1. Responses to EGLE comments dated Feb. 24, 2022

Technical Comments for
WDI 2021 Permit Application Attachment B6 Engineering Plans

Comments by EGLE on February 24, 2022

Responses by CTI on April 20, 2022

Attachment A-1 Slope Stability Analysis

1. The cross section name of C-C' or G-G' should be marked on the cross section locations shown on the plans of Page 2 of 4 in Attachment A-1.1, Page 2 of 4 in Attachment A-1.2, Page 2 of 4 in Attachment A-1.3, Page 2 of 4 in Attachment A-1.5, and Page 2 of 4 in Attachment A-1.7.

   Response:

   Revised per comment

   In the original submittal, the attachment IDs are very confusing. We streamlined the attachment IDs in this submittal. For you convenient for comparing the original IDs with new IDs, we prepared the attached spreadsheet.

2. By comparing Tables 1, 2, and 3 (i.e., Table 1 Material Properties in Attachment A-1 Slope Stability Analysis, Slope Stability Analysis Report Form in Attachment A-1.1 C-C' Foundation Stability, and Table 1 Soil Properties for Settlement Analysis in Attachment A-2.2 Sample Calculations), it can be found that some soils and waste were used different unit weight values in the slope stability analysis and settlement calculations, such as the unit weight of the final cover soil was 130 pcf in Tables 1 and 2 for the slope stability analysis and 135 pcf for the settlement calculations, the unit weight of the existing waste was 86 pcf in Table 1 and 91 pcf in Table 2 for the slope stability analysis and 82 pcf in Table 3 for the settlement calculations, and the unit weight of the lower clay was 133 pcf in Tables 1 and 2 for the slope stability analysis and 128 pcf in Table 3 for the settlement calculations. In addition, the unit weight of the upper clay was missed in Table 3 for the settlement calculations. These discrepancies also exist in Attachment A-1.2 to Attachment A-1.7. CTI must clarify these discrepancies and used the corrected values to conduct analysis and calculations.

Table 1 – Soil and waste properties for slope stability analysis (i.e., Table 1 Material properties on Page 2 of 3 in Attachment A-1 Slope Stability Analysis)

Material	Name	Color in Profile	Unit Wt(s) (pcf)	Strength ϕ or δ (deg.)	Strength C or Ca (psf)
1	Final Cover	Orange	130	0	1500
2	Existing Waste	Teal	86	34	0
3	New Waste	Light Green	103[A]	26[B]	300[B]
4	Upper Clay	Brown	131	0	1325
5	Middle Clay	Yellow	136	0	3000
6	Lower Clay	Maroon	133	$0.22\sigma'_v$	
7	Silt	Blue	125	28	0
8	Sand	Red	115	32	0
9	Liner (floor)	Magenta	120	13.2	0
10	Liner (sideslope)	Magenta	120	9	0

Notes:
[A] unit weight of waste determined from site survey data reported in 2020.
[B] representative value of waste strength as reported by Qian et al. (2002)
All other properties obtained from NTH (2012)

Table 2 – Soil and waste properties for slope stability analysis (i.e., Slope Stability Analysis Report Form on Page 1 of 4 in Attachment A-1.1 C-C' Foundation Stability)

Material	Name	Color in Profile	Unit Wt(s) (pcf)	Strength ϕ or δ (deg.)	Strength C or Ca (psf)
1	Final Cover	Orange	130	0	1500
2	Existing Waste	Teal	91	34	0
3	New Waste	Light Green	103	26	300
4	Upper Clay	Brown	131	0	1325
5	Middle Clay	Yellow	136	0	3000
6	Lower Clay	Maroon	133	$0.22\sigma'_v$	
7	Silt	Blue	125	28	0
8	Sand	Red	115	32	0
9	Liner (floor)	Magenta	120	13.2	0
10	Liner (sideslope)	Magenta	120	9	0

Table 3 – Soil and waste properties for settlement calculations (i.e., Table 1 Soil properties for settlement analysis on Page 2 of 11 in Attachment A-2.2 Sample Calculations)

Soil Type	Thickness [ft]	Moist Unit Weight [pcf]
Final cover soil	4	135
New waste	Varies	103*
Existing cover soil	Varies	135
Existing waste	Varies	82
In-situ middle clay	Varies	136
In-situ lower clay (moist)	Varies	128
In-situ lower clay (saturated)	Varies	128
In-situ silt (saturated)	18	125
In-situ sand (saturated)	45	115

* *New waste unit weight obtained from email correspondence with WDI dated 11/18/2020*

Response:

Properties of soil and waste were revised to be consistent per comment.

3. In the same Attachment A-1 Slope Stability Analysis document, there are two sets of sub-attachments used the same names, such as Attachment A-1.2 to Attachment A-1.7 for the foundation stability analysis and liner stability analysis under final condition with zero adhesion of the Cross-Sections C-C' and G-G', and Attachment A-1.2 to Attachment A-1.12 in the Slope Stability Analysis for WDI Final Cover Grading Modification that is the second part of Attachment A-1. It is suggested that CTI should revise the names of Attachment A-1.2 to Attachment A-1.12 to avoid confusion. Or CTI can divide Attachment A-1 into two sub-attachment documents. They are one for the foundation and liner stability analysis for new Subcells G4 to G7 and another for the slope stability analysis for WDI final cover grading modification.

   Response:

   Revised per comment

4. Stability of slopes can be analyzed using either effective stress or total stress methods. In effective stress analyses the interface and internal shear strengths of the liner materials and soils are related to the effective normal stress on the potential slip surface by means of effective stress shear strength parameters. Pore pressures within the liner and soils must be known and are part of the information required for analysis. In total stress analyses the interface and internal shear strengths of the liner materials and soils are related to the total normal stress on the potential slip surface by means of total stress shear strength parameters. Pore pressures within the liner and soil mass need not be known and are not required as input for analyses. In Attachments A-1.1, A-1.2, and A-1.6, both the "Effective Stress" and "Total Stress" are marked on Page 1 of 4 (Table 4). Does this mean that both the effective stress and total stress methods were used to conduct the slope stability analysis? If so, why were only the effective shear strengths without the total shear strengths shown on the calculation result sheets of Pages 3 of 4 and 4 of 4?

Table 4 – Slope Stability Analysis Report Form in Attachment A-1.1 C-C' Foundation Stability

<table>
<tr><td>Purpose of Analysis:</td><td colspan="3">To determine the factor of safety of the proposed final waste grades using cross-section C-C'. This case considers a west-facing slope, with fill to the final permitted grade elevations.</td></tr>
<tr><td>☒ Effective Stress
☒ Total Stress</td><td>☒ Static ☐ Seismic</td><td>☒ Pore Pressure</td><td>☒ Optimized Surface</td></tr>
<tr><td>Additional Details:</td><td colspan="3">The friction angle of the liner system was set equal to the required minimum interface friction angle determined from the liner stability analysis performed on Cross Section C. The groundwater level was set at elevation 655ft based on historical borings as documented in the Basis of Design Report (NTH 2012).
Drained strength parameters were used for material with a relatively high permeability where excess pore-pressure conditions are not expected due to loading.
Undrained strength parameters were used for low-permeability materials (clays) since excess pore pressure conditions may occur after loading. This is considered a conservative approach for this scenario.</td></tr>
</table>

Table 5 – Slope Stability Analysis Report Form in Attachment A-1.5 C-C' Liner Stability under Final Condition with non-zero adhesion (previously tested values)

Purpose of Analysis:	To determine the minimum required liner interface friction angle to achieve an acceptable factor of safety of the proposed final waste grades using cross-section C-C'. This case considers a west-facing slope, with fill to the final permitted grade elevations under undrained conditions.		
☐ Effective Stress ☒ Total Stress	☒ Static ☐ Seismic	☒ Pore Pressure	☒ Optimized Surface
Additional Details:	The liner system was modeled in 2 sections (floor and sideslope) to allow use of Peak and Large-Displacement strength parameters appropriately. The liner interface strength properties are based on interface strength test results of a similar liner system installed on site. The required factor of safety of 1.5.		

