REPORT

Geotechnical Design of MSE Wall - Preliminary

Erskine Park Landfill

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1.0 INTRODUCTION

Enviroguard Pty Ltd (Enviroguard) has engaged Golder Associates (Golder) to provide design for the proposed Mechanically Stabilised Earth (MSE) wall for the Erskine Park landfill at 4 Quarry Rd, Erskine Park NSW 2759.

The MSE wall is proposed at the site for increasing the capacity of the landfill. The wall is to be built around approximately 0.9 km of the perimeter of the landfill on the south-western, southern and eastern extents of the landfill. The toe of the wall is located a minimum 5 m offset from the project boundary. The maximum height of the wall is approximately 20 m. The top of the wall is about 12 m to 13.5 m wide at the top, to facilitate a roadway, barrier, shoulder and surface drainage.

1.1 Scope

The purpose of this technical memo is to provide the geotechnical design details of the preliminary MSE wall based on anticipated ground conditions, interpreted geotechnical parameters, minimum geometry, construction sequences, and potential loading on the walls.

1.2 References

The standards, codes and documents adopted for the design of the MSE wall are provided below.

- Design and Construction of Mechanically Stabilized Earth Walls and Reinforced Soil Slopes – Volume I, Publication No. FHWA-NHI-10-024, Federal Highway Administration FHWA GEC 011 – Volume I (FHWA GEC 011).
- AASHTO LRFD Bridge Design Specifications, Seventh. Edition, 2014.
- AS 4678-2002 - Earth-retaining structures.
- AS1170.4 -1993 - Minimum Design Loads on Structures, Part 4: Earthquake Loads.

1.3 Design Development

The design of the proposed retaining wall considers a design life of 100 years. The following factors have been considered in the design of the MSE wall:

Site and Subsurface Conditions:

- Ground conditions including thickness of different units.
- Extent of the existing landfill and quarry geometry.
- Topography of the existing ground in front and behind the wall.
- Groundwater and flood conditions.
- Short-term and long-term behaviour of the soils.
- Soil behaviour based on its history.

Geometry and Loading:

- Geometry of the MSE wall including project boundary, extent of the existing and future landfill, and the amount of air space for future landfill.
- Permanent load including the lateral earth pressure induced by the future landfill.
- Extreme loading events such as earthquake and collision loading.

- Future traffic conditions and drainage requirements.
- Potential variations in load during operation.
- Ground support for landfill lining and capping installation.

Construction Materials and Methods:

- Foundation requirements and foundation treatment (if required).
- Wall reinforcement by geogrid, including degradation over time for durability considerations.
- Material requirement for reinforced soil and liner support fill.
- Wall facing and its durability.
- Construction sequences including wall construction, capping installation and landfill placement.

Wall Stability and Deformation in Service:

- Stability of the wall including internal and external stability, and overall stability.
- Serviceability of the wall including settlement and lateral movement.

1.4 Wall Geometry

A schematic diagram of typical geometry of the MSE wall is shown in Figure 1. A summary of the wall geometry along the control line is provided in Table 1. For the initial assessment, the wall width at top of the wall (W) varies from 12 m to 13.5 m to accommodate a roadway and drainage system. This width includes the width of drainage chimney and facing. The preliminary sizing has accommodated space for two vehicle widths at the top of the wall. It is understood that the top of wall is to be used for vehicles exiting the project site in one direction. Hence two-way traffic at the top of the wall may not be required. There is an opportunity to reduce the wall width during design development. The design development could result in a single vehicle width forming part of the final design. This design refinement has been explored further in Section 10.

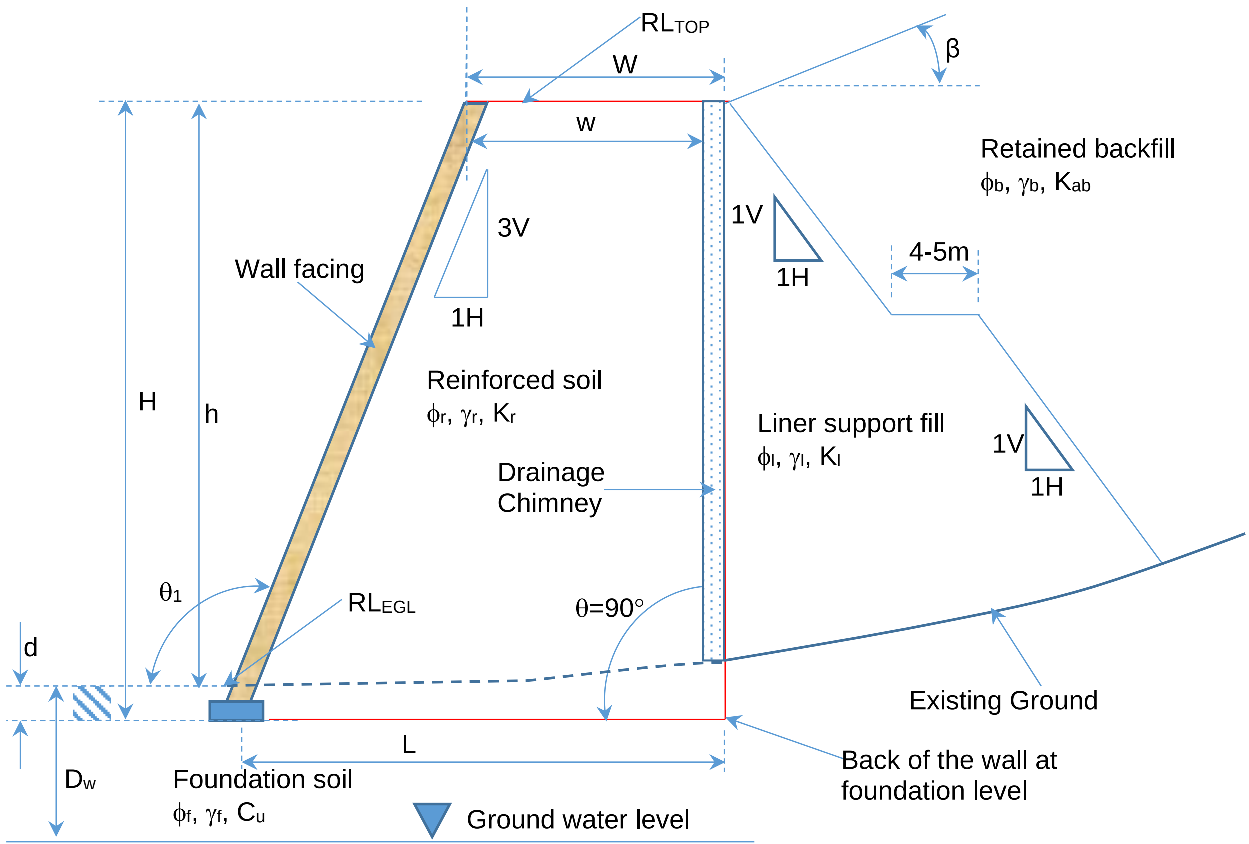


Figure 1: Typical Geometry of the MSE Wall

Table 1: Minimum Dimensions Adopted for the Preliminary Wall Design

Ch	RL_{TOP} (mAHD)	RL_{EGL} (mAHD)	W (m)[3]	h (m)	d (m)[1]	H (m)	Anticipated foundation material
100	69.001	64.227	12	4.77	1.20	5.97	Unit 1a - Fill
200	78.633	66.800	12	11.83	3.00	14.83	Unit 1b - Controlled Fill
300	79.980	67.611	12	12.37	3.00	15.37	
400	78.362	63.073	13.5	15.29	1.80	17.09	
500	76.774	61.342	13.5	15.43	1.80	17.23	
600	75.126	57.532	13.5	17.59	2.10	19.69	Unit 2 /Unit 4a (Note 2)
700	73.508	56.484	13.5	17.02	2.10	19.12	
800	71.890	62.333	12.85	9.56	1.20	10.76	
850	70.260	64.158	12.44	6.10	1.20	7.30	

Note

1. *Embedment depth of the MSE wall provided is based on the wall height, H and slope in front of the wall.*

2. *Landfill waste material is likely to be encountered below the foundation level between Ch650 to Ch850. Foundation treatment may be required to provide a stable ground condition below the wall. Refer Section 2.1.*
3. *Preliminary wall width sizing – refinement possible during detailed design.*

For the MSE wall in the region from Ch600 to Ch700, a minimum embedment depth of H/10 has been adopted considering the slope in front of the wall. The minimum embedment adopted is generally consistent with the references presented in Section 1.2. It is noted that greater embedment may also be required based upon bearing, settlement, and/or global stability calculations. The minimum horizontal bench in front of the MSE wall was maintained at 1.2 m.

