# 

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)<sup>1</sup>. 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)<sup>2</sup>. 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.

<sup>&</sup>lt;sup>1</sup> Koerner R.M. & Daniel D.E. (1997). Final Covers for Solid Waste Landfills and Abandoned Dumps, Thomas Telford, London.

<sup>&</sup>lt;sup>2</sup> Jones D.R.V & Dixon N (1998). The Stability of Geosynthetic Landfill Lining System, pp 99-117, Geotechnical Engineering of Landfills. Thomas Telford, London.

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

#### Table 2: Summary of Gas Pressure Analysis

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)<sup>3</sup>. 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.

<sup>&</sup>lt;sup>3</sup> 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.

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 3: Leachate Pipework Deflection Results – Existing Waste

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)<sup>4</sup> 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

Author: WY Htike/BZ/DRVJ/ab

Russel gres

Dr DRV Jones Commercial Director

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

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

## Vehicle Loading Analysis

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Thickness of cover soil, h

 $\frac{\text{Geosynthetic interface shear strengths:}}{\text{Cover soil/GCL friction angle, } \delta_1}$ 

Cover soil/GCL cohesion intercept,  $\alpha_1$ 

GCL/Blinding Layer friction angle,  $\delta_2$ 

GCL/Blinding Layer cohesion intercept,  $\alpha_2$ 

Height of slope, H

Slope angle,  $\beta$ 

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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 <sup>2</sup>	
acceleration due to gravity			9.81	m/s <sup>2</sup>	
Dynamic Force per unit width			0		
Effective Equipment Force nor	mal to failu	re Plane	0		
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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 <sub>A</sub>			3638.15	kN	
Weight of passive wedge, W <sub>P</sub>			124.398	kN	
Pore pressure perp. to slope, U <sub>n</sub>			0		
Pore pressure in interwedge surfac	e, U <sub>h</sub>		0		
Force normal to active wedge, N <sub>A</sub>			3588.26	kN	
a			592.233		
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2. Integrity of Geosynthetics					
(i) GCL					
Mobilised shear stress at upper	· interface		601.335	kN	
Shear strength at lower interfac	ce 📃		1652.22	kN	
Tension developed in the GCL	,		0	kN	
Tensile strength of the GCL			12	kN kN	

Factor of Safety against rupture

Infinite

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Slope angle, β

Geosynthetic interface shear strengths: Cover soil/GCL friction angle,  $\delta_1$ 

Cover soil/GCL cohesion intercept,  $\alpha_1$ 

GCL/Blinding Layer friction angle,  $\delta_2$ 

GCL/Blinding Layer cohesion intercept,  $\alpha_2$ 

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PROJECT Ref.

70097470 Attachment 1

**Blue Haze Landfill SRA Schedule 5 Response** 

Made By:	WYH	Date:	07/06/2022
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Reviewed:	DRVJ	of:	6

### **Fibre-reinforced Geosynthetic** Clay Liner (GBR-C)



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

Bentofix® NSP 4300 is a shear strength transmitting geosynthetic clay barrier (GBR-C), continuously needlepunched 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):	•	1	
Mass per unit area	EN ISO 9864	g/m²	110
Bentonite layer (sodium bentonite p	owder):		
Mass per unit area	EN 14196 (P 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 (P 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 <sub>10</sub> )	EN 16416 / ASTM D5887	m/s	2 x 10 <sup>-11</sup>
Index Flux (q <sub>10</sub> )	EN 16416 / ASTM D5887	(m³/m²)/s	4.5 x 10 <sup>-9</sup>
Roll dimensions:			
width x length, / diameter	-	m x m / m	5.0 x 50 / Ø 0.65

### Gas Pressure Analysis

Job No.       70097470       Made By: WYH       Date:       07/06/202:         Ref.       Attachment 2       Checked: BZ       Sheet:       1         INTRODUCTION       Image: Strate in the stability of the capping system has been assessed for the final steepest capping geometry.       Image: Strate in the stability of the capping system has been assessed for the final steepest capping geometry.         Image: Strate in the stability of the capping system has been assessed for the final steepest capping geometry.       Image: Strate in the stability of the capping system has been assessed for the final steepest capping geometry.         Image: Strate in the stability of the capping system has been assessed for the final steepest capping geometry.       Image: Strate in the stability of the capping system has been assessed for the final steepest capping geometry.         Image: Strate in the stability of the capping system has been assessed for the final steepest capping geometry.       Image: Strate in the stability of the capping system has been assessed for 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.         Image: Strate in the geosynthetic clay liner has been assessed by considering the shear strength developed above and below 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.         Image: Strate in the geosynthetic clay liner has been aspring layer comprises a a Geosynthetic Clay Liner (GCL) with 1.5 m	2
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reported by Jones & Dixon (1998). Based on this and our experience of geosynthetic interfaces, a conservative assessment of the interface shear strength parameters is:	of
• Cover soils / GCL $\alpha_p' = 0$ kPa $\delta_p' = 24$ Deg.	
• GCL / Blinding layer $\alpha_p' = 0$ kPa $\delta_p' = 24$ 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.	