Response:

For the analysis of the foundation global stability conditions, we have intentionally selected drained strength parameters for the sand layers and undrained strength parameters for the clay layers. The reason is that during a landslide event, shear-induced pore pressure increases within clays are not modeled by the slope stability analysis software and therefore the software will overestimate the available shear strength from these layers when using drained strength parameters. Accordingly, CTI has opted to model the stiff clay layers using constant values of undrained shear strength (thereby ignoring possible strength gain from overburden) and to model the soft clay layers using an undrained shear strength-to-vertical effective stress factor (s_u/σ'_v). The use of a strength factor is advocated by Ladd (1991) as a way to take credit for strength gain over time from overburden while also not overestimating resistance to shear during a landslide event. By using vertical effective stress to compute undrained shear strength, we are accounting for the effect of the piezometric surface on the shear strength of the soft clay layer. However, it is noted that the undrained shear strength values obtained from the use of the strength ratio are less than the shear strength values that are found using the drained strength parameters.

This explanation has been added to the calculation sheet per comment.

5. In Attachments A-1.5 and A-1.7, the "Total Stress" was marked on Page 1 of 4 (Table 5). Does this mean that a total stress method was used to conduct stability analysis? If so, why were the effective shear strength parameters of cohesion and friction angle without the total shear strengths shown on the calculation result sheets of Pages 3 of 4 and 4 of 4?

Response:

See comment #4 – additional explanation regarding the material properties used in the analyses has been added to Attachment A-1

6. In Attachments A-1.2, A-1.3, A-1.4, and A-1.5 of the second section of "WDI Final Cover Grading Modification" (Table 6), both the "Effective Stress" and "Total Stress" were marked on Page 1 of 4. If so, why were only the effective shear strengths without the total shear strengths shown on the calculation result sheets of Pages 3 of 4 and 4 of 4?

Table 6 – Slope Stability Analysis Report Form in Attachment A-1.2 G-G' Foundation Stability of the Section of WDI Final Cover Grading Modification

Purpose of Analysis:	To determine the factor of safety of the proposed final waste grades using cross-section G-G'. This case considers a north-facing slope, with fill to the final proposed grade elevations.		
☒ Effective Stress ☒ Total Stress	☒ Static ☐ Seismic	☒ Pore Pressure	☒ Optimized Surface
Additional Details:	The strength parameters used to model the interface strength of the liner system were those determined from the slope stability analysis of MC-VI-G, Sub Cell G4. The groundwater level was set at elevation 655 based on historical borings as documented in the Basis of Design report (NTH 2012). Drained strength parameters were used for material with a relatively high permeability where excess pore-pressure conditions are not expected due to loading. Undrained strength parameters were used for low-permeability materials (clays) since excess pore pressure may accumulate after loading. This is considered a conservative approach for this scenario.		

7. Attachments A-1.12 of the second section of "WDI Final Cover Grading Modification" (Table 7), the "Total Stress" was marked on Page 1 of 4. Does this mean that a total stress method was used to conduct stability analysis? If so, why were the effective shear strength parameters of cohesion and friction angle without the total shear strengths shown on the calculation result sheets of Pages 3 of 4 and 4 of 4?

Table 7 – Slope Stability Analysis Report Form in Attachment A-1.2 J-J' Liner Stability under Interim Conditions (example interim stability calculation)

Purpose of Analysis:	To determine the required interface friction angle of the liner system to achieve an acceptable interim factor of safety of 1.3 using cross-section J-J'. This case considers a west-facing slope, with fill to the final permitted grade elevations at an interim slope of 3H:1V. The failure surface is defined such that failure occurs in the underlying liner in order to evaluate the stability of the liner system.		
☐ Effective Stress ☒ Total Stress	☒ Static ☐ Seismic	☒ Pore Pressure	☒ Optimized Surface
Additional Details:	The required factor of safety is 1.3 for temporary conditions.		

Response:

See comment #4 – additional explanation regarding the material properties used in the analyses has been added to Attachment A-1

8. CTI conducted a slope stability analysis for Cross-Section C-C' liner stability under

interim condition to obtain a factor of safety of 1.3 against the translational failure mode. The result is shown in Attachment A-1.7. CTI must explain why CTI did not conduct a slope stability analysis for Cross-Section G-G' liner stability under interim condition?

Response:

Cross-section C-C' was selected for this analysis since it represents the worst case for interim stability as follows: 1) steepest floor slope, sloping outward from the waste slope and 2) narrowest floor width. Based on the results of the analyses, interim slopes that are constructed no steeper than 3H:1V and with a bottom width no less than 370 feet shall be considered acceptable for waste filling operations. If tested values of the cell floor liner system interface strength vary from those presented, or the filling geometry deviates from this case, the interim slope shall be reevaluated according to this example.

Attachment A-2 Settlement Calculations

9. The moist unit weight of the new waste listed in Table 1 – Soil Properties for Settlement Analysis on Page 2 of 7 in Attachment A-2 Settlement Calculations is 103 pcf. A note shown at the bottom of this table is "*New waste unit weight obtained from email correspondence with WDI dated 11/18/2020*." Please provide this email and field testing data to indicate how WDI determine the unit weight of the new waste placed in the landfill of 103 pcf.

   Response:

   The data table developed from site survey data has been included with the revised calculation.

10. Some old existing landfill cells of WDI belong to the type of trench fill landfill as shown in Figure 1. The waste was filled in a series of deep and narrow trenches for this type of landfill. Did the existing landfill cells located beneath Subcells G4, G5, G6, and G7 also belong to the type of trench fill landfill? If so, were the distributions (including locations and directions) of the trenches and berms in the existing landfill cells beneath the Subcells G4, G5, G6, and G7 investigated? In the calculations of the foundation settlement of the expanded landfill cells over the existing landfill, has CTI considered non-uniformity of the compressibility of the waste and berm clay in the existing landfill underneath the new cells? A series of deep and narrow trenches and berms consisting of undisturbed natural clay may cause the differential settlement of the liner systems of the expanded landfill over the existing landfill.

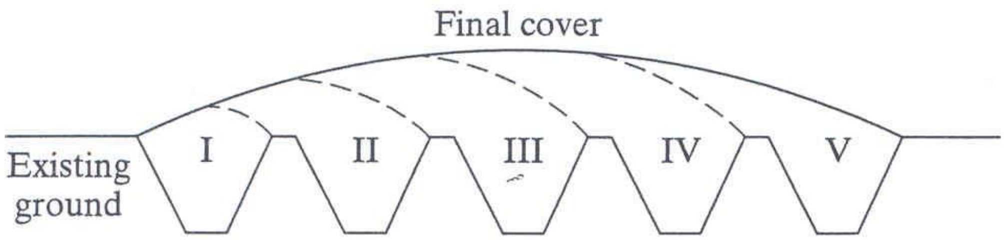


Figure 1 – Trench fill landfill

Response:

Existing information about the existing landfill below Subcells G4, G5, G6, and G7 indicate that it was a trench fill type of landfill. The precise configuration of the trenches is unknown. It is believed that each trench is bounded by intracell dikes. Using available subsurface information (MASW surveys and Geoprobe boring logs – see in the revised calculation sheet), CTI has considered the profile of known existing dikes in these settlement analyses by placing additional settlement calculation points at the estimated crests and toes of these dikes. Differential settlement estimates are accordingly greater at the transitions between these points than between points without a difference in subgrade. As a result, CTI proposes enhanced floor grades with steeper slopes in portions of the cell floor to counteract the effect of differential settlement on post-settlement slopes. CTI also proposes that the location of these enhanced floor grade zones be adjusted if MC VI- G4, G5, G6, and G7 construction activities reveal additional information about the location of the corresponding dikes.

Adjustment of the enhanced floor grade zones should proceed as follows:

a. *During excavation of the cell floor subgrade, the materials encountered at the bottom of excavation shall be logged.*
b. *The transition from waste to soil indicating the location of an intercell dike shall be surveyed across the width of the intersecting subgrade excavation bottom.*
c. *The waste-to-soil transition shall be potted in plan view and in cross section view and compared to the estimated dikes location figures in the calculation sheet.*
d. *If the waste-to-soil transition differs in horizontal location from that presented in the calculations, the corresponding zone of enhanced floor grade (applicable to G5 pipe flowline, G6 pipe flowline, G4 cross slope, and G5 cross slope – see conclusions) shall be moved so that its boundaries align with the projected toe of sideslope for the corresponding dike.*

This information has been added to the calculation sheet.