Overall stability of the MSE wall is a key consideration due to the significant retaining wall height (up to 20 m), lateral earth pressure from the future landfill (i.e., landfilling of the additional airspace), in some areas the presence of an existing pond (slope in front of the wall), and in some areas the presence of significant uncontrolled fill below the foundation of the wall.

2.0 GROUND CONDITIONS

A site investigation program has been undertaken and a report prepared. The Geotechnical Investigation Report (ref:19135652-002-R-RevA) includes a detailed description of the anticipated subsurface conditions along the project alignment and the subsurface geotechnical units identified.

Geotechnical long and cross sections developed along the wall alignment based on the geotechnical site investigation data are provided in **APPENDIX A**.

The MSE wall is currently proposed to be founded on the uncontrolled fill (Unit 1a) from Ch25 to Ch100, Controlled fill (Unit 1b) from Ch100 to of Ch550 (approx.), and on Residual soil/Very low to low strength sedimentary rock from Ch550 to Ch920. In addition, landfill waste is likely to be encountered partly within the foundation (not to entire width of foundation) between Ch650 and Ch850. The details of the landfill extent are further discussed in Section 2.1 below.

Figure 2 shows the undrained shear strength estimate of the foundation soil based on CPT data. The estimated undrained shear strength is generally equal to or greater than 100 kPa below the depth of 2.0 m (except CPT 1005).

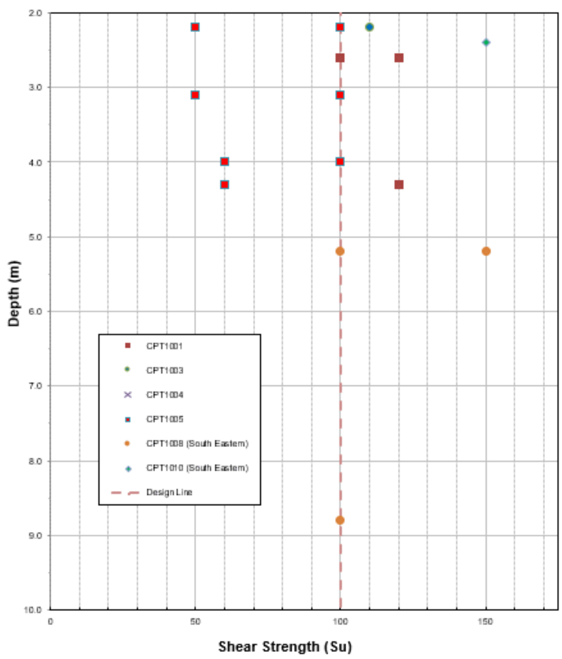


Figure 2: Undrained Shear Strength Based on CPT results

Figure 3 shows the SPT blow counts within the Unit 1a and Unit 1b. The data shows the large scatter ranging from 5 to 35. A design value of 15 has been adopted and this value can be correlated to an approximate undrained strength equal to the undrained shear strength estimated from the CPTs data.

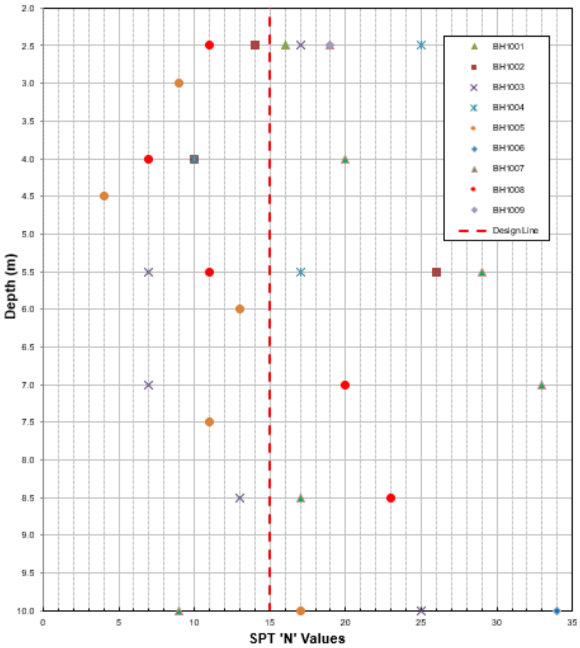


Figure 3: SPT blow counts in Unit 1a and Unit 1b vs Depth

Based on the available geotechnical investigation details, the subsurface profile has been characterised into a number of units, identifying the material consistencies for each unit likely to be encountered along the wall alignment. These units are provided in Table 2.

Table 2: Stratigraphic Units along Wall Alignment

Unit	Description
Unit 1a	Uncontrolled fill Gravelly, Sandy, Clayey fill
Unit 1b	Controlled fill Gravelly, Sandy, Clayey fill
Unit 1c	Landfill Waste - mix of cohesive and granular with consistencies of firm to stiff or loose to medium dense.
Unit 1d	Southern stockpile fill
Unit 2	Residual soil High plasticity silty clay/sandy clay, Very stiff
Unit 3a	Very Low and low Strength Volcanic Breccia and Dolerite
Unit 3b	Medium Strength Volcanic Breccia and Dolerite
Unit 4a	Very Low and Low strength Siltstone bedrock
Unit 4B	Medium strength or better Siltstone bedrock

2.1 Landfill below Wall Foundation

Information about the location of the edge of existing liner/waste has been provided by Enviroguard. Golder has also undertaken geotechnical investigation and geophysical surveys in some areas, and has considered available design and construction information from previous site works, to help verify the information provided. The outcome of the geotechnical investigation and geophysical survey is that there is broad agreement between the information provided and the recent investigation.

An area requiring close consideration is the area from Ch650 to Ch850 (approx.) where we infer that the edge of waste extends partially within the proposed foundation of the MSE wall, as shown in Figure 4.

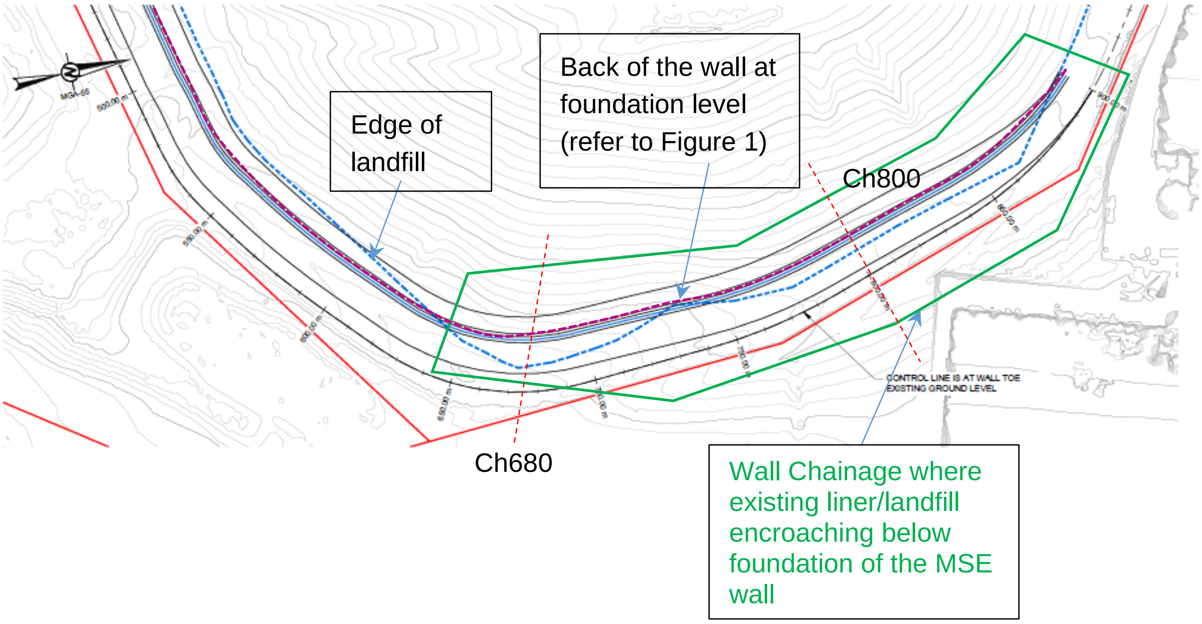


Figure 4: Extent of Existing Liner/Landfill related to the proposed MSE wall

Figure 5 shows the interpreted geotechnical sections at Ch680. The landfill is expected to extend approximately to the toe of the MSE wall. At this location, the thickness of the landfill is inferred to be approximately 2.5 m at the toe of the MSE wall and 7.2 m at the back of the wall as shown in Figure 5. As per the embedment depth proposed in Table 1, the MSE wall for this section requires a minimum embedment depth of 2.1 m (H/10) below the existing ground level (at toe of the wall). This depth of foundation depth will result in a landfill thickness of approximately 0.5 m at the toe of MSE wall foundation and 2.5 m at the back of the wall.