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	<u></u>			l		-	
1. Stability of Cover Soils							
Calculated Parameters							
Length of slope, L			139.3537	m			
Thickness of water, h <sub>w</sub>			0	m			
Weight of active wedge, W <sub>A</sub>			3638.153	kN			
Weight of passive wedge, W <sub>P</sub>			124.3979	kN			
Pore pressure perp. to slope, U <sub>n</sub>			0	kN			
Pore pressure in interwedge surface, U <sub>h</sub>			0	kN			
Force normal to active wedge, N <sub>A</sub>			3588.258	kN			
Force normal to active wedge from gas	pressure, N <sub>G</sub>		696.7687	kN			
Vert pp on passive wedge, U <sub>V</sub>			0	kN			
a			592.2334				
b			-1360.02				
c			85.84668				
		Fac	tor of Safe	tv against o	cover soils sliding		2.23
2. Integrity of Geosynthetics							
Mobilised shear stress at upper inte	rface		740 4195	kN			
			, 10.11)5				
Shear strength at lower interface			1652 221	kN			
			1052.221				
Tension developed in the geosytheti	C		0	kN			
Tension developed in the geosylicit				KI V			
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										(0	) Г 1	mu	e sic	ope						1										
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Input Par	rame	ters	ight	(dry)	N 07									18		kΝ/	m <sup>3</sup>													
Cover soil	ls uni	t we	ight	(satu	, id Irate	ry ed)γ	- 4							20		kN/	m <sup>3</sup>													
Cover soil	ls inte	ernal	she	ar str	eng	νth. φ	ai							20		Des	<b>y</b> .													
Cover soil	ls coh	esio	n, c		5.12	,, φ								0		kPa														
Thickness	of co	over	soil	s, h										1.5		m														
Height of	slope	, Н												23		m														
Slope ang	le, β													9.5		Deg	3.													
Geosynthe	etic ir	nterfa	ace	shear	str	ength	s:																							
Cover	Soils	s/GC	CL fr	riction	n ar	ngle, å	$5_1$							24		Deg	g.													
Cover	Soils	s/GC	CL co	ohesi	on i	interc	ept, α	1						0		kPa D														
GCL	Blind	ing l	layer	r frici	tion	angl	$\delta_2$							24		Deg	3.													
GCL/	Biind	ing c	Jone	sion	inte	ercepi	$, \alpha_2$							0		кра														
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Geosynthe	etic te	ensile	e str	enoti	is:		-							.23																
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Gas press	ure												· · ·	5		kPa														
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	PROJECT	Blue I	Haze Lan	dfill SR	A Schedule	5 Respoi	ıse		
	Job No.	70097470		Made By	: WYH	-	Date:	07/06	5/2022
<b>GOLDER</b>	Ref.	Attachment	2	Checked	: BZ		Sheet:	5	
•				Reviewee	d: DRVJ		of:	8	
	<u></u>						1		
1. Stability of Cover Soils									
Calculated Parameters									
Length of slope, L			139.3537	m					
Thickness of water, h <sub>w</sub>			0.375	m					
Weight of active wedge, W <sub>A</sub>			3741.804	kN					
Weight of passive wedge, W <sub>P</sub>			125.2618	kN					
Pore pressure perp. to slope, U <sub>n</sub>			511.1495	kN					
Pore pressure in interwedge surface, $U_h$			0.703125	kN					
Force normal to active wedge, N <sub>A</sub>			3179.454	kN					
Force normal to active wedge from gas	pressure, N <sub>G</sub>		696.7687	kN					
Vert pp on passive wedge, $U_V$			4.201709	kN					
a			609.1253						
b			-1180.251						
c			73.70953						
		Fac	tor of Safe	ety against	t cover soils sli	ding		1	.87
2. Integrity of Geosynthetics									
(i) GCL									
Mobilised shear stress at upper inte	rface		906.6247	kN					
Shear strength at lower interface			1698.116	kN					
Tension developed in the geosythet	ic		0	kN					
Tensile strength of the geosythetic			12	kN					
		Fac	tor of Safe	ety against	t rupture			Inf	inite
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	C		τ.				Job	No		70097	470				Made	By:	: 1	WY	Н				•	Dat	te:	0	7/06	/2022	2
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															Revie	wed	ł: I	DRV	/J					of:			8		
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Section	В				PS	R	=	0	.5																				
Aim: To a	ssess	the e	ffect	of ga	s pro	essur	e or	ı sta	bilit	y of co	ver s	soils	s and	inte	grity c	f ge	osy	nth	etic	s.			1						
Approach	i: Use	e the a	appro	bach p	ropo	osed	by J	one	s &	Dixon	(199	8).		1	<u> </u>							1	1						
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Input Par	amet	ters														-													
Cover soil	s unit	t weig	,ht (c	lry), γ	dry								18		kN/m	3													
Cover soil	s unit	t weig	ht (s	aturat	ted),	$\gamma_{sat}$							20		kN/m	3													
Cover soil	s inte	ernal s	hear	stren	gth,	¢							22		Deg.														
Cover soil	s coh	esion	, c										0		kPa	_													
Thickness	ofco	over so	oils,	h									1.5		m														
Height of	slope	, Н											23		m	_													
Slope ang	le, $\beta$										_		9.5		Deg.	_	_							_			_		
Geosynthe	etic in	terfac	e sh	ear sti	reng	ths:							24		Dec														
Cover	Solls		, Iric	tion a	ngle	e, o <sub>1</sub>							24		Deg.														
	Solis	ing la	, con	riction	inte	rcep	$\alpha_1$						24		кра Dog	-	-												
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Gas pressi	ıre												5		kPa		+												
Gas pressi	ıre												5		kPa														