11. In the second paragraph of Design Criteria and Assumption on Page 1 of 7 of Leachate Collection System Settlement Analysis of Attachment A-2 Settlement Calculations, what does the sentence of "*In addition, points were selected at the top and bottom of existing below-grade subcell berms.*" mean? What and where are the "**top**" and "**bottom**" of existing "**below-grade subcell berms**"?

Response:

"Top" and "bottom refer to the crest and toe of dikes separating the existing waste trenches. The locations of these features have been estimated from available subgrade data as discussed in the response to comment #10.

12. By comparing Tables 8 and 9 (i.e., Table 1 Material properties in Attachment A-1 Slope Stability Analysis and Table 2 Compressibility parameters of waste and soils in Attachment A-2 Settlement Calculations), it can be found that there is a upper clay layer existing in the slope stability analysis (see Table 8). However, this soil layer was missed in the settlement calculations (see Table 9). What is the thickness of the upper clay layer used in the slope stability analysis? CTI must clarify these discrepancies and used the corrected values to conduct settlement calculations.

Table 8 – Soil and waste properties for slope stability analysis (i.e., Table 1 Material properties on Page 2 of 3 in Attachment A-1 Slope Stability Analysis)

Material	Name	Color in Profile	Unit Wt(s) (pcf)	Strength ϕ or δ (deg.)	Strength C or Ca (psf)
1	Final Cover	Orange	130	0	1500
2	Existing Waste	Teal	86	34	0
3	New Waste	Light Green	103[A]	26[B]	300[B]
4	Upper Clay	Brown	131	0	1325
5	Middle Clay	Yellow	136	0	3000
6	Lower Clay	Maroon	133	$0.22\sigma'_v$	
7	Silt	Blue	125	28	0
8	Sand	Red	115	32	0
9	Liner (floor)	Magenta	120	13.2	0
10	Liner (sideslope)	Magenta	120	9	0

Notes:
[A] unit weight of waste determined from site survey data reported in 2020.
[B] representative value of waste strength as reported by Qian et al. (2002)
All other properties obtained from NTH (2012)

Table 9 – Compressibility parameters of waste and soils (i.e., Table 2 Compressibility parameters of waste and soils on Page 2 of 7 in Attachment A-2 Settlement Calculations)

Soil Type	Primary Compression Ratio $C_c/(1+e_0)$	Secondary Compression Ratio $C_\alpha/(1+e_0)$	Recompression Ratio $C_r/(1+e_0)$
Existing cover	0.102[B]	0.005[B]	0.017[A]
Existing waste	0.147	0	0.0245[A]
In-situ middle clay	0.102	0.005	0.017[A]
In-situ lower clay	0.171	0.009	0.0285[A]
In-situ silt	0.15[B]	0[B]	0[B]
In-situ sand	0.1[B]	0[B]	0[B]

[A] Estimated from $C_r = C_c/6$.
[B] Assumed values.

Response:

The upper clay is encountered outside the limits of waste or at the toe of the waste slopes. Accordingly, the upper clay has a significant effect on stability of the waste slopes because it forms part of the buttressing soil at the slope toe. However, this location is not significant for the settlement analyses since it occurs at the flowline low points and cell floor, which is lower (i.e. locations at which a more compressible subgrade would increase the slope and help the drainage). Accordingly, it is both accurate and conservative to ignore the upper clay in the settlement analyses.

13. It is difficult to see the exact locations of the settlement points shown in the plan of Attachment A-2.1 Settlement Analysis points (also see Figure 2). The settlement points located at the same settlement lines should be connected by drawing settlement lines.

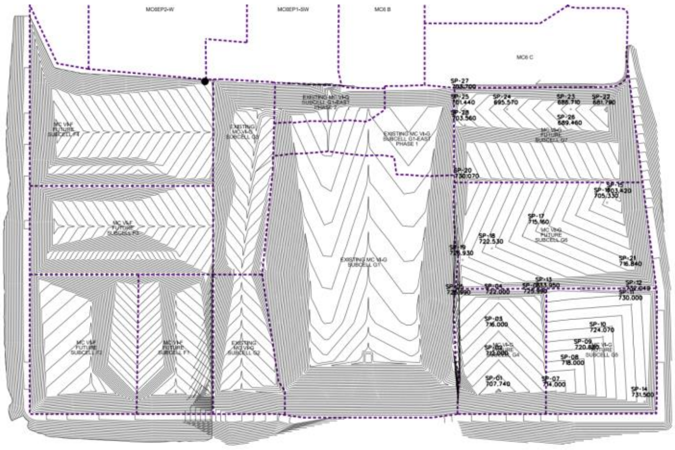


Figure 2 – Settlement analysis points

Response:

Revised per comment.

14. CTI should provide the load distribution diagram and the condition of the subsoil layers for each settlement line such as your Company did in "Application for Permit to Construct A Solid Waste Disposal Area, Pine Tree Acres, Inc. – East Development Area, Lenox Township, MI, January 2014" as shown in Figures 3 and 4.

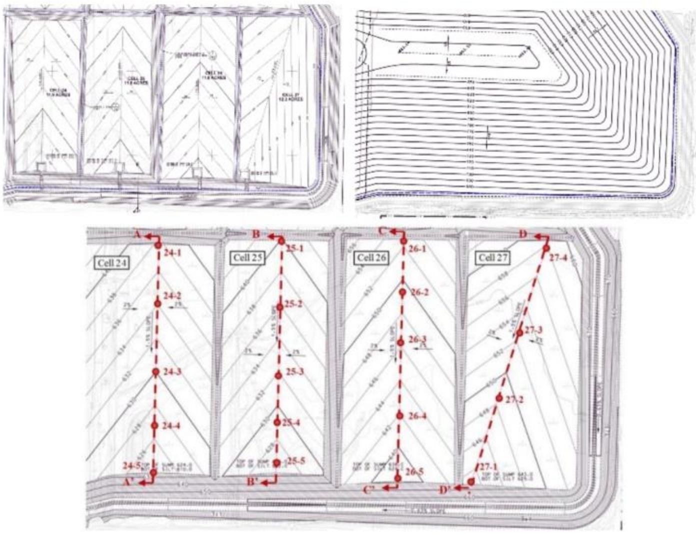
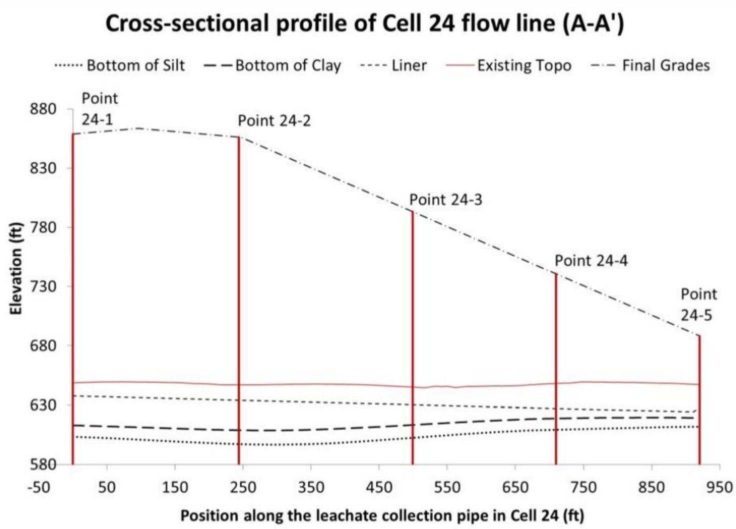
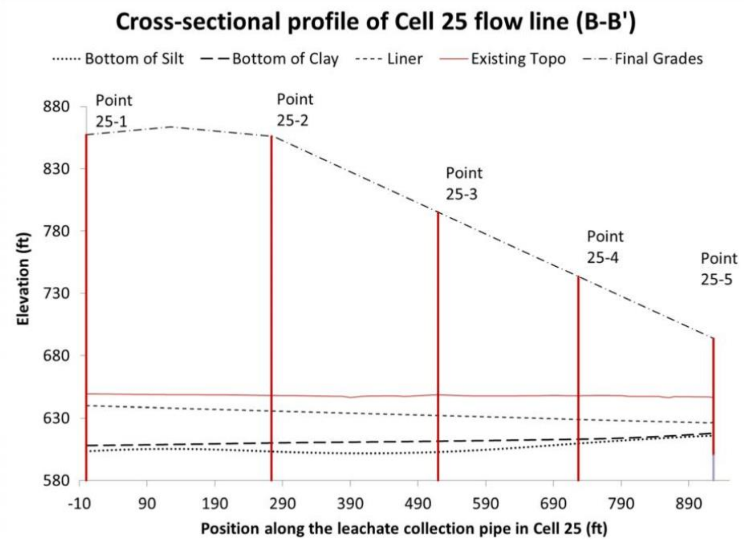