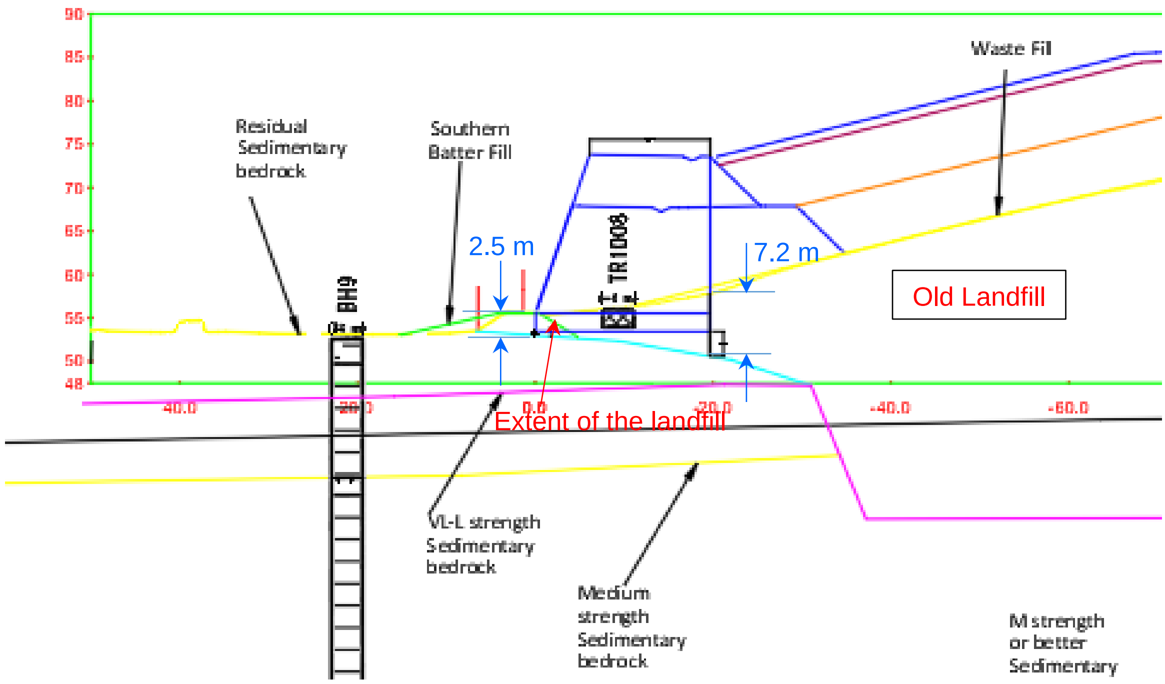


Figure 5: Interpreted Ground Condition at Critical Section (Ch680)

Figure 6 shows the interpreted geotechnical sections at Ch800. The landfill is expected to underlie approximately one third of the foundation from the back of the wall. At this location, the thickness of the landfill is approximately 8.5 m at the back of the wall. As per the embedment depth proposed in Table 1, the MSE wall for this section requires a minimum embedment depth of 1.2 m (H/10) below the existing ground level (at toe of the wall). This depth of foundation will result in a landfill thickness of approximately 4 m to 5 m at the back of the wall.

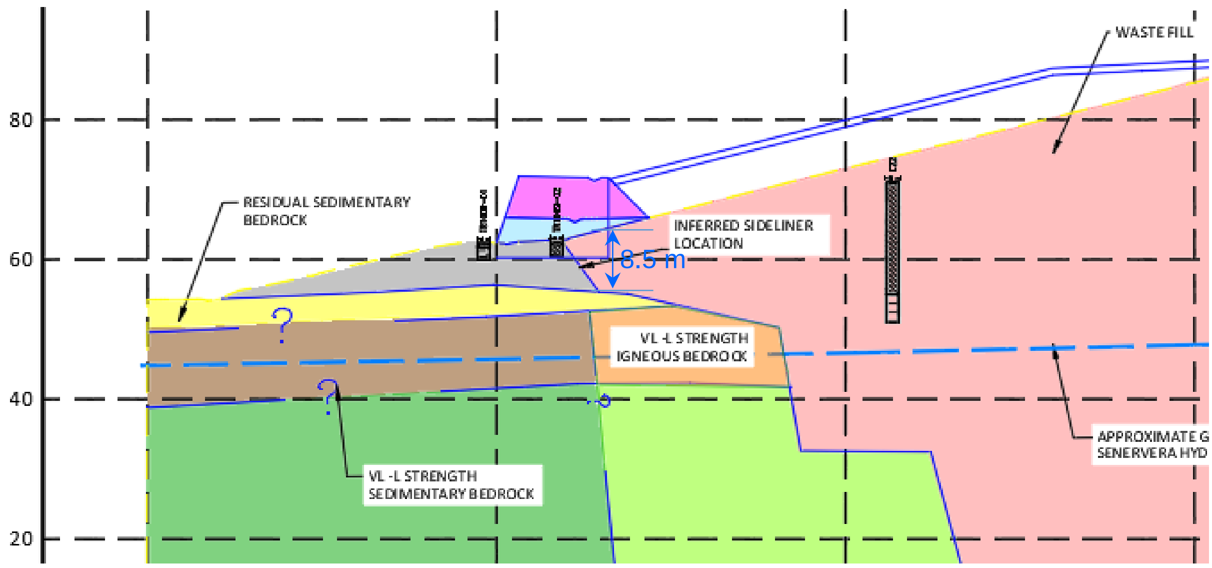


Figure 6: Interpreted Ground Condition at Critical Section (Ch800)

2.1.1 Seismic Refraction Survey

To supplement the provided information and the boreholes, CPTs and test pits undertaken during the site investigation, Seismic Refraction (SR) has also been used on the site to investigate the ground conditions. SR measures the velocity of compression waves (P-waves) and provides continuous 2D subsurface sections. The P-wave velocity (V_P) is directly controlled by the parameters of elasticity (moduli) and density of the subsurface strata. The SR method can yield the subsurface P-wave velocity structure, which is used to help model subsurface stratigraphic characteristics. Where significant change in P-wave velocity occurs (e.g. soil/ rock interface), estimates of the depth to layer interfaces can be made for assessing depth to bedrock and thickness of overburden.

The SR indicates that there are variable ground conditions on site. As landfill waste is likely to be encountered below the MSE wall foundation between Ch650 and Ch850, details of the results of the seismic survey are discussed further below.

A low velocity (<1000 m/s) is generally observed across the site, and this varies between 2 m to >20 m in thickness. Typically, this is underlain by a moderate velocity layer (1000 m/s to 1600 m/s) and also a high velocity layer (>1600 m/s). A correlation between the modelled V_P sections and available borehole data indicates that we can infer that top of rock correlates with an average V_P value of 950 m/s.

Figure 7 shows the seismic refraction survey results obtained near to Ch680. At this cross section, the rock is anticipated at a depth of approximately 5 m below the existing ground level.

