.64**	mm
	=	**0.4**	% of the nominal pipe inside diameter

The calculations indicate that under the current condition, assuming the gravel surround is compacted to 95% standard density, the leachate drainage pipe will deflect up to approximately 0.4% which is less than the maximum allowable deflection ratio of 2.7% for HDPE pipes with a SDR value of 11. This is considered satisfactory.

Leachate Pipework Strength Calculations

Aim: To assess strength of the leachate drainage pipe with a diameter of 355mm under the existing waste loading.

Approach: To use the Modified Iowa formula to predict the long term deformation of the leachate drainage pipe.

References:

1 Environment Agency, R&D Technical Report P1-397/TR, Landfill Engineering: Leachate Drainage, Collection and Extraction Systems, September 2002.

2 Qian X., Koerner R.M., and Gray D.H., Geotechnical Aspects of Landfill Design and Construction. Prentice Hall, 2002.

The Modified Iowa Formulae can be used to predict the deformation of a pipeline at any stage in its life. The primary design limitation of long term deformation can be calculated using the following equation:

$$\delta_v = \frac{D_L K_X W_c}{(EI/r^3)+(0.061 E')}$$ Equation 1.

Where:

W_c = Static Loading (simple prismatic loading is assumed)
= ((depth of existing waste$\cdot\gamma_{waste}$)+(restoration soil thickness$\cdot\gamma_{restor\ soils}$))$\cdot$OD of pipe
= ((28 m x 10 kN/m^3) + (1.5 m x 20 kN/m^3) +) x 0.355
= **110.1** kN/m

D_L = Deflection lag factor (dimensionless)
= **1.5** (assumed)

K_x = Bedding factor
= **0.103** (value assumed is as recommended by the Water Research Centre)

r = Mean radius of pipe
= **177.5** mm

t = Wall thickness of pipe
= **32.27** mm

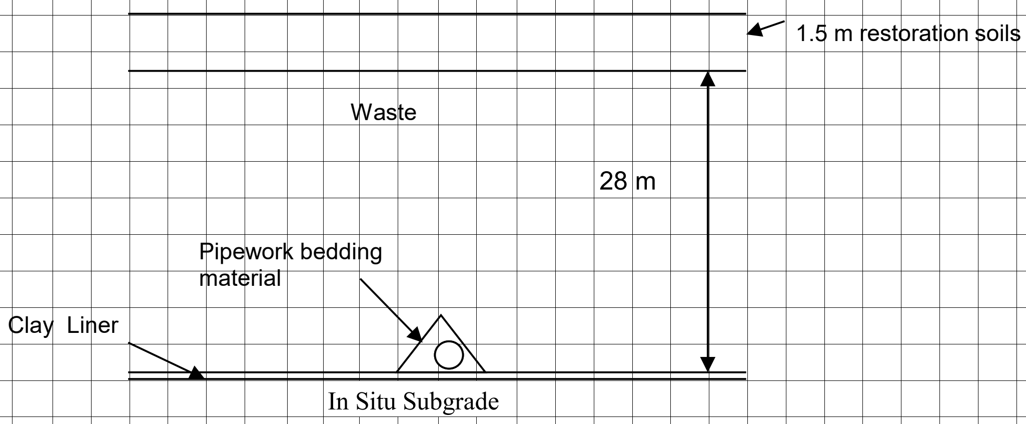
I = Moment of inertia of pipe wall per unit length
= **2801.1** mm^3

E = Modulus of elasticity of the pipe material (long term)
= **160,000** kPa

S_L = (EI/r^3) = Long-term stiffness of pipe
= **80.1** kPa
`

E' = Modulus of soil reaction under high stress conditions
= **64,000** kPa Gravel surround with little no fine compacted to 85% Standard Proctor density
= **220,000** kPa Gravel surround with little no fine compacted to 95% Standard Proctor density

Geometry:



Calculation:

From Equation (1), the pipe deflection assuming gravel surround compacted to 85% standard density is given by:

δ_v	=	**0.004**	m
	=	**4.27**	mm
	=	**1.2**	% of the nominal pipe inside diameter

The calculations indicate that under the current condition, assuming the gravel surround is compacted to 85% standard density, the leachate drainage pipe will deflect up to approximately 1.2% which is less than the maximum allowable deflection ratio of 2.7% for HDPE pipes with a SDR value of 11. This is considered satisfactory.