Figure 3 – Example of determination of settlement lines and calculations of overburden pressures at each settlement point





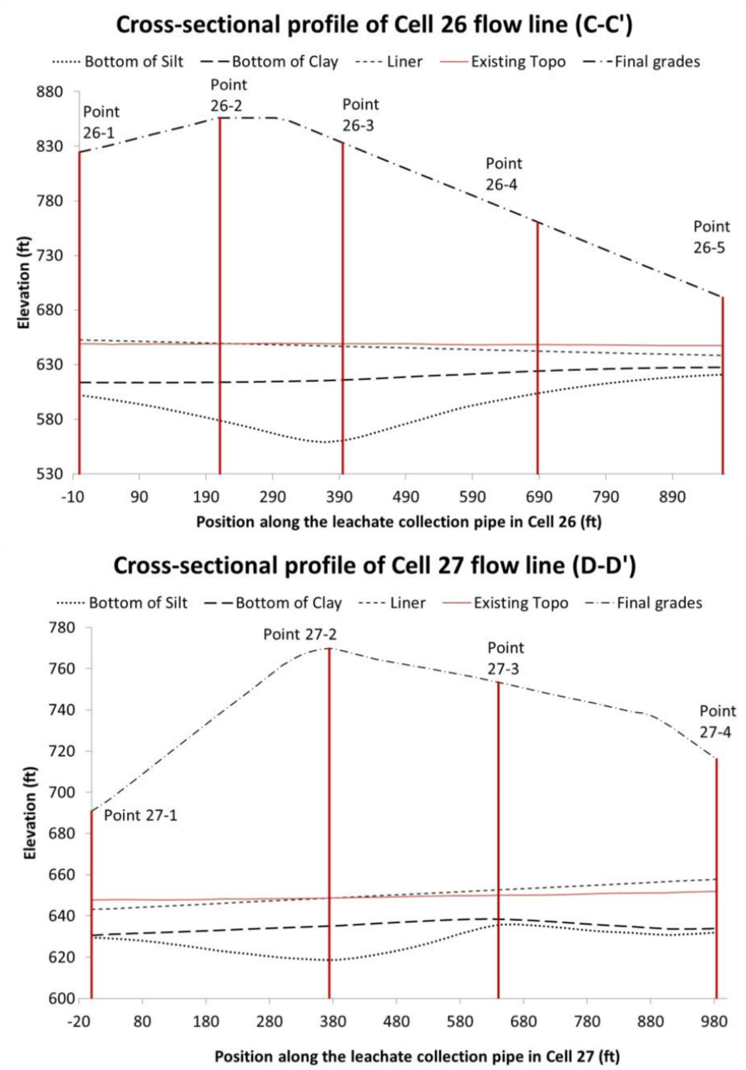


Figure 4 – Example of cross-section profile of foundation soil layers and overburden pressure along the Settlement Lines 24, 25, 26, and 27

Response:

Revised per comment

15. What type of leachate collection system in the existing landfill and what is the slope of the base floor of the existing landfill cell, MC I (Figure 5) under the new vertical expanded landfill? Is there any leachate collection pipes installed in the existing landfill? CTI should evaluate or explain whether the total or differential settlement caused by the new overburden pressure from the vertical expanded landfill will affect the gravity drain of the leachate collection system in the existing landfill (MC I).

Figure 5 – WDI landfill existing and vertical expanded cell layout map

Response:

There is no existing lateral leachate collection system in MC I. Therefore, CTI has not analyzed the effects of settlement on an existing leachate collection system.

16. There are two sub-attachment documents of Attachment A-2.1 Settlement Analysis Points and Attachment A-2.2 Sample Calculation included in Attachment A-2 Settlement Calculations. The second Attachment A-2.1 and Attachment A-2.2 look likely to be useless because there is no any settlement point shown on the cell plan and it is not known why a MC VI-E Phase II NE cell plan appears in Attachment A-2.1. In addition, only two result tables are included in Attachment A-2.2 without any explanation.

Response:

Lines and profiles are added to the plans per previous comments # 13 and 14.

Attachment A-3 Pipe Strength and Deflection Calculations

17. What type of leachate collection system was used in the existing landfill MC I (Figure 5)? What was the designed sub-grade slope for leachate drainage? Is it possible the new extra waste filling due to the vertical expansion will cause the slope reversal of the cell floor of

the existing landfill? Were some leachate collection pipes installed in the existing landfill? If so, the slope, strength, and deflection of the pipes must be re-evaluated under the new load due to the vertical expansion.

Response:

There is no existing lateral leachate collection system in MC I.

Attachment A-4 Leachate Collection System Flow Capacity Analysis

18. The calculations for the design of the sumps cannot be found in Attachment A-4 Leachate Collection System Flow Capacity Analysis. The design of the sumps include the sump sizing, the calculation of the effective storage volume of the sumps, determination of the pump-on and pump-off levels in the sumps, and calculation of the pump cycle time. The depth of the sump, elevation of the invert of the leachate collection pipe outlet at the sump, and actual elevations of the pump-on and pump-off at the sump were also not shown in the Engineering Drawings or indicated in the Engineering Report.

Response:

The typical sump dimension was provided in Detail 1 on Sheet 18 of the Engineering Drawings. The sump depth was 3.2 ft. Per your request, the calculation of the sump size, effective storage volume, and pump cycle time is performed and included in this response package. We will also include the following table on the Engineering Drawing (will be submitted in later time).

Parameter	Symbol	Value	Unit	Notes and Equations
Sump Length (Top)	L_T	44	ft	Source: Drawing SH18 Leachate Collection System Details
Sump Width (Top)	W_T	34	ft	Source: Drawing SH18 Leachate Collection System Details
Sump Top Area	A_T	1,496	ft^2	L X W
Sump Length (Bottom)	L_B	25	ft	Source: Drawing SH18 Leachate Collection System Details
Sump Width (Bottom)	W_B	15	ft	Source: Drawing SH18 Leachate Collection System Details
Sump Bottom Area	A_B	375	ft2	L X W
Depth of Sump	h_0	3.2	ft	Based on Slope 3:1 and Sump Dimensions
Porosity of Aggregates	n	0.3		assumption
Pump On from Bottom	h_1	36	inch	
Pump Off from Bottom	h_2	12	inch	
Pump On Area	A_1	1,419	ft^2	L=43 ft; W=33 ft
Pump Off Area	A_2	651	ft^2	L=31 ft; W=21 ft
Leachate Generation Rate	Q_{in}	47,693	GPD	Source: Attachment A-4_Leachate Collection System Flow Capacity Analysis
		33.1	GPM	
Leachate Pumping Rate	Q_{out}	37.0	GPM	typical pumping rate; minimum pumping rate is 34 gpm.
Effective Sump Storage Volume	V_{eff}	4,535	gal.	$V_{eff} = \frac{1}{3} n \frac{h_1 - h_2}{12} (A_1 + A_2 + \sqrt{A_1 A_2}) \times 7.481$
Pump Off Time	T_{off}	2.3	hr	$T_{off} = \frac{V_{eff}}{Q_{in}} \times 24$
Pump On Time	T_{on}	19.5	hr	$T_{on} = \frac{V_{eff}}{Q_{out} \times 60 - \frac{Q_{in}}{24}}$
Pump Cycle Time	T_{cycle}	21.8	hr	$T_{cycle} = T_{on} + T_{off}$
Pump Cycle	N	1.1		$N = \frac{24}{T_{cycle}}$

NOTES
1. Dimension of the sump used in this calculation is an example. The actual dimension of the sump can be varied.
2. The effective sump storage volume presented in this calculation is a minimum required volume.
3. The leachate pumping rate presented in this calculation is for illustration purpose. Other size of pump with different flow rate can be used as long as the leachate level above the primary liner can be maintained below 12-inch.