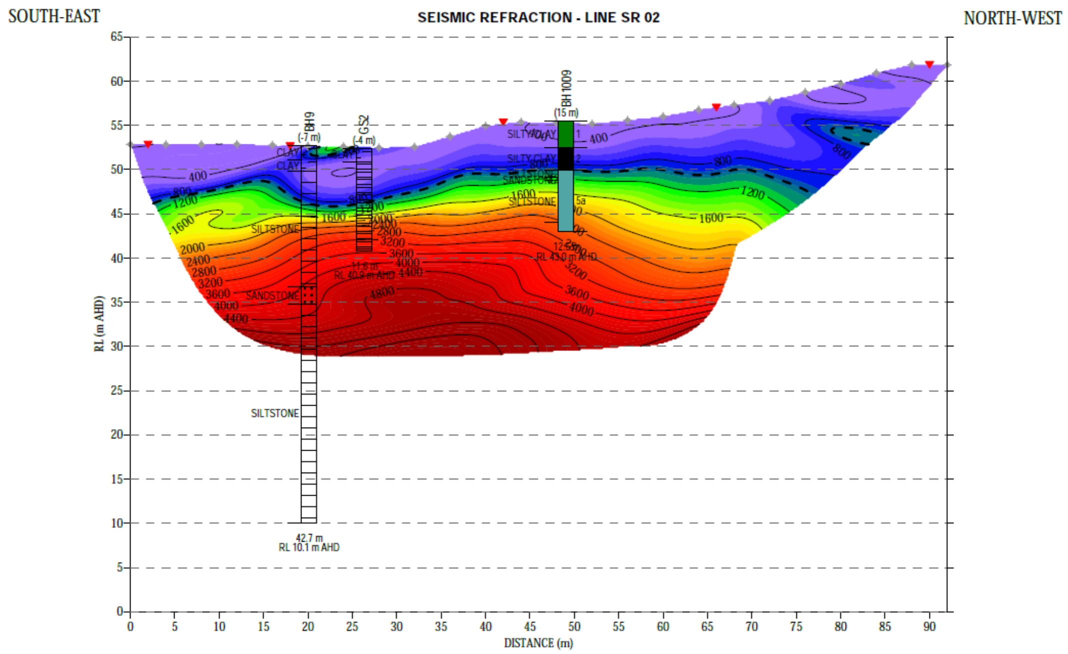


Figure 7: Seismic Survey Results close to Ch680

Figure 8 shows the seismic refraction survey results close to Ch800. At this cross section, the rock is anticipated at a depth varying from approximately 6 m to 8 m below the existing ground level, in the vicinity of the footprint of the proposed wall. The interpreted rock level at these chainages is consistent with geotechnical sections discussed above.

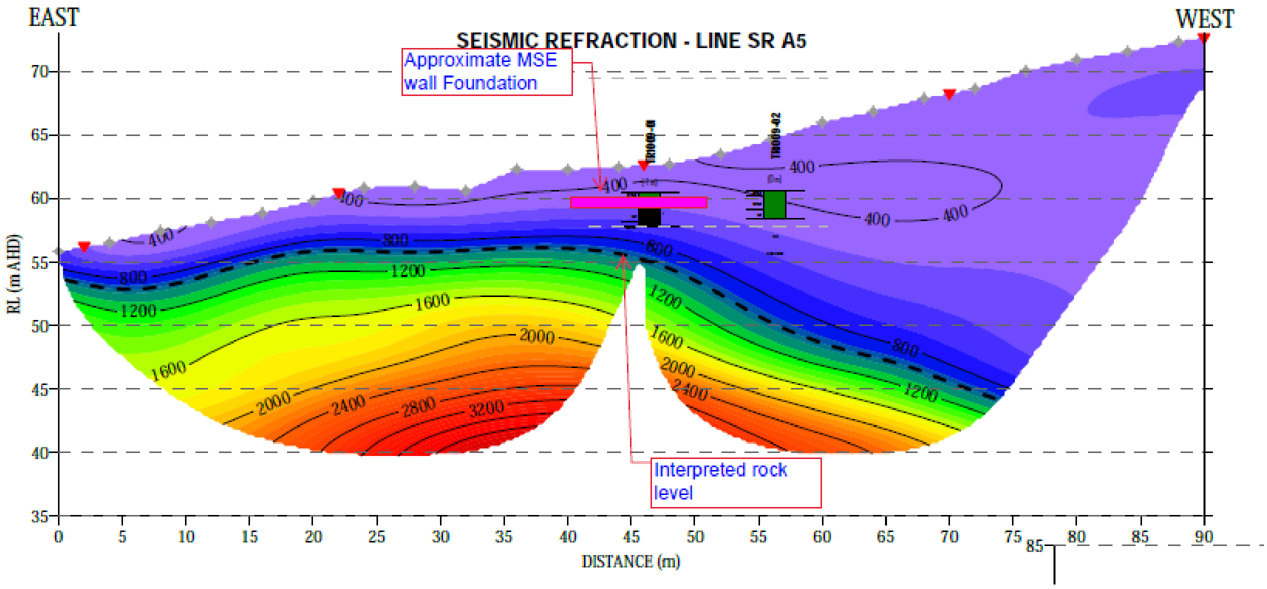


Figure 8: Seismic Survey Results close to Ch800

2.2 Groundwater

The groundwater levels along the MSE wall are basically a subdued reflection of the surface topography, which slopes gently towards South Creek in the west. To the east of the landfill, the standing water levels vary from RL37 mAHD to RL48 mAHD, while to the west and south of the landfill the standing water levels are typically RL37 mAHD to RL39 mAHD.

For the design of the retaining wall, the groundwater was assumed with an elevated level (RL53 mAHD) that is within Unit 2 (Residual Clay). This elevated GWL represents the approximate top of the southeast pond. Generally, the Unit 2 is underlying the controlled fill at RL varying from 48 mAHD to 55 mAHD (approx.) along the wall alignment. We anticipate that ongoing leachate management will continue to influence the groundwater levels in the vicinity of the site.

3.0 DESIGN PARAMETERS

3.1 Geotechnical Parameters

The geotechnical parameters adopted for the design of the retaining wall are derived from test results and previous engineering experiences in similar ground conditions. Table 3 presents a summary of geotechnical properties adopted in the design.

Table 3: Geotechnical Design parameters

Unit		Bulk Unit Weight, (kN/m^3)	Undrained Shear Strength[2], S_u (kPa)	Drained Cohesion, c' (kPa)	Drained Friction Angle, φ' (deg)	Young's modulus, E' (MPa)	Poisson's Ratio
1a & 1b		18	100	2	30	15	0.30
1c	Old	17	75	1	27	-	-
	New	16	75			-	-
2		19	150	5	29	25	0.3
3a		21	-	40	35	125	0.25
3b		23	-	30	40	500	0.2
5		22	-	20	33	75	0.3
Reinforced Fill(1)		20	-	0	32	50	0.3
Liner supporting Fill(1)		20	-	0	32	30	0.3

Note:

1. *Parameters for imported materials may be subject to change, should the design development process result in different materials being considered to be appropriate for the design.*
2. *Undrained parameters were adopted for appropriate materials below the design groundwater level.*

Table 4 presents a summary of consolidation properties for compressible soils expected along the wall alignment. Empirical correlations and previous engineering experience have been used to assist with characterisation of compressibility behaviour. Over-consolidation ratio (OCR) for the units provided in Table 4 have been chosen based on correlations from Cone Penetrometer Tests (CPTs) and previous engineering experience on similar materials in Sydney.

We understand that the old landfill was placed in an uncontrolled manner with the source material being mainly soil waste and some construction waste. The landfill material is considered to undergo ongoing settlements over time. However, its behaviour is different from the traditional 'creep' settlement observed in cohesive soils. The landfill waste is expected to progressively compress over time due to decomposition of materials and potential re-orientation of waste particles within the soil mass.

For analyses of landfill waste, we have assessed that the material can be modelled with long term ‘creep’ defined as logarithmic volume change expressed as a percentage of fill height per log time cycle of 1% (Old landfill) and 2% (new landfill).

Consolidation parameters for volcanic layers (i.e. Unit 3a and 3b) have not been considered on the basis that these materials are likely to have high overconsolidation ratio (OCR) values and low void ratios. These units are not considered to contribute to creep settlement.

Table 4: Preliminary Consolidation Parameters

Unit	Design OCR	Design Compression Ratio, $C_c/(1+e_0)$	Design Recompression Ratio, $C_r/(1+e_0)$	Design Creep Coefficient/Ratio $C\alpha/(1+e_0)$
Unit 1a and 1b	1.5	0.15	0.022	0.008
Unit 1c – New[1]	1.0	0.4	0.10	0.02
Unit 1c – Existing[1]	1.2	0.2	0.05	0.01
Unit 2	3.0	0.1	0.015	0.005

Note:

1. *Parameters for new and existing landfill may be subject to change in detailed design based on performance of the existing landfill and its properties.*

3.2 SHANSEP Parameters

As part of the analysis, we have considered the development of strength in the foundation soils as the proposed MSE wall is constructed. Stress History and Normalised Soil Engineering properties (SHANSEP) are established based on the history of the soil deposit. Based on this concept, the undrained shear strength, S_u of the clayey soil (residual soil underneath the groundwater level) is established as below.

$$S_u = \alpha^* \sigma'_v * (OCR)^m$$

Where,

α and m- Soil parameters

OCR- Over-consolidation ratio

σ'_v- Vertical effective stress

As recommended by Mesri (1975), for Clayey soil (PI>20%), α = 0.22 and m = 0.95 (near unity) are adopted to estimate the undrained shear strength.