	PROJECT	Blue	Haze Lan	dfill SRA	Schedule 5 Respo	nse	
	Job No.	70097470		Made By:	WYH	Date: (	07/06/2022
<b>GOLDER</b>	Ref.	Attachmen	t 2	Checked:	BZ	Sheet:	7
•				Reviewed:	DRVJ	of:	8
1. Stability of Cover Soils							
Calculated Parameters							
Length of slope, L			139.3537	m			
Thickness of water, h <sub>w</sub>			0.75	m			
Weight of active wedge, W <sub>A</sub>			3843.728	kN			
Weight of passive wedge, W <sub>P</sub>			127.8534	kN			
Pore pressure perp. to slope, U <sub>n</sub>			1013.779	kN			
Pore pressure in interwedge surface, U	h		2.8125	kN			
Force normal to active wedge, N <sub>A</sub>			2777.699	kN			
Force normal to active wedge from gas	pressure, N <sub>C</sub>	ĩ	696.7687	kN			
Vert pp on passive wedge, U <sub>V</sub>			16.80684	kN			
a			625.7743				
b			-1000.768				
c			61.78164				
		Fac	ctor of Safe	ty against c	cover soils sliding		1.53
2. Integrity of Geosynthetics							
(i) (GCL							
Mobilised shear stress at upper inte	erface		1136.218	kN			
Shear strength at lower interface			1744.011	kN			
				1.5.7			
Tension developed in the geosythet	tic		0	kN			
			10	1.5.7			
I ensile strength of the geosythetic			12	kN			
		Б					T C 1
		Fac	ctor of Safe	ty against i	rupture		Infinite
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PROJECT Ref.

70097470 Attachment 2

**Blue Haze Landfill SRA Schedule 5 Response** 07/06/2022 Made By: WYH Date: Sheet: Checked: BZ Reviewed: DRVJ of:

### Fibre-reinforced Geosynthetic Clay Liner (GBR-C)

Bentofix® NSP 4300



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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 needlepunched 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 p	owder):	•	•
Mass per unit area	EN 14196 (P 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 (P 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 <sub>10</sub> )	EN 16416 / ASTM D5887	m/s	2 x 10 <sup>-11</sup>
Index Flux (q <sub>10</sub> )	EN 16416 / ASTM D5887	(m³/m²)/s	4.5 x 10 <sup>-9</sup>
Roll dimensions:			
width x length, / diameter	-	m x m / m	5.0 x 50 / Ø 0.65