Attachment A-6 Surface Water Management System Design Calculations

19. It can be seen on Drawing No. 2B Revised Stormwater Management Systems dated April 21, 2020 prepared by CTI (also see Figure 6) that a diversion berm is located at the west boundary of MC VI-G4 and MC VI-G5 and a diversion berm is located at the south boundary of Master Cell I. There is no any diversion berm or drainage trench located at the south boundary of the new vertical expanded cells MC VI-G5, MC VI-G6, and MC VI-G7. CTI should explain whether a diversion berm or drainage trench is needed at the south boundary of MC VI-G5, MC VI-G6, and MC VI-G7 and a temporary diversion berm or drainage trench is also needed at the east boundary of MC VI-G7. If need, the new design for these diversion berms or drainage trenches should be added in Attachment A-6 Surface Water Management System Design Calculations. If not, CTI also need to provide the reasons.

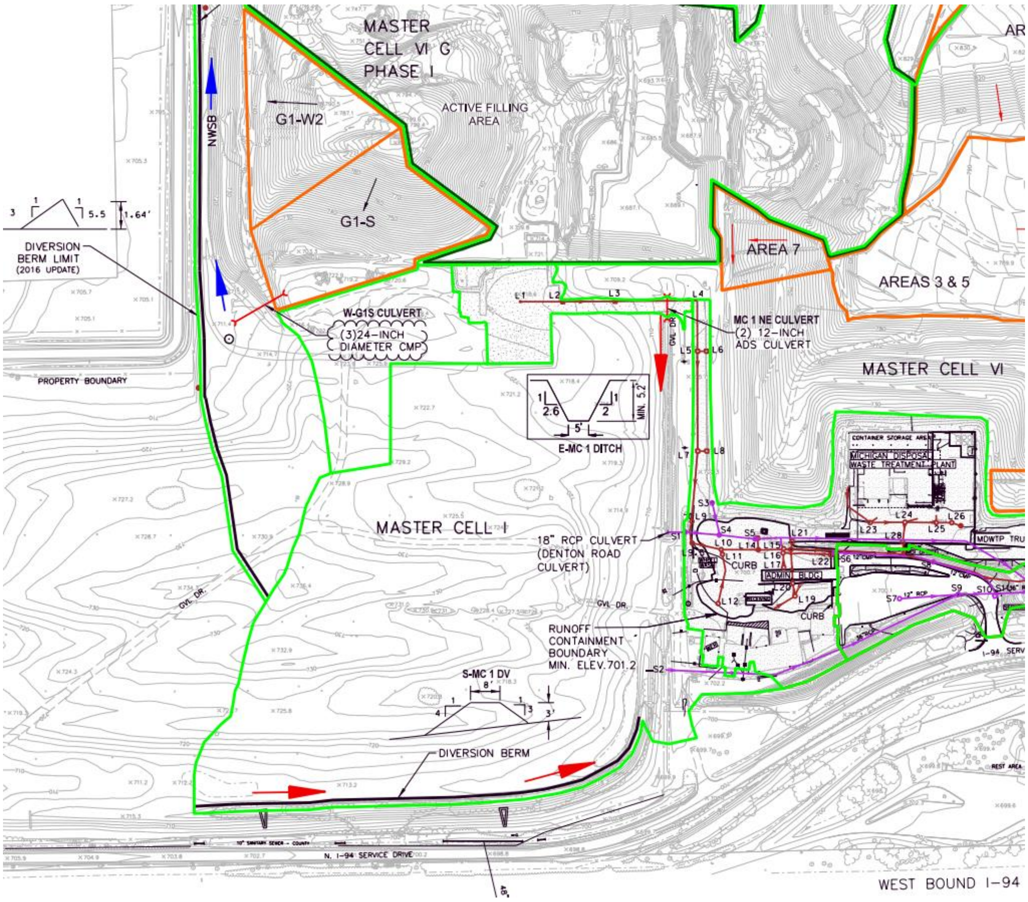


Figure 6 – Revised stormwater management systems dated on April 21, 2020

Response:

The Revised Stormwater Management Systems dated April 21, 2020 prepared by CTI is prepared for the current conditions at WDI. The design in this permit is for future conditions after all cells are completed and the final cover is installed.

All necessary stormwater management structures are included on Figures 1 and 2 provided in Attachment A-6.1.4 and these structures are included in the analysis as required to manage run-off from the final cover. The calculations provided in Attachment A-6, and Figures 1 and 2 provided in Attachment A 6.1.4, show the evaluation of the required surface water management structures south of the final cover. No additional surface water management structures beyond those discussed in Attachment A 6 are needed.

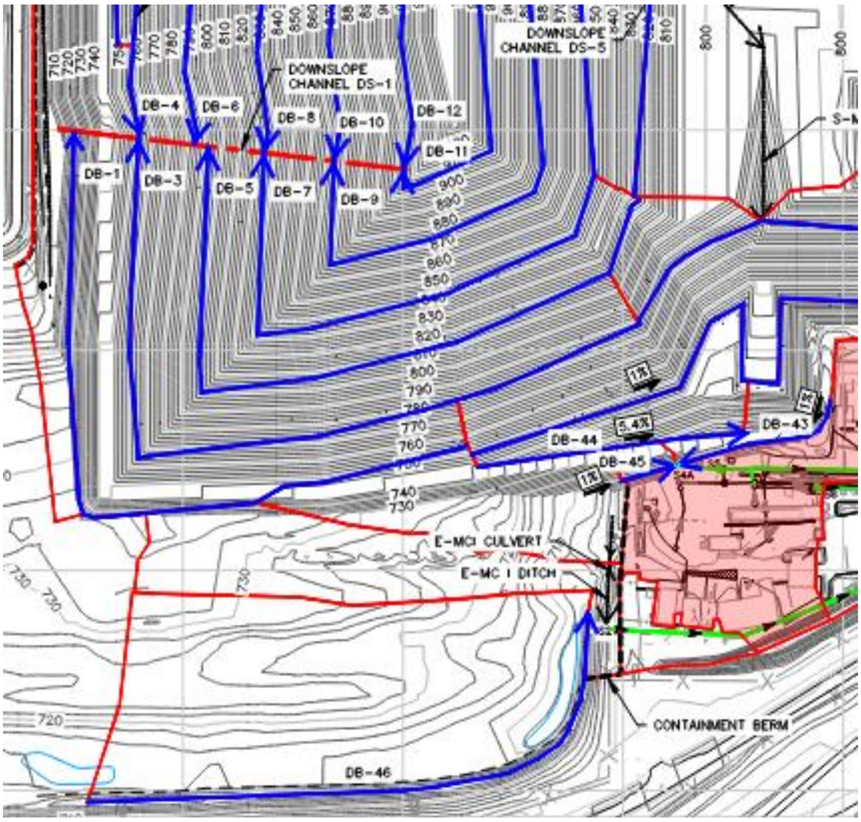


EXHIBIT 6A: Portion of Figure 1 of Attachment A-6.1.4

20. Only the diversion berms and downslope channels are shown on Drawing Nos. 10 and 11 for the traditional cover system. The peripheral drainage system at the boundaries of the new vertical expanded cells MC VI-G4, MC VI-G5, MC VI-G6, and MC VI-G7 are not marked on these stormwater management drawings.

Response:

Engineering Drawings 10 and 11 are intended to show the surface water management features directly associated with the final cover. The peripheral drainage system features are shown in Figures 1 and 2 provided in Attachment A-6.1.4. Calculations that discuss the evaluation of the North Sedimentation Basin, South Sedimentation Basin and related surface water control structures/features are provided in Attachment A-6. The North and South Sedimentation Basins and related surface water management features have sufficient capacity to manage surface water run-off flows under final conditions.

ATTACHMENTS

A-1.1 MC VI-G4 through G7 Slope Stability Analysis
A-1.2 MC VI-E and F Slope Stability Analysis
A-2.1 Settlement Analysis for MC VI-G4 to G7
A-2.2 Settlement Analysis for MC VI-E, F and G1 to G3

WDI Permit Cal List of Attachments for A-1 and A-2.xlsx

RESPONSE TO EGLE TECHNICAL COMMENTS ON WDI 2021 PERMIT APPLICATION

Attachment B Engineered Artificial Turf Cover Alternate

Commented by EGLE on March 22, 2022

Responded by CTI on April 18, 2022

Please note the figures referenced by EGLE and the list of references at the end of the original document are appreciated but have been eliminated from this response for clarity. EGLE comments are reproduced in italics below for convenience. CTI responses follow in blue color.