3.3 Reinforcement

Tensile reinforcement are required to enhance the stability of the MSE wall. Reinforcement is required both for the internal stability of the wall and the global stability. Uniaxial geogrid has been considered with the ultimate tensile strength (T_{ult}). An example geogrid product has been considered for the preliminary design of the MSE wall. Details for the example product used in the analysis are provided in **APPENDIX B**. Further product specifications and requirements for installation and testing will be provided in a detailed project technical specification.

The available long-term strength of the geogrid reinforcement (T_{al}) is assessed as below.

$$T_{al} = T_{ult} / (RF_{ID} * RF_{CR} * RF_{D})$$

Where the following factors are applied for the example product. Reduction factors commensurate with other approved products should be adopted when alternative design measures are adopted:

- RF_{ID} - Reduction factor for installation damage = 1.10.
- RF_{CR} - Reduction factor for creep rupture = 1.43
- RF_{D} - Reduction factor for chemical/biological degradation = 1.05. The RF_{D} was increased to 1.25 for the reinforcement which is placed within liner supporting fill considering potential exposure to leachate as a contingency for unanticipated leakage through the liner system.

The axial strain of the geogrid is assessed by using manufacturer data regarding the strain at various levels of tensile load, presented for loading durations representative of end-of-construction and end-of-design-life. For the example product, such data indicates that strain levels may be acceptable for applied tensile loads up to approximately 60% of ultimate tensile strength.

4.0 DESIGN METHODOLOGY

The Load and Resistance Factor Design (LFRD) approach was adopted in the design of MSE wall as described in Federal Highway Administration FHWA GEC 011 – Volume I (FHWA GEC 011). This approach adopts load factors greater than 1.0 for the estimation of design loads. The design resistance is determined by multiplying the resistance by a factor typically less than 1.0.

4.1 Loads

4.1.1 Self-weight (EV)

The weight of the MSE wall is estimated based on its geometry and unit weight of the compacted reinforced fill.

4.1.2 Lateral Earth Pressure (EH)

Lateral earth pressure is developed on the back face of the MSE wall due to the landfill. The earth pressure is estimated based on the Coulomb earth pressure theory. The active earth pressure coefficient is estimated as a function of slope of back fill (β), angle of friction between the retained soil and reinforced soil (δ), effective friction angle of retained soil (ϕ_b) and the angle of back face of the MSE wall (θ).

4.1.3 Live Load (LL)

AS4678 outlines that "*In the calculation of traffic surcharge, the unfactored value has to be 20 kPa for roads of functional road classes 1, 2, 3, 6 or 7 (see HB77)*. For all other functional class roads or temporary roads (e.g. ramps) the unfactored traffic loading has to be 10 kPa."

For the design of MSE wall, 20 kPa of traffic surcharge has been adopted for long term condition and 10 kPa has been adopted for short term (construction stage).

4.1.4 Earthquake Load (EQ)

An acceleration coefficient (a) of 0.08 is considered for Sydney in accordance with AS1170.4 for an earthquake event with 1 in 500 years return period and site factor of 1.0 as per AS4678 – 2002 Table I2.

As per AS4678 cl.I13, the horizontal coefficient of acceleration is estimated as below:

$$a_h = 0.5*a = 0.04$$

AS4678 outlines that "*Dynamic numerical analysis for reinforced soil walls with metallic reinforcement and granular backfill have indicated amplification of motions within both the structure and the retained soil*". The amplification is estimated as below.

$$a_{mh} = (1.45 – a_h) * a_h = 0.056$$

Even though the amplification is for metallic reinforcement, the AS4678 further states that "*Although the above amplification has been developed for a particular wall type, it may be useful as a first approximation for other forms of reinforcement and other retaining structures in the absence of other information.*"

So, for the assessment global stability, we have adopted a_{mh} = 0.0564. A multiplier 0.5 was adopted on the horizontal acceleration coefficient for the pseudo-static stability analysis in accordance with AS4678 "*For larger structures such as reinforced earth walls, it is common to take 0.5 of the wall inertia effects in recognition of the likelihood that acceleration of the backfill and the wall may not be exactly in phase.*"

$$a_{mh} = 0.0564 * 0.5 = 0.027$$

Where traffic load or other live load is directly applied on the retaining structure, the partial factors for live load should be taken as 0.5 (10 kPa).

4.1.5 Vehicle Impact load (CT)

The traffic barriers are to be installed at the top of the MSE wall. The vehicular impact on the barrier will induce additional load on the MSE wall which is expected to affect only the internal stability of the wall. The impact load (dynamic load) is considered as static impact load for the design.

As detailed in FHWA GEC 011, the static impact load is considered acting on the upper two layers of reinforcements. The top layer of the layer is designed with static impact load of 33.5 kN/m and second layer with 8.8 kN/m.

4.2 Load Combinations

The design of the MSE wall has been carried out considering a few combinations of the above loads that the MSE wall will experience during construction and its operation. For the design of the MSE wall, the following combinations with appropriate load factors have been considered. The load factors are provided in the next section below.

- Load Case 1: “EV” + “EH” + “LL” (Strength)
- Load Case 2: “EV” + “EH” + “LL” + “EQ” (Earthquake)
- Load Case 3: “EV” + “EH” + “LL” (construction)
- Load Case 4: “EV” + “EH” + “LL” + “CT” (collision)
- Load Case 5: EV” + “EH” + “LL” (Serviceability)

4.3 Load Factors

In the LFRD method, the load factors have been adopted as provided in Table 5.

Table 5: Load factors adopted for each load case

Load case	Load				
	EV (max/min)[1]	EH (max/min)[1]	LL	EQ	CT
1	1.35/1.00	1.50/0.90	1.75	-	-
2	1.00	1.00	1.00	1.00	-
3	1.35/1.00	1.50/0.90	1.75	-	-
4	1.0	1.00	0.50	-	1.00
5	1.0	1.0	1.0	-	-

Note

1. *The minimum value was applied to the load combination where the corresponding load reduces the force effect.*

4.4 Resistance factors

Resistance factors for the external stability analyses are provided below.

Table 6: Resistance factor adopted for the external stability

Mode of failure	Value
Bearing resistance	0.65

Mode of failure	Value
Sliding	1.00
Overall stability[1,2,3,4,5]	0.65
	0.75
	0.90

Note

1. *Where geotechnical parameters are defined based on limited information.*
2. *The resistance factor of 0.65 is approximately equivalent to the safety factors of 1.5. This safety factor was adopted for short-term and long-term loading except the EQ loading.*
3. *The resistance factor of 0.75 is approximately equivalent to the safety factors of 1.35. This safety factor was adopted for Short-term (temporary) loading including construction.*
4. *The resistance factor of 0.91 is approximately equivalent to the safety factors of 1.1. This safety factor was adopted for rapid loading including i.e. earthquake.*
5. *Overall stability of MSE wall was carried out using the working stress design approach.*

The resistance factor for the internal stability of the MSE walls is provided in Table 7.

Table 7: Resistance factor adopted for the internal stability

Load Type (Load Cases)	Failure Mechanisms	
	Tensile	Pull-out
Static loading (Load Cases 1, 3, 5)	0.65	0.90
Combined Static/EQ (Load Case 2)	1.20	1.20
Combined Static/Collision (Load Case 4)	1.20	1.00

4.5 Design Assumptions

The design of MSE wall has assumed the following:

- The ground condition below the MSE wall and Liner supporting fill is interpreted based on the existing geotechnical information.
- The Geotechnical parameters are interpreted based on available field testing.

5.0 ANALYSIS

5.1 External Stability

For the MSE walls, four potential external failure mechanisms are considered as follows.

- Sliding at the foundation.
- Overturning.
- Bearing resistance.
- Overall/global stability (addressed in Section 5.3).