# **Pipework Deflection Analysis**

	PROJEC	T Blue Haze La	andfill SRA Schedule 5 Res	ponse	
Colder	Job No.	70097470	Made By: WYH	Date:	07/06/2022
	Ref.	Attachment 3	Checked: BZ	Sheet:	1
Associates			Reviewed: DRVJ	of:	8

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	=	160,000	kPa																							
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	PROJECT Blue Haze La	ndfill SRA Schedule 5 Respo	nse
Colder	Job No. 70097470	Made By: WYH	Date: 07/06/2022
<b>T</b>	Ref. Attachment 3	Checked: BZ	Sheet: 5
Associates	1	Reviewed: DRVJ	of: 8
Leachate Pipework Strength Calcu	lations		
Aim: To assess strength of the leacha	ate drainage pipe with a diameter	of 180 mm under the proposed devel	lopment.
Approach: To use the Iowa formula	to predict the long term deformat	ion of the leachate drainage pipe.	
<b>References:</b> 1 Environment Ager	ncy, R&D Technical Report P1-39	97/TR, Landfill Engineering: Leacha	ite Drainage,
Collection and Ext	traction Systems, September 2002		
2 Oian X Koerner I	R M and Gray D H. Geotechnic	al Aspects of Landfill Design and C	onstruction

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:

Prentice Hall, 2002.

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Wh	ere:																															-
		W <sub>c</sub>	=	Stati	c Load	ling	(sir	npl	e pri	sma	ntic 1	oad	ing	is a	ssur	ned	)															-
			=	((de	pth of	exis	ting	, wa	aste.	Ywast	e)+(	dep	th o	of fut	ture	was	ste∙γ	/w)+	(res	tora	tion	soil	thick	nes	s·γ <sub>r</sub>	estor s	soils)	)·OI	) of	pip	e	
			=	(( ]	28 m	х	10	kN	$V/m^3$	)	+	(		5	m x	1	0	kN/	$/m^3$	)+	(	1.5	m	х	20	kN/	m <sup>3</sup>	)) x		0.18	8	-
			=	64.	8 kN	/m																										
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		$D_L$	=	Defl	ection	lag	fact	tor	(dim	ensi	ionle	ess)																				-
			=	1.5	5 (as	sum	ed)																									-
		K <sub>x</sub>	=	Bed	ding fa	ctor	•																									-
			=	0.	103	(va	lue	ass	ume	d is	as re	ecoi	nm	ende	ed b	y th	e W	ater	Re	sear	ch (	Centre	e)									
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		r	=	Mea	n radi	us of	f pir	be										t	=	Wa	ll th	nickne	ess o	f pij	pe							-
			=		90	mm	ı												=	16	.36	mm										
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the	lead	chat	e dr	aina	ige ]	pipe	W1	ll de	tlec	t up	to a	appr	OX11	nat	ely	1.4%	6 Wł	nch	1s les	s tha	n the	e ma	xımu	m a	llov	vabl	e de	tlec	tion	rati	o of	
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Leachate Pipework Strength Calcu	lations																		
Aim: To assess strength of the leacha	te draina	age pi	pe w	ith a c	liame	eter	of 35	5 m	m u	Inde	r the	prop	oose	d dev	eloj	oment.			

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

Re	fere	ence	s:	1	Env	viroi	nmei	nt A	lgen	icy,	R&	DT	ech	nica	ıl R	epor	t Pl	-39	7/T]	R, L	and	lfill	Engi	neer	ing:	Lea	icha	te D	rair	nage	;,	
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Equation 1.

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:

	$D_L K_X W_c$	_
$\delta_v =$	(EI/r <sup>3</sup> )+(0.061 E')	
		1

Where	:																															
	Wc	=	Sta	tic I	Load	ling	(sir	nple	e pri	isma	atic l	oad	ling	is a	issui	ned	)															
		=	((de	epth	of	exis	ting	wa	ste∵	γ <sub>wast</sub>	te)+(	dep	th c	of fu	ture	wa	ste∙γ	/w)+	(res	tora	tion	n soil 1	thic	knes	$ss \cdot \gamma_r$	estor	soils)	)·O	Do	f pip	e	
		=	((	28	m	х	10	kN	$/m^3$	)	+	(		5	m x	1	0	kN/	$/m^3$	)+	(	1.5	m	х	20	kN	$/m^3$	)) 🤉	ĸ	0.3	55	
		=	12	7.8	kN/	/m																										
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	$D_L$	=	Det	flect	tion	lag	fact	tor (	dim	ens	ionle	ess)																				
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### **Cross Section B**