From Equation (1), the pipe deflection assuming gravel surround compacted to 95% standard density is given by:

δ_v	=	**0.001**	m
	=	**1.26**	mm
	=	**0.4**	% of the nominal pipe inside diameter

The calculations indicate that under the current condition, assuming the gravel surround is compacted to 95% standard density, the leachate drainage pipe will deflect up to approximately 0.4% which is less than the maximum allowable deflection ratio of 2.7% for HDPE pipes with a SDR value of 11. This is considered satisfactory.

Leachate Pipework Strength Calculations

Aim: To assess strength of the leachate drainage pipe with a diameter of 180 mm under the proposed development.

Approach: To use the Iowa formula to predict the long term deformation of the leachate drainage pipe.

References:
1. Environment Agency, R&D Technical Report P1-397/TR, Landfill Engineering: Leachate Drainage, Collection and Extraction Systems, September 2002.
2. Qian X., Koerner R.M., and Gray D.H., Geotechnical Aspects of Landfill Design and Construction. Prentice Hall, 2002.

The Modified Iowa Formulae can be used to predict the deformation of a pipeline at any stage in its life. The primary design limitation of long term deformation can be calculated using the following equation:

$$\delta_v = \frac{D_L\, K_X\, W_c}{(EI/r^3)+(0.061\ E')} \qquad \text{Equation 1.}$$

Where:

W_c = Static Loading (simple prismatic loading is assumed)

= ((depth of existing waste$\cdot\gamma_{waste}$)+(depth of future waste$\cdot\gamma_w$)+(restoration soil thickness$\cdot\gamma_{restor\ soils}$))$\cdot$OD of pipe

= ((28 m x 10 kN/m^3) + (5 m x 10 kN/m^3) + (1.5 m x 20 kN/m^3)) x 0.18

= **64.8** kN/m

D_L = Deflection lag factor (dimensionless)

= **1.5** (assumed)

K_x = Bedding factor

= **0.103** (value assumed is as recommended by the Water Research Centre)

r = Mean radius of pipe

= **90** mm

t = Wall thickness of pipe

= **16.36** mm

I = Moment of inertia of pipe wall per unit length

= **365.1** mm^3

E = Modulus of elasticity of the pipe material (long term)

= **160,000** kPa

S_L = (EI/r^3) = Long-term stiffness of pipe

= **80.1** kPa

`

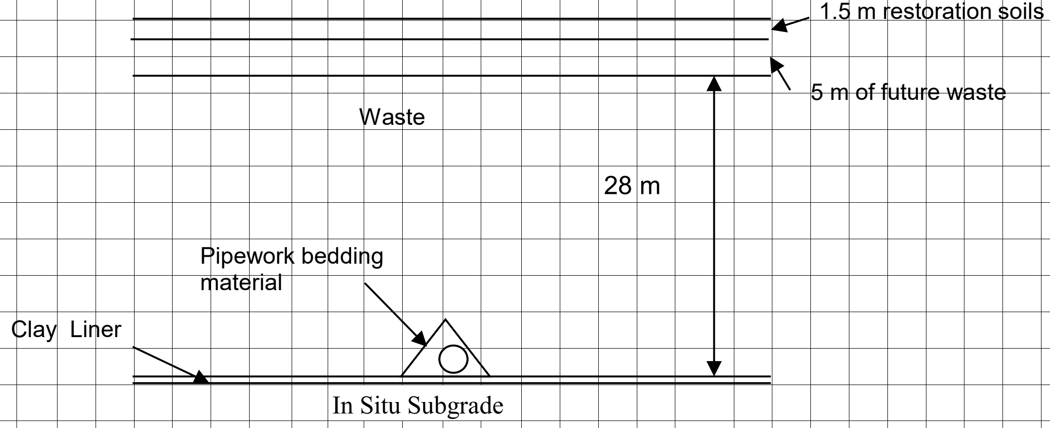
E' = Modulus of soil reaction under high stress conditions

= **64,000** kPa Gravel surround with little no fine compacted to 85% Standard Proctor density