External stability analysis has been carried out using a worksheet prepared in Mathcad. The sample design calculations for external stability are provided in **APPENDIX C** at Ch200 and Ch600. As detailed in FHWA GEC 011, a minimum reinforcement length as shown in Table 8 was initially adopted for the external stability of the MSE walls. It is noted that longer lengths are required for the MSE wall where foundation conditions affect the overall stability. The requirement of reinforcement length considering the overall stability is addressed in Section 5.3.

The analysis results show that the proposed dimensions of the wall are satisfactory for the sliding, overturning and bearing.

Table 8: External Stability Check except Overall Stability

Ch	h (m)	H (m)	d (m)	w (m) (Note 1)	λ (Note 2)	Sliding Check	Overturning Check	Bearing Check
100	4.77	5.97	1.20	11	0.7	OK	OK	OK
200	11.83	14.83	3.00	11	0.7			
300	12.37	15.37	3.00	11	0.7			
400	15.29	17.09	1.80	12.5	0.7			
500	15.43	17.23	1.80	12.5	0.7			
600	17.59	19.69	2.10	12.5	0.75			
700	17.02	19.12	2.10	12.5	0.75			
800	9.56	10.76	1.20	12.5	0.7			
850	6.10	7.30	1.20	11.44	0.7			

Note

1. *Top width of the reinforced MSE wall is estimated as w = W – width of drainage chimney (taken as 1.0m)*
2. *Minimum ratio between reinforcement length (L) and wall height (H) as per FHWA – GEC 011 to satisfy external stability.*

5.2 Internal Stability

Internal failure of the MSE wall can occur in two different mechanisms as below:

- Tensile failure of reinforcement.
- Pullout failure of reinforcement.

Internal stability analysis has been carried out using a worksheet in Mathcad and Excel spreadsheets calculations for the strength and serviceability. **APPENDIX D** provides the sample calculations on tensile and pullout failure of reinforcements within the MSE wall at Ch200 and Ch600.

As detailed in FHWA GEC 011, a minimum reinforcement length of 0.7H was initially adopted for the internal design of the MSE walls.

To control long-term deformation, the geogrid layout was designed to limit post-construction strains to acceptable levels.

Table 9 shows the proposed reinforcement including spacing, strength, minimum reinforcement length and number of reinforcement layers. The internal stability check indicates that the proposed reinforcement is adequate for both pull-out and tensile failures. It is noted that longer lengths are required for the MSE wall where foundation conditions affect overall stability.

Table 9: Internal Stability Check

Ch	h (m)	H (m)	Location	Reinforcement Details (Note 1)				Tensile Check	Pull-Out Check
				T_{al} (kN/m)	S_v (m)	L (m)	n		
100	4.77	5.97	Bottom	72	0.6	4.2	3	OK	OK
			Middle	72	0.6	4.2	3		
			Top	72	0.6	5.7	3		
200	11.83	14.83	Bottom	121	0.3	10.4	8	OK	OK
			Middle	121	0.6	10.4	10		
			Top	72	0.6	10.5	10		
300	12.37	15.37	Bottom	121	0.3	10.8	8	OK	OK
			Middle	121	0.6	10.8	10		
			Top	72	0.6	10.8	10		
400	15.29	17.09	Bottom	121	0.3	12.0	14	OK	OK
			Middle	121	0.6	12.0	10		
			Top	72	0.6	12.0	10		
500	15.43	17.23	Bottom	121	0.3	12.1	14	OK	OK
			Middle	121	0.6	12.1	10		
			Top	72	0.6	12.1	11		

Ch	h (m)	H (m)	Location	Reinforcement Details (Note 1)				Tensile Check	Pull-Out Check
				T_{al} (kN/m)	S_v (m)	L (m)	n		
600	17.59	19.69	Bottom	121	0.3	14.8	22	OK	OK
			Middle	121	0.6	14.8	10		
			Top	72	0.6	12.5	11		
700	17.02	19.12	Bottom	121	0.3	14.4	22	OK	OK
			Middle	121	0.6	14.4	10		
			Top	72	0.6	12.5	10		
800	9.56	10.76	Bottom	121	0.6	7.5	6	OK	OK
			Middle	72	0.6	7.5	6		
			Top	72	0.6	7.5	5		
850	6.10	7.30	Bottom	72	0.6	5.1	4	OK	OK
			Middle	72	0.6	5.1	4		
			Top	72	0.6	5.8	3		

Notes:

1. *T_{al} – Long term reinforcement strength, S_v – Maximum vertical spacing of the reinforcement, L – Minimum length of reinforcement, n – No. of reinforcement*
2. *Minimum reinforcement length (L) was assessed as per FHWA – GEC 011 to satisfy the internal stability.*

5.3 Overall Stability

5.3.1 Analysis Cases

Overall stability is assessed at different sections for the following load cases based on the preliminary geometry of the MSE walls. The targeted minimum factor of safety is provided for the cases considering long term and short-term conditions as below.

- Case 1: End of MSE wall construction (the wall is built to its full height, new landfill is not placed, construction surcharge of 10 kPa on existing landfill slope and the retaining wall) – SHANSEP parameters for the soil (residual soil) below the groundwater level (GWL) and drained parameters (Mohr-Coulomb) for soil above the GWL. Targeted minimum FoS = 1.35.
- Case 2: End of landfill construction (the wall is built to its full height, the landfill is placed at its maximum level, capping is installed, and a surcharge of 10 kPa is applied along the landfill slope and top of the retaining wall expecting construction traffic) – SHANSEP parameters for the soil (residual soil) below the GWL and drained parameters for soil above the GWL. Targeted minimum FoS = 1.35.

- Case 3: Long term with undrained condition (the wall is built to its full height, the landfill is placed to its maximum level, capping is completed, and traffic surcharge of 20 kPa is applied on top of the retaining wall) – SHANSEP parameters for the soil below the GWL (with strength gain/loss) and drained parameters for soil above GWL. Targeted minimum FoS = 1.50.
- Case 4: Long term with drained condition (the wall is built to its full height, the landfill is placed to its maximum level, capping is completed, and traffic surcharge of 20 kPa is applied on top of the retaining wall) – Drained parameters for the soils below and above the GWL. Targeted minimum FoS = 1.50.
- Case 5: Long term extreme GWL - Drained parameters for the soil below and above GWL (the wall is built to its full height, the landfill is placed to its maximum level, capping is completed, and traffic surcharge of 10 kPa is applied on top of the retaining wall with an elevated groundwater level) . The GWL is assumed to be at RL53 mAHD which represents the top of southeast pond. Targeted minimum FoS = 1.35.
- Case 6: Earthquake (the wall is built to its full height, the landfill is placed to its maximum level, capping is completed, and traffic surcharge of 10 kPa is applied on the top of retaining wall with an earthquake loading) - SHANSEP parameters for the soil below WT (with strength gain/loss) and drained parameters for soil above WT. Targeted minimum FoS = 1.1.

Cases 1 and 2 are assumed as rapid installation of the MSE wall and rapid placement of landfill, the short-term undrained strength is estimated based on the OCR in Table 3. For the estimation of effective vertical stress (σ'_v), the average depth of the residual soil from existing ground level has been used. In these cases, the SlopeW model has been set such that MSE wall and Landfill above existing ground will increase the pore-pressure equal to the MSE wall and Landfill weight. The effective stress will remain the same (undrained shear strength is not changed).

Cases 3 and 6, with time, the excess pore resulting from the MSE wall and landfill placement will dissipate. The effective vertical stress will increase, and OCR will reduce. The undrained strength is estimated based on the increase of effective vertical stress (σ'_v) with the consideration of the reduction in OCR as detailed in Table 4.

5.3.2 Length of Reinforcement

The length of reinforcement has been optimised considering the overall stability of the MSE wall as below.

- Model the MSE wall with geometry detailed in Table 1 and interpreted ground condition.
- Model the reinforcement within the MSE wall (reinforcement length = ~ 0.7H and spacing and strength as provided in Table 9).
- Evaluate Factor of Safety (FoS) for the overall stability of the MSE wall for the cases detailed in Section 5.3.1.
- If the FoS is less than detailed in Table 6 and critical failure surface is passing through the MSE wall and liner supporting fill, increase the reinforcement length within the MSE wall and introduce the reinforcement within the liner supporting fill until achieving the adequate FoS.
- The overall stability of the liner supporting fill will be further assessed in Section 6.0 considering its overall stability in isloation. If required, the reinforcement length within the MSE wall will be further revised.