1. *Per the Part 111 Rules requirement for the equivalent demonstration of the alternative final cover design, CTI should demonstrate if the proposed engineered artificial turf cover system is equivalent to the standard Part 111 cover system shown in Figure 1 and not a conventional solid waste cover system or a so-called permitted cover system. For example, the title of Table 1 – Comparison of Engineered Artificial Turf Landfill Cover to Convention Cover in 3.0 Equivalency Demonstration should be changed to Table 1 – Comparison of Engineered Artificial Turf Landfill Cover to Part 111 Rules Hazardous Landfill Cover, and the cover system used to conduct HELP model evaluation in Attachment I: Hydrologic Evaluation of Landfill Performance (HELP) Model also should be a standard Part 111 cover system vs. a proposed engineered turf cover system, and so on.*

Response: Table 1 in Section 3.0 is a qualitative analysis of the Engineered Artificial Turf Landfill Cover to the cover system called out in Part 111 of the Rule. The title of Table 1 has been modified to be: "*Equivalency of EATLC to Part 111 Rules Hazardous Landfill Cover*"

The HELP model for the conventional final cover (Attachment B1 of Attachment I) have been modified to the same requirements in Part 111.

The results from this revised model did not significantly change (Table 1 and 2 in Attachment I).

2. *CTI should indicate that the reinforced GCL must be used on the slope when the cover slop is greater than 10%.*

Response: Reinforced GCL is planned for use at WDI as part of the cap system on all landfill slopes. The following language has been added to the Engineered Turf Final Cover Detail Drawings: *"Reinforced GCL must be used on all slopes as part of the closure system*."

3. *GCL should be minimized expose time and be covered by geomembrane, engineered turf, and sand infill to minimize bentonite drying in the GCL during installation period if the final cover system is constructed in high temperature season. The normal moisture content of the bentonite in GCL is 35%. The shrinkage for the different types of GCLs can be up to from 7 to 22% (Thiel et al., 2006). Koerner and Koerner (2005a, 2005b) considered three possible mechanisms for GCL panel shrinkage: (1) Cyclic wetting and drying; (2) Longitudinal steep slope tensioning of the GCL; and (3) GCL contraction*

on relatively flat slopes caused by "gathering" under textured geomembranes. Of these three possibilities they believed that Mechanism 2 (slope tensioning) is the major factor that contributes to the problem in the field. Three actual projects in which GCL panels had either lost a portion of their original overlap or had completely separated are shown in Figure 2. The overlap distance is typically 6 to 12 inches (150 to 300 mm). The warm temperatures can reduce the moisture content of the bentonite and cause reduction or even complete loss of overlap distance. This is obviously unacceptable. If reduction in overlap distance from elevated temperatures is anticipated, the overlap should be increased (USEPA, 1993; Daniel and Koerner, 2007). In addition, unlike conventional Part 111 cover system, the proposed engineered artificial turf cover system does not have 2-ft protective soil and topsoil load applying on the GCLs and the normal load over the GCL is very small. It makes more potential movement and deformation of the GCLs due to construction equipment movement during construction, or temperature deformation and other effects during operation and long-term closure periods. The languages regarding minimize expose time for GCL during installation period and increasing the GCL overlap distance on the sideslope area should be added in the construction specifications.

Response: We agree that due to the lower confined normal pressure, the GCL could be subjected to the shrinkage issues under the artificial turf. However, comparing to the bare geomembrane cover over the GCL, the GCL in the EATLC is also protected by geomembrane, artificial turf, and sand infill. The confined normal pressure is significantly higher than the bare geomembrane. The artificial turf will also reduce UV radiation exposure for the geomembrane. With the sand infill layer, the temperature variation of the EATLC will be significantly lower than GCL under bare geomembrane. At this point, there is no study conducted on the potential GCL shrinkage in the EATLC to our knowledge. To take a conservative approach, we will propose the overlap distance based on information from the studies on GCL under bare geomembrane. We will revisit this issue when more information is available.

The GCL overlap between adjacent panels has been increased to 18-inches, which is the higher end of the range recommended in Thiel and Rowe (2010), to compensate for separation. In addition to the increased overlap distance, the installation procedure will include the use of heat tacking all seams, which was also recommended as a strategy to control shrinkage (Thiel and Rowe 2010). The following language has been added to the Engineered Turf Final Cover Detail Drawings:

"All geosynthetic clay liner (GCL) shall be covered with geomembrane within 24 hours after placement of GCL and overlapped a minimum of 18-inches during installation

All overlapped seams are to be heat-tacked using a quick pass of a flame torch followed by quick application of appropriate pressure as provided by a roller, foot pressure, or other means."

4. *CTI should provide the detail information to explain what type of geomembrane will be used in the proposed engineered artificial turf cover system, a special 50-mil LLDPE Agru Super Gripnet® geomembrane developed by Agru America, Inc. for a standard ClosureTurf cover system shown in Figure 3 or a general 40-mil textured HDPE geomembrane. If a special 50-mil LLDPE Agru Super Gripnet® geomembrane is selected, a drainage layer will be located between the geotextile base of the engineered synthetic turf and the Agru Super Gripnet® geomembrane (see Figure 4 and Figure 5). Then, Detail 1/19A Final Cover System in Drawing No. 19A must be revised to show this drainage layer in the cover cross section.*

Response: A special 50-mil LLDPE Agru Super Gripnet membrane is not expected to be needed for the slopes at WDI. Instead, as noted in the submittal, the version that includes a 40-mil textured HDPE membrane is expected to be used. Based on the veneer stability analysis (Attachment C), it will be adequate for maintaining the integrity of the overlying materials and will adhere to the underlying substrate.

5. *There is no any drainage trench located at the toe of the proposed engineered artificial turf cover system (see both Detail 3/19A Dike and Cover Tie-in Detail (Typ.) MC VI-F, MC VI-G, & East Side of MC VI-E and Detail 4/19A Dike and Cover Tie-in Detail North Side of MC VI-E. CTI should explain why the toe drainage is not needed for the proposed engineered artificial turf cover system. The traditional and artificial turf cover systems have different materials, structures, and thickness. They may cause the different amounts of the surface run-on and run-off, lateral drainage, soil moisture storage, infiltration, and evaporation and evapotranspiration. Because the proposed cover system uses the artificial turf that will not have the transpiration function and only have the evaporation function. In addition, this type of cover system does not have 2-ft soil protective layer and topsoil and also does not have soil moisture storage function. Thus, if the engineered turf cover system is used to replace the traditional cover system, the amount of the surface run-off may be increased.*

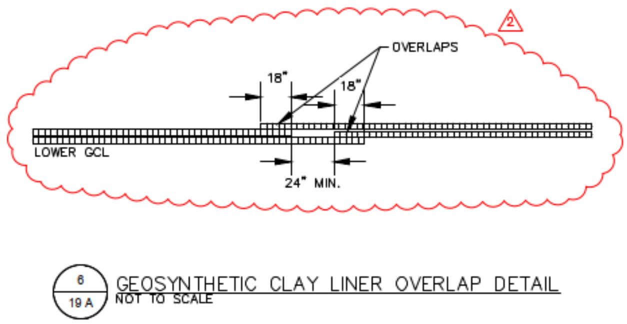
Response: The amount of runoff from EATLC is expected to be greater than traditional cover for the reasons noted. An evaluation of the surface runoff was conducted utilizing HydroCAD and recommended input parameters from the Closure Turf manufacturer. This evaluation was included in Attachment K. The weighted curve number for the EATLC is 95 which is significantly higher than the value used for the conventional cover which is 84.

Because the runoff remains on the surface, there is no need for a subsurface drainage collection system or toe drainage. Details 3, 4, and 5 were focused specifically on the different layers of the EATLC cap and their interaction/tie-ins with each other and the liner system. However, the closure turf will be terminated beyond a perimeter ditch. These perimeter ditches will capture the stormwater runoff which is managed above the liner/turf. Details 4 and 5 in Sheet 19A have been modified with a typical perimeter ditch for clarity.