= **220,000** kPa Gravel surround with little no fine compacted to 95% Standard Proctor density

PROJECT	Blue Haze Landfill SRA Schedule 5 Response		
Job No. 70097470	Made By: WYH	Date: 07/06/2022	
Ref. Attachment 3	Checked: BZ	Sheet: 6	
	Reviewed: DRVJ	of: 8	

Geometry:



Calculation:

From Equation (1), the pipe deflection assuming gravel surround compacted to 85% standard density is given by:

δ_v	=	**0.003**	m
	=	**2.51**	mm
	=	**1.4**	% of the nominal pipe inside diameter

The calculations indicate that under the current condition, assuming the gravel surround is compacted to 85% standard density, the leachate drainage pipe will deflect up to approximately 1.4% which is less than the maximum allowable deflection ratio of 2.7% for HDPE pipes with a SDR value of 11. This is considered satisfactory.

From Equation (1), the pipe deflection assuming gravel surround compacted to 95% standard density is given by:

δ_v	=	**0.001**	m
	=	**0.74**	mm
	=	**0.4**	% of the nominal pipe inside diameter

The calculations indicate that under the current condition, assuming the gravel surround is compacted to 95% standard density, the leachate drainage pipe will deflect up to approximately 0.4% which is less than the maximum allowable deflection ratio of 2.7% for HDPE pipes with a SDR value of 11. This is considered satisfactory.

Leachate Pipework Strength Calculations

Aim: To assess strength of the leachate drainage pipe with a diameter of 355 mm under the proposed development.

Approach: To use the Iowa formula to predict the long term deformation of the leachate drainage pipe.

References:
1 Environment Agency, R&D Technical Report P1-397/TR, Landfill Engineering: Leachate Drainage, Collection and Extraction Systems, September 2002.

2 Qian X., Koerner R.M., and Gray D.H., Geotechnical Aspects of Landfill Design and Construction. Prentice Hall, 2002.

The Modified Iowa Formulae can be used to predict the deformation of a pipeline at any stage in its life. The primary design limitation of long term deformation can be calculated using the following equation:

$$\delta_v = \frac{D_L K_X W_c}{(EI/r^3)+(0.061\ E')}$$ Equation 1.

Where:

W_c = Static Loading (simple prismatic loading is assumed)
= ((depth of existing waste·γ_{waste})+(depth of future waste·γ_w)+(restoration soil thickness·$\gamma_{restor\ soils}$))·OD of pipe
= ((28 m x 10 kN/m^3) + (5 m x 10 kN/m^3) + (1.5 m x 20 kN/m^3)) x 0.355
= **127.8** kN/m

D_L = Deflection lag factor (dimensionless)
= **1.5** (assumed)

K_x = Bedding factor
= **0.103** (value assumed is as recommended by the Water Research Centre)

r = Mean radius of pipe
= **177.5** mm

t = Wall thickness of pipe
= **32.27** mm

I = Moment of inertia of pipe wall per unit length
= **2801.1** mm^3

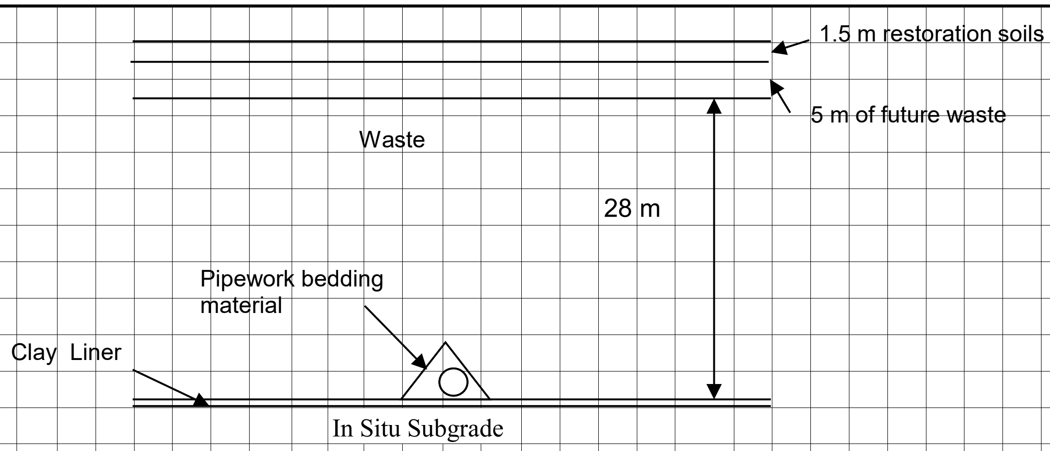
E = Modulus of elasticity of the pipe material (long term)
= **160,000** kPa