For example, Figure 9 below shows the FoS (FoS = 1.26) of the wall at Ch650 in long-term with the minimum reinforcement length of 0.7H (~14.5m). This result indicates that the MSE wall does not have adequate FoS with this length of reinforcement.

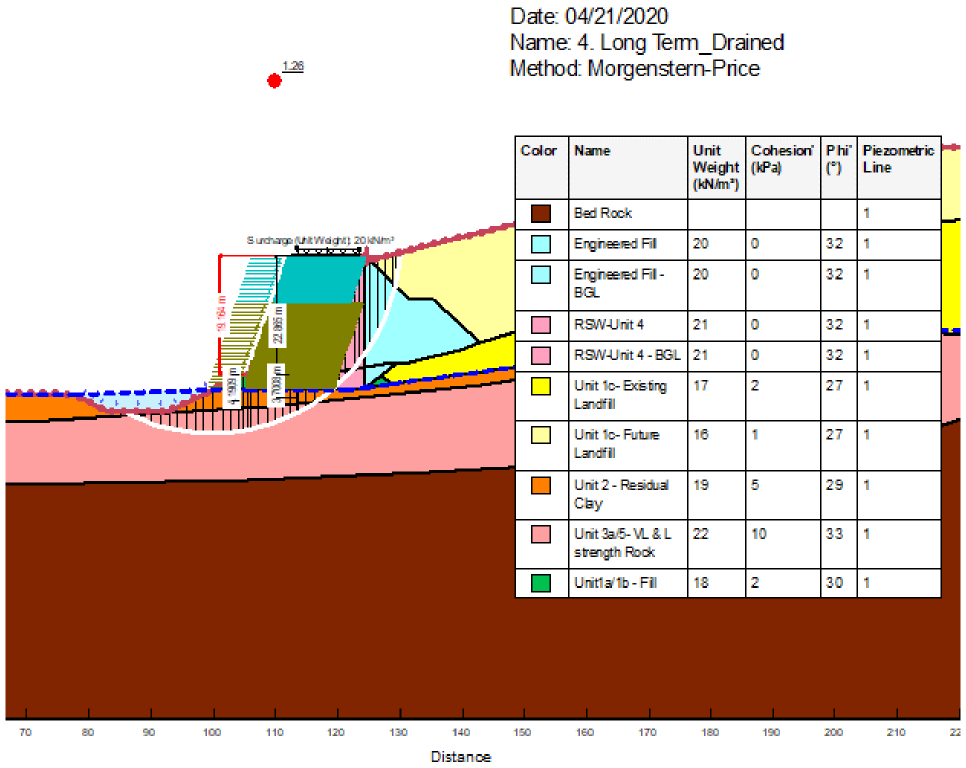


Figure 9: FoS of the MSE wall (Ch 650m) in Long Term with Reinforcement of 0.7H in Length

Figure 10 below shows the FoS (1.39) of the wall at Ch650 in long-term with the fully reinforced MSE wall (reinforcement is installed to entire width of wall). This result indicates FoS has been improved by the increase of reinforcement length, but FoS is still not adequate to meet the design requirement.

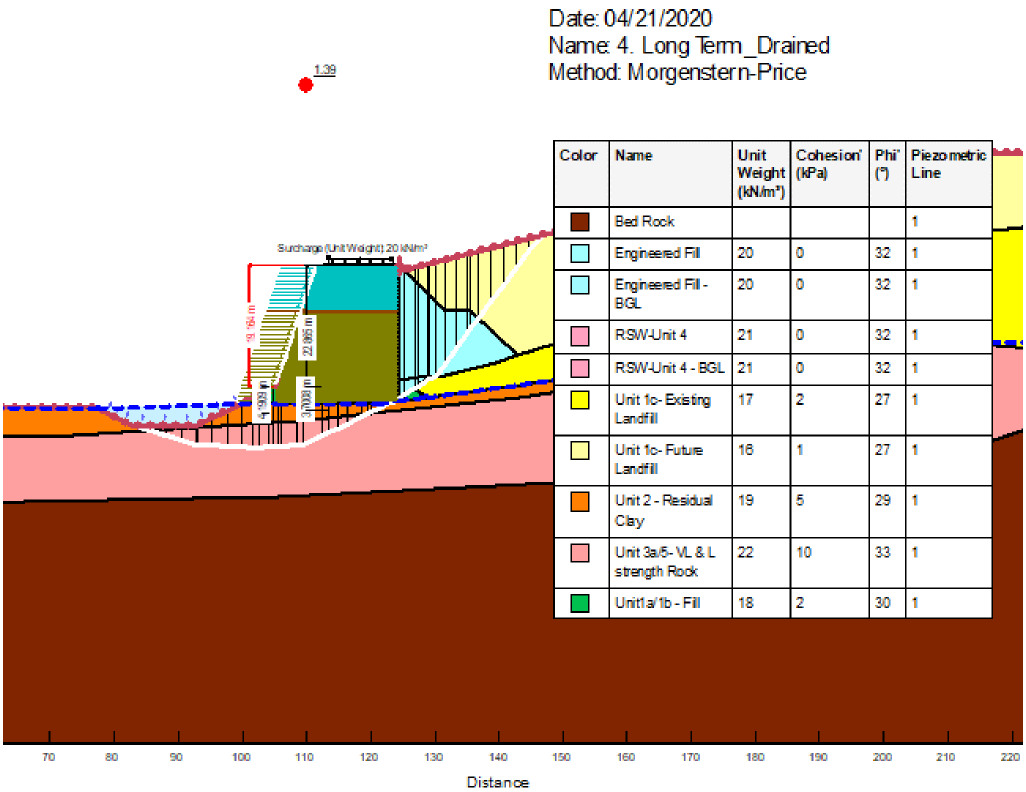


Figure 10: FoS of the MSE wall (Ch 650m) in Long Term with Reinforcement to Entire Wall Width

In order to meet the overall stability requirement, the additional reinforcements have been introduced within the liner supporting fill. The details of the reinforcement are addressed in Section 6.0.

5.3.3 Analysis Outcome

A summary of stability assessment results is presented in Table 10 and **APPENDIX E** for long-term, short-term and earthquake conditions. Based on the analysed preliminary geometry of the critical sections, the results indicate that the MSE wall meets the target minimum factors of safety for overall stability.

Wall height outside of the chainage is smaller than the critical section reported in Table 10. We consider that FoS against the overall stability of the wall outside of the chainages will be adequate. Additional stability analyses at a few sections (outside of the changes) will be carried out at detailed design stage to demonstrate the adequacy of FoS.

Table 10: Summary of Overall Factor of Safety

Cross section	Factor of Safety					
	Case 1 (1.35)	Case 2 (1.35)	Case 3 (1.50)	Case 4 (1.50)	Case 5 (1.35)	Case 6 (1.10)
CS4 (Ch250)	2.46	1.98	1.98	2.02	1.90	1.72
CS5 (Ch375)	1.83	1.66	1.65	1.68	1.68	1.49
CS6 (Ch550)	2.21	1.79	1.76	1.98	1.98	1.62
CS7 (Ch650)	2.10	1.58	1.59	1.65	1.65	1.43

Cross section	Factor of Safety					
	Case 1 (1.35)	Case 2 (1.35)	Case 3 (1.50)	Case 4 (1.50)	Case 5 (1.35)	Case 6 (1.10)
CS8 (Ch750)	2.00	1.78	1.91	1.65	1.64	1.75

Note

1. *The factor provided in the bracket is the targeted minimum FoS.*

5.4 Wall Movement

Settlement analyses of the MSE walls have been carried out using the finite element program PLAXIS2D. PLAXIS2D is a commercial two-dimensional finite element software used in calculations of stresses and displacements for a wide range of geotechnical, civil engineering and mining problems.

The purpose of the analyses has been to estimate the settlement and lateral movement of the MSE wall during its construction and operation.

For this assessment, the reinforcement has been modelled as elastic and strain at the available load (P_{al}) is assumed as 10%. The available load has been estimated as below.

For short term loading, the P_{al} was estimated considering the appropriate reduction factors for construction damage and environmental effects.

For long term loading, the P_{al} was estimated considering the reduction factors for construction damage, environmental effects, and long-term creep.

The MSE wall is installed along the existing quarry slope and the wall may experience differential settlement at the foundation level which may induce additional load in the reinforcement. The analyses can be used to estimate such additional loading.

5.4.1 Construction sequences:

- Stage 1: Set Initial stress conditions using K_o method including the old landfill
- Stage 2: Consolidate the old landfill and other layers (time interval: five years)
- Stage 3: Excavate the footprint of the MSE wall for up to foundation depth prior to foundation construction
- Stage 4: Install MSE wall gradually (0.3 m of layer thickness and simultaneously install reinforcement)
- Stage 5: Reset the displacement to zero. Place the new landfill in three layers in three different stages
- Stage 6: Apply the 10 kPa load on the MSE wall
- Stage 7: Consolidate the landfill for 100 years

It is noted that at the start of Stage 5, the displacement of the model was reset to zero to simulate the post construction shape and geometry of the MSE wall prior placing the new landfill. The field stresses in the surrounding areas will remain unchanged. The displacement output results at the end of Stage 7, present the impact of the proposed new landfill on the vertical and lateral movement of the MSE wall after 100 years.

5.4.2 Results

Figures F.1 to F.18 in **APPENDIX F** present graphical representations of the geotechnical model, induced horizontal and vertical movements during construction and at 100 years, estimated using finite element analyses. The preliminary deformation analysis has been undertaken as a first past assessment of the potential movements that may occur during the design life of the structure. We consider that the movements are likely to significantly overestimate post-construction waste settlement in some areas. This is considered suitable for preliminary assessment of settlement effects on the retaining wall structure, however the preliminary analysis results are not considered suitable for assessment of post-construction deformation of the waste landform.

The displacement output results present the impact of the proposed new landfill on the vertical and lateral movement of the MSE wall after 100 years.

Table 11 provides the predicted vertical and horizontal displacements at the level of wall foundation. The level of wall foundation is expected to move laterally during construction from 45 mm to 105 mm and settle from 95 mm to 535 mm. Lateral movement at the wall foundation level is expected to be in the range of 15 mm to 135 mm and settlement is expected to vary from 40 mm to 230 mm during its design life. It should be noted that the lateral and vertical movement of the MSE wall at the end of design life (100 years) does not include the movement induced during construction.

CS3A is located at approximately at Ch680 where landfill is expected to be below the toe of the MSE wall. Table 11 shows the expected maximum settlement at the foundation level of the MSE wall after 100 years is about 230 mm. If the existing landfill below the footprint of the MSE wall is excavated and replaced with MSE wall type material, the settlement after 100 years is expected to reduce to 40 mm.

Table 11: Assessed vertical and horizontal displacement at foundation level

Section	End of Construction		End of Design Life (100 Year)	
	Max. horizontal movement (mm)	Max. vertical settlement (mm)	Max. horizontal movement (mm)	Max. vertical settlement (mm)
CS3 (Ch200)	105	165	15	50
CS7 (Ch650)	65	95	65	45
CS3A (Ch680)	55	535	135	230
CS3A (Ch680)[1]	45	110	60	40

1. *Assuming old landfill beneath MSE wall will is replaced with reinforced soil material*

Table 12 provides the predicted vertical and horizontal displacements at the top of wall foundation. Top of the wall is expected to move laterally during construction from 165 mm to 375 mm and settle from 225 mm to 620 mm. It is also expected to deform laterally during its design life from 20 mm to 230 mm and settle from 55 mm to 340 mm. It should be noted that the lateral and vertical movement of the MSE wall at the end of design life (100 years) does not include the movement induced during construction.

Table 12 shows the expected maximum post construction settlement at top of the MSE wall after 100 years is about 340 mm. If the existing landfill below the footprint of the MSE wall is excavated and replaced with MSE wall type material, the settlement after 100 years is expected to reduce to 120 mm. We note that these magnitudes of movement are within the tolerable for these types of structures.

Table 12: Assessed vertical and horizontal displacement at top of the wall

Section	End of Construction		End of Design Life (100 Year) (post construction movement)	
	Max. horizontal movement (mm)	Max. vertical settlement (mm)	Max. horizontal movement (mm)	Max. vertical settlement (mm)
CS3 (Ch200)	160	225	30	55
CS7 (Ch650)	310	520	20	115
CS3A (Ch680)	375	620	230	340
CS3A (Ch680)[1]	375	530	55	120

Note

1. *Assuming old landfill beneath MSE wall will is replaced with reinforced soil material*

5.4.3 Settlement Induced Load on Reinforcement

As foundation experiences the differential settlement at the foundation level, this settlement would likely increase the load on the reinforcement. The increase in the load was estimated as below.

- Model the MSE wall with ground condition interpreted (Actual Model).
- Estimate the load on the reinforcement. This load includes the load induced by internal stability and ground settlement.
- Model the MSE wall with a rigid foundation material (Base Model) to eliminate the settlement induced load on reinforcement.
- Estimate the load on the reinforcement. This load includes the load induced by internal stability.
- Estimate the difference between reinforcement loads. This will result in the reinforcement load induced by the ground settlement.

Table 13: Expected induced load on reinforcements due to settlement at the End of Design Life

Section	Geogrid Depth (m)	Geogrid Load (kN/m)		
		Actual Model	Base Model[1]	% Increment
CS3 (Ch200)	3.9 m	9.3	9.2	1%
	4.5 m	13.9	13.8	1%
	10.2 m	21.0	20.0	5%
CS7 (Ch650)	6.6 m	20.6	22.0	-
	7.2 m	28.3	29.2	-
	18.9 m	22.0	27.0	-

Note

1. *Base model is a case where uncontrolled fill (Unit 1) and residual (Unit 2) under the footprint of the MSE wall is replaced with Unit 3a.*

The assessment indicates that reinforcement load is expected to increase by up to 5%. For internal design, 5% increase in reinforcement load has been considered in all reinforcement layers.

6.0 LINER SUPPORT FILL

6.1 Primary Reinforcement

The liner support fill is to be installed behind the MSE wall to support the landfill liner system. The liner support fill is to be constructed at a 1H:1V external slope with a maximum individual batter height of 8 m. Assessment of overall stability detailed in Section 5.2 indicates that reinforcement within the liner support fill is required to provide adequate Factor of Safety for overall stability of the MSE wall between approximately Ch520 and Ch700 where the wall height, H is approximately 20 m.

Figure 11 below shows the FoS (FoS = 1.54) of the wall at Ch650 in long-term with the reinforcement within entire MSE wall and with the reinforcement with liner supporting fill. This result indicates that the MSE wall have adequate FoS with this reinforcement arrangement.

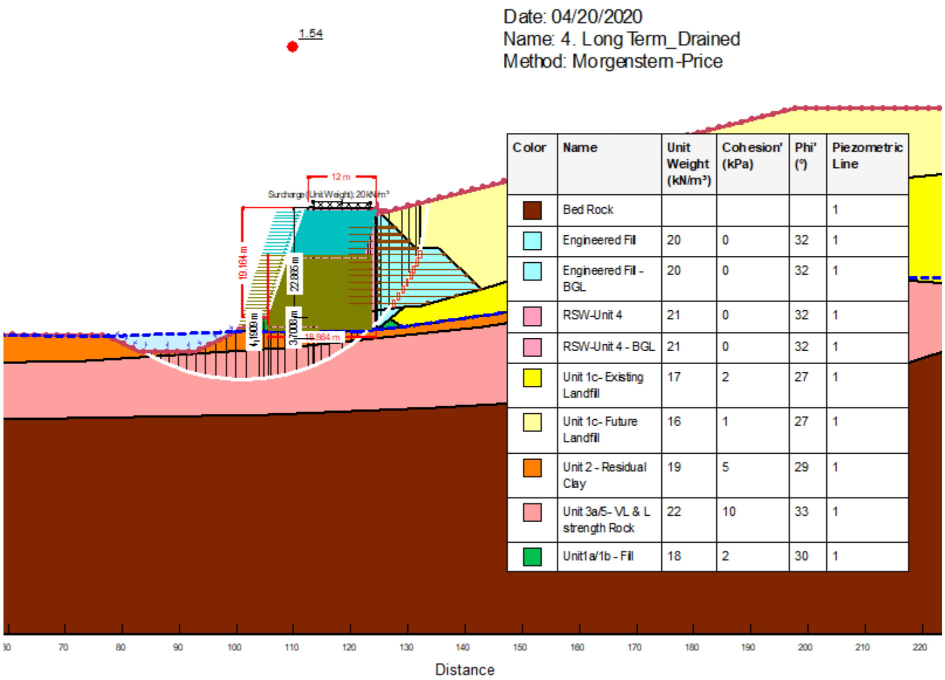