6. *Two layers of GCL will be used in the engineered artificial turf cover system to replace 3-ft compacted clay layer. CTI should add a notice in construction specifications to stagger the overlapping positions of upper and lower layers of GCLs and avoid the side edge and end overlaps of the GCLs at the same locations for both layers of GCL*

Response: The following language has been added to the Engineered Turf Final Cover Detail Drawings: *"The Contractor shall install the two layers of the GCL staggered by a minimum 24-inches between upper and lower GCL seams."*

The following detail has also been added on Sheet 19A.



7. *The geomembrane used in the proposed cover system shown in Figure 1 Cross Section (Typical ClosureTurf Cover) of Attachment C Veneer Stability Analysis (Page 52) is 40-mil LLDPE. However, the geomembrane used in the proposed cover system shown in Figure 2 Engineered Artificial Turf Landfill Cover System in 2.0 Configuration of the Final Cover System is 40-mil Textured HDPE Geomembrane. CTI must clarify this discrepancy.*

Response: The veneer stability analysis has been revised to be consistent with Figure 2 in Section 2.0 (using 40-mil HDPE geomembrane). This change does not alter the results or conclusions of the veneer stability analysis.

8. *The interface shear strengths for the current proposed engineered turf cover system is not only the interface friction angle between engineered components above geomembrane, δ_A (i.e., the interface friction angle between the engineered turf and geomembrane) and the interface friction angle between engineered components below geomembrane, δ_A (i.e., the interface friction angle between the geomembrane and soil). Based on the designed engineered turf cover system, the interface shear strengths that must be considered include the interface between the (1)base geotextile of the engineered turf and geomembrane, the (2)interface between the geomembrane and the top layer of GCL, the(3) interface between two layers of GCLs, and the (4)interface between the bottom layer of GCL and leveling soil layer. In addition, (5)the internal shear strength of two layers of GCL should also be considered in possible hydrated condition. All these considerations were not mentioned Attachment C: Veneer Stability Analysis. CTI must add some design and construction specifications to explain how to determine the critical interface or internal shear strength for the proposed final cover*

system to meet the required shear strength based on the calculations.

Response: The following language has been added to the Engineered Turf Final Cover Detail Drawings:

Construction of the final cover system shall be supported by a veneer stability analysis. Each geosynthetic/geosynthetic and geosynthetic/soil interface included in the EATLC system shall be considered for prequalification testing prior to installation of the cover system. The critical interface shall be determined by a comparison of interface strength test results of representative samples of materials used for construction for each interface. The critical interface is the lowest shear strength test result that corresponds to the normal stress exhibited by the EATLC cover system which is expected to be between 5 and 6 psf.

At minimum, the following interfaces shall have interface test results for comparison.

- *Base geotextile of the EATLC and Geomembrane*
- *Geomembrane and Top layer of GCL*
- *Top layer of GCL and Bottom layer of GCL*
- *Bottom layer of GCL and Leveling soil layer*
- *Internal shear strength of the GCL under hydrated conditions*

	Veneer Stability Interface Strength Requirements	
	Final Cover Slopes > 10%	Final Cover Slopes ≤ 10%
Required Interface Strength Test Data	Large-Displacement Strength	Peak Strength
Min. Equiv. Acceptable Interface Strength*	18 Degrees	20.5 Degrees
Min. Acceptable FS for Veneer Stability	1.3	1.5

*Combinations of tested interface friction angle and shear strength intercept measured at the normal stress (5 to 6 psf) exhibited by the EATLC cover system that satisfies the factor of safety criteria are acceptable.

9. *Landfill usually settles a considerable amount during the filling operation and even after closure. The settlement of the waste fill causes the large deformation of the landfill cover system, which tends to induce shear stresses in the final cover system. These shear stresses induce large shear displacements along specific interfaces in the cover systems that may lead to the mobilization of reduced or residual interface strength (Mitchell et al., 1990; Seed et al., 1990). In addition, thermal expansion and contraction of the final cover systems during construction may also contribute to the accumulation of shear displacements and the mobilization of residual interface strength (Stark and Poeppel, 1994). Filz et al. (2001) conducted finite-element analyses using a displacement softening interface to investigate the effects of progressive failure on the slope stability of the geosynthetic lined and covered landfills. It was found that the interface (or internal) shear strength of the liner and cover systems on the sideslope are almost always in post-peak condition (i.e., in large-deformation or residual condition) even if the waste height is not high. It can be seen that the waste depth can be up to 210 feet from Drawing No. 12 of Attachment C of B6_Permit Engineering Drawings and the slope height of the final cover can be up to 190 feet from Drawing Nos. 12 and 14 of Attachment C of*

B6_Permit Engineering Drawings. The vertical riser pipes installed in MC VI-A-to-D have been damaged due to the waste settlement caused by about only 80-ft waste depth as shown in Figure 6. This shows that the settlement that occurs in the hazardous waste landfill is also quite large. Therefore, residual interface or internal shear strengths should be assessed and considered for the final cover system for a normal design condition to make a long-term landfill stable and safe even after being subjected to large waste settlements (Byrne, 1994; Gilbert et al., 1996; Esterhuizen et al., 2001; Filz et al., 2001).

Response: The following language has been added to the Engineered Turf Final Cover Detail Drawings:

Interface friction tests performed to qualify acceptable materials for placement of EATLC cover materials shall consider large-displacement strength test results on the final cover side-slopes steeper than 10% and peak strength test results for all other slopes. The minimum design factor of safety for static stability shall be 1.3 for final cover side-slopes steeper than 10% that consider large-displacement strength testing results. The minimum design factor of safety for static stability shall be 1.5 for all other final cover slopes that consider peak strength.

10. *A testing report entitled "Aerodynamic Evaluations of Closure Turf Ground Cover" included in Attachment D Wind Uplift Evaluation indicated that the geomembrane used to conduct test to evaluate the aerodynamic properties by GTRI is the 50-mil LLDPE Agru Super Gripnet® geomembrane and not a general 40-mil textured HDPE geomembrane indicated in the CTI proposed report. Therefore, the aerodynamic properties investigated by using the 50-mil LLDPE Agru Super Gripnet® geomembrane cannot be used to represent the aerodynamic properties for a general 40-mil textured HDPE geomembrane. CTI can have three options: (1) change to use the same tested 50-mil LLDPE Agru Super Gripnet® geomembrane, (2) use the selected 40-mil textured HDPE geomembrane to rerun the aerodynamic test, or (3) use theoretical certification and practical experiences to demonstrate the aerodynamic properties of the selected geomembrane is equivalent to or better than the tested 50-mil LLDPE Agru Super Gripnet® geomembrane.*

Response: Super Gripnet is not necessary for this application. Rerunning this test specifically for the 40 mil textured HDPE geomembrane is also not practical given the current access to the testing apparatus used to generate the test results provided. It should be noted that the fundamental aspect being measured in the aerodynamic testing is the ability of the turf to adhere to the underlying geomembrane without any mechanical connection between the two besides frictional resistance. By inspection, the Super Gripnet represents a conservative scenario in relation to the textured HDPE geomembrane. For the textured HDPE geomembrane, there is no appreciable gap between the geomembrane and the overlying geotextile backing allowing for more intimate contact and a greater degree of resistance to movement.

A recent paper (Zhu, M., at el. 2022) offers an additional method of analysis for evaluating the resistance of EATLC to uplift pressure. Using the results from the wind tunnel experiment, a mean wind uplift pressure was calculated for the site based on ClosureTurf above a textured geomembrane (Attachment D2 - Amendment #1 Calculations). The calculated maximum uplift pressure was 3.60 psf and the weight of the ClosureTurf cover is 5.4 psf per unit area. The factor of safety for wind uplift is 1.5 under worst case wind conditions per ASCE 7-16.

11. *Similar to above Comment 10, a 50-mil LLDPE Agru Super Gripnet® geomembrane and not a proposed 40-mil textured HDPE geomembrane was used to conduct the ClosureTurf Integrity Study by SGI Testing Services, which was reported in Attachment E: ClosureTurf Integrity Study. The study was based on a condition that the 50-mil LLDPE Agru Super Gripnet® geomembrane was directly placed on the foundation soil and not on the GCL, see Figure 7 (i.e., Figure 1 in Attachment E) and Figure 8 (i.e., Figure 2 in 1.0 Purpose). Thus, there are at least two conditions for the proposed engineered turf cover (i.e., geomembrane and the material beneath the geomembrane are different with the demonstration study including in Attachment E. CTI must provide detailed description to explain whether or not the study results presented in Attachment E based on the cross section shown in Figure 7 is also suitable for the cross section shown in Figure 8. If so, why?*

Response: Review of the calculations provided in the demonstration performed by SGI show that the only fundamental difference in the demonstration and the proposed plan for WDI is how the material will perform on a slope, because the friction angle anticipated using 40-mil textured HDPE geomembrane is slightly lower than what would be anticipated using Super Gripnet. That is the only input parameter that varies from the assumptions in the SGI analysis and no other characteristic differences between the two geomembranes are germane to the analysis.