S_L = (EI/r^3) = Long-term stiffness of pipe
= **80.1** kPa
`

E' = Modulus of soil reaction under high stress conditions
= **64,000** kPa Gravel surround with little no fine compacted to 85% Standard Proctor density
= **220,000** kPa Gravel surround with little no fine compacted to 95% Standard Proctor density

PROJECT	Blue Haze Landfill SRA Schedule 5 Response		
Job No. 70097470	Made By: WYH	Date: 07/06/2022	
Ref. Attachment 3	Checked: BZ	Sheet: 8	
	Reviewed: DRVJ	of: 8	

Geometry:



Calculation:

From Equation (1), the pipe deflection assuming gravel surround compacted to 85% standard density is given by:

δ_v = **0.005** m
= **4.96** mm
= **1.4** % of the nominal pipe inside diameter

The calculations indicate that under the current condition, assuming the gravel surround is compacted to 85% standard density, the leachate drainage pipe will deflect up to approximately 1.4% which is less than the maximum allowable deflection ratio of 2.7% for HDPE pipes with a SDR value of 11. This is considered satisfactory.

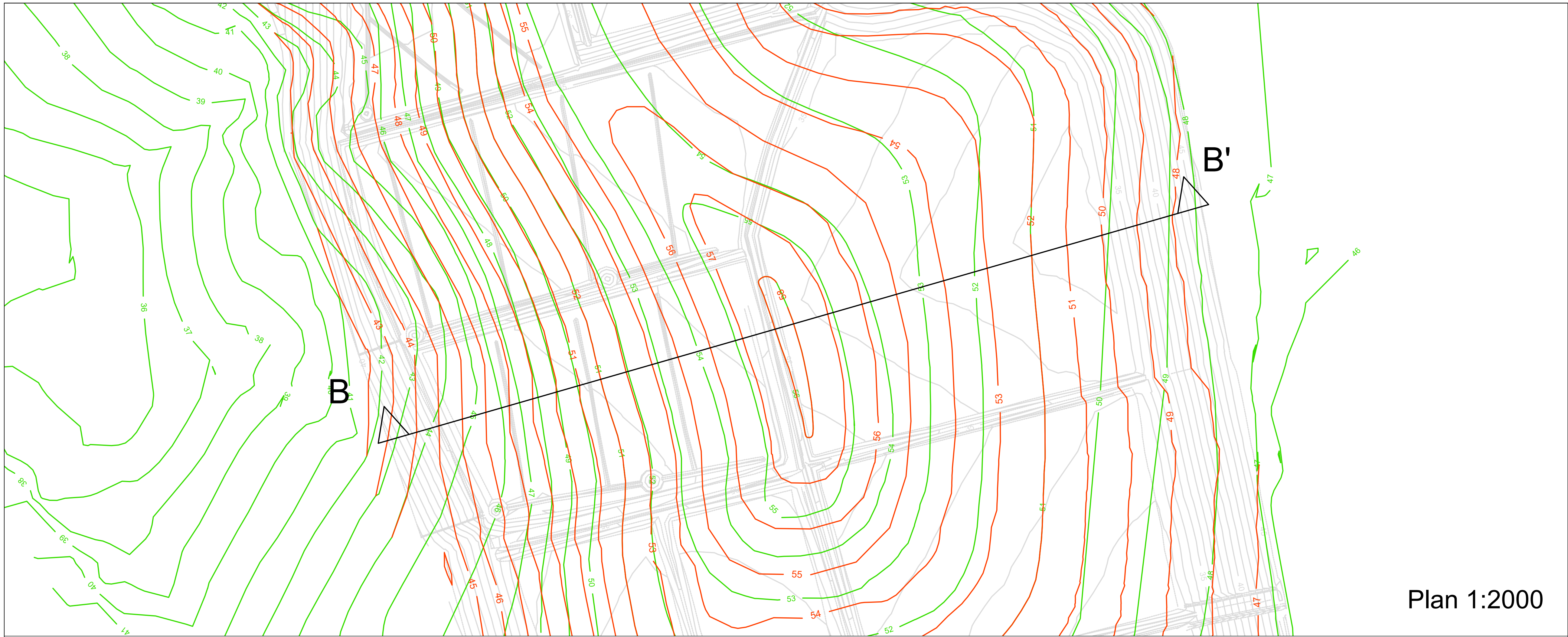
From Equation (1), the pipe deflection assuming gravel surround compacted to 95% standard density is given by:

δ_v = **0.001** m
= **1.46** mm
= **0.4** % of the nominal pipe inside diameter

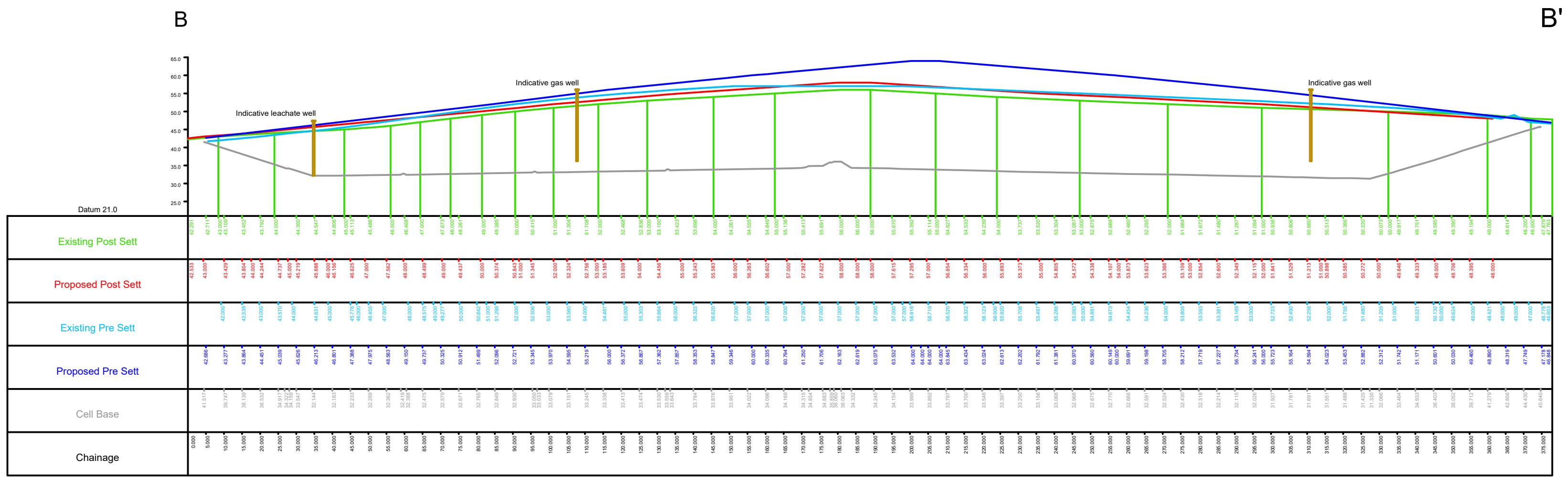
The calculations indicate that under the current condition, assuming the gravel surround is compacted to 95% standard density, the leachate drainage pipe will deflect up to approximately 0.4% which is less than the maximum allowable deflection ratio of 2.7% for HDPE pipes with a SDR value of 11. This is considered satisfactory.

ATTACHMENT 4

Cross Section B



Plan 1:2000



Section 1:2000

Rev	Description of revision	Drawn	Chkd	App	Date

VEOLIA

Norwood Industrial Estate,
Rotherham Road,
Killamarsh, Sheffield, S21 2DR

Project

BLUE HAZE

Title

Existing & Proposed
Post Settlement
Cross Section

	Initials	Date	Scale	Sheet size
Drawn	AP	27/05/22	See drawing	A3
Checked				
Approved				

Job No.

Drawing No.	Revision
BH_PSCS_0522	0