The submittal has been revised to include an amendment (Attachment E – Amendment #1) using the minimum friction angle derived from the veneer stability analysis. It shows a factor of safety lower than the result included in the SGI analysis, but still acceptable for traffic on the slope.

12. *It can be seen from Figure 9 (i.e., Detail 3/19A in Drawing No. 19A of Attachment A: Drawings) that there are total eight (8) layers of geosynthetic materials will buried in a anchor trench. They are a layer of geogrid, 4 layers of GCL, 2 layers of 80-mil HDPE geomembrane, and a layer of double-side geocomposite. Although the anchor trench has expanded to 3-ft wide and 3-ft deep, it is impossible to bend 8 layers of geosynthetic materials to two 90-degree angles as shown in Figure 9, especially for the geocomposite with relatively large thickness and not easily curved. It is suggested that the end of the geocomposite can be stopped at the top edge of the anchor trench and not extended into the anchor trench. This will much improve the anchoring function of the barrier materials buried in the anchor trench.*

Response: The details on Drawing 19A have been modified to show the termination of the geocomposite prior to the bend in the anchor trench.

13. *Drawing Nos. 10 and 11 in the Permit Engineering Drawings show the storm water management and sedimentation plan, which include the diversion berms parallel to the contour lines and the downslope channels perpendicular to the contour lines. Drawing Nos. 10A and 11A in the Permit Engineering Drawings and Attachment A: Drawings of Attachment B: Engineered Artificial Turf Landfill Final Cover Alternate show the storm water management and sedimentation plan by using the alternative engineered artificial turf cover system. No any downslope channel was shown on this alternative cover system. The diversion berms were only arranged near the south toe area and a part of the east toe area. Do the size of the diversion berms used on the alternative cover system are the same as the diversion berm used on the traditional cover system because the details of the cross section sizes of the diversion berms are not shown on Drawing Nos. 10A and 11A?*

Response: The diversion berms used for the engineered artificial turf cover (EATLC) system are larger than the diversion berms designed for the traditional final cover.

The diversion berms for the traditional cover system are shown on Drawings 10 and 11, and are discussed in detail in Appendix A-6.2 of the original submittal. Except DB-36, the traditional cover diversion berms are 2 ft high. DB-36 is 2.5 ft high.

The diversion berms for the EATLC are shown on Drawings 10A and 11A, and are discussed in detail in Appendix K-2 of the original submittal. The diversion berm heights for the EATLC are generally 2.5 or 3.0 ft high. Due to the small drainage area, the 2 ft high DB-G berm is the only EATLC berm less than 2.5 ft high.

14. *Only the diversion berms are shown on Drawing Nos. 10A and 11A for the alternative engineered turf cover system. The peripheral drainage system at the boundaries of the new vertical expanded cells MC VI-G4, MC VI-G5, MC VI-G6, and MC VI-G7 are not shown on these stormwater management drawings.*

Response: Engineering Drawings 10A and 11A are intended to show the surface water management features directly associated with the final cover. The peripheral drainage system features are shown in Figures 1 and 2 provided in Attachment K-1.5 of the original submittal. Calculations that discuss the evaluation of the North Sedimentation Basin, South Sedimentation Basin and related surface water control structures/features are provided in Attachment K-1.6. The North and South Sedimentation Basins and related surface water management features have sufficient capacity to manage surface water run-off flows under final conditions.

REFERANCES

Thiel, R., & Rowe, R. K. (2010). Technical Developments related to the Problem of GCL Panel Shrinkage when placed below an Exposed Geomembrane. 3rd International Symposium on Geosynthetic Clay Liners.

Zhu, M., Sarkar, P., Hou, F., & Zheng, J. (2022), Wind Tunnel Study and Uplift Analysis of Geosynthetic Covers. Geo-Congress

1. Responses to EGLE comments dated May 5, 2022

DRAFT

Comments on CTI's Responses for MMD's Technical Comments

May 5, 2022

Responded by CTI on May 18, 2022

Attachment A-2.1 Settlement Analysis for MC VI-G4 to G7

1. Figure 2 (Page 3 of 13 of Attachment A-2.1) is a north-south cross section. The north and south directions should be marked on the left or right side of Figure 2 or add the arrows on the north-south cross section line in Figure 1 (Page 2 of 13 of Attachment A-2.1). As the same, Figure 3 (Page 4 of 13 of Attachment A-2.1) is a west-east cross section. The west and east directions should be marked on the left or right side of Figure 1 or add the arrows on the west-east cross section line in Figure 1.
   Response: we have added these labels as requested.

2. The secondary compression ratio shown in Table 2 (as depicted below) is 0. Although the biodegardation of the old waste in the existing cells may be close to zero, the structural creep of the waste skeleton still exist like secondary compression of soils. Thus, theretically, the secondary compression ratio of the existing waste will not be zero and it will reduce to a value less than that of the new waste.

Table 2. Compressibility Parameters of Waste and Soils
(on Page 6 of 13 of Attachment A-2.1)

Soil Type	Primary Compression Ratio $C_c/(1+e_0)$	Secondary Compression Ratio $C_\alpha/(1+e_0)$	Recompression Ratio $C_r/(1+e_0)$
Existing cover	0.102[B]	0.005[B]	0.017[A]
Existing waste	0.147	0	0.0245[A]
In-situ middle clay	0.102	0.005	0.017[A]
In-situ lower clay	0.171	0.009	0.0285[A]
In-situ silt	0.15[B]	0[B]	0[B]
In-situ sand	0.1[B]	0[B]	0[B]

[A] Estimated from $C_r = C_c/6$.
[B] Assumed values.

Response: we have added secondary compression to the calculations.

3. The material of the intercell dikes should be the natural clay. It may be in-situ middle clay or in-situ lower clay shown in Table 2 (as depicted above). This clayey material should be the overconsolidated clay. The settlement of the intercell dikes in the existing cells should be calculated by using the method of the overconsolidated soils.

   Response: the preconsolidation pressure for the in-place clay layers has been calculated using the vertical overburden pressure due to the pre-development ground surface(elevation 705 – as determined from undeveloped portions of the site depicted in the permit topo maps). All subsequent changes (i.e., MC-I excavation, waste, placement, and cover soil placement) are subsequently modeled. Finally, weight from the proposed overlying waste cells are added to compute the final vertical stress. The net settlement due to the change from current conditions to proposed final conditions is calculated as either normally-consolidated (i.e., compression is calculated using C'_c) or overconsolidated (i.e., compression is calculated using C'_r and C'_c) depending on whether the current stress is greater than the preconsolidation stress or not, respectively.

4. Because the precise configuration of the intercell dikes is unknown, the locations of the intercell dikes shown in Figure 1 (Page 2 of 13 of Attachment A-2.1) and a figure in Attachment A-2.2.1 Settlement Analysis Points (Page 16/total page 70) are based on the estimation and assumption. If the precise profiles and locations of the intercell dikes can be determined after the excavation of the cell subgrade in MC VI-F1 to F4 and MC VI-G4 to G7, CTI should make comparisons of the profiles and locations between the estimated and actual intercell dikes to see whether or not the settlement calculations results based on the estimated intercell dikes are still acceptable at the conservative side. If not, the recalculations should be conducted based on the new information. The new drawings shown the precise profiles and locations of the intercell dikes in MC VI-F1 to F4 and MC VI-G4 to G7 should be developed to submit to MMD, Michigan EGLE.

   Response: Current design and calculations are prepared using best available information regarding the existing waste and configuration of the existing dikes. A procedure to evaluate additional information during construction has been provided. If an engineering evaluation of the additional information results in a recommendation to alter the liner grades, the construction quality assurance officer will evaluate the change to determine that it provides equivalent or better certainty that components meet the design specifications. The construction quality assurance officer will provide certification of this evaluation in the construction certification report submitted to EGLE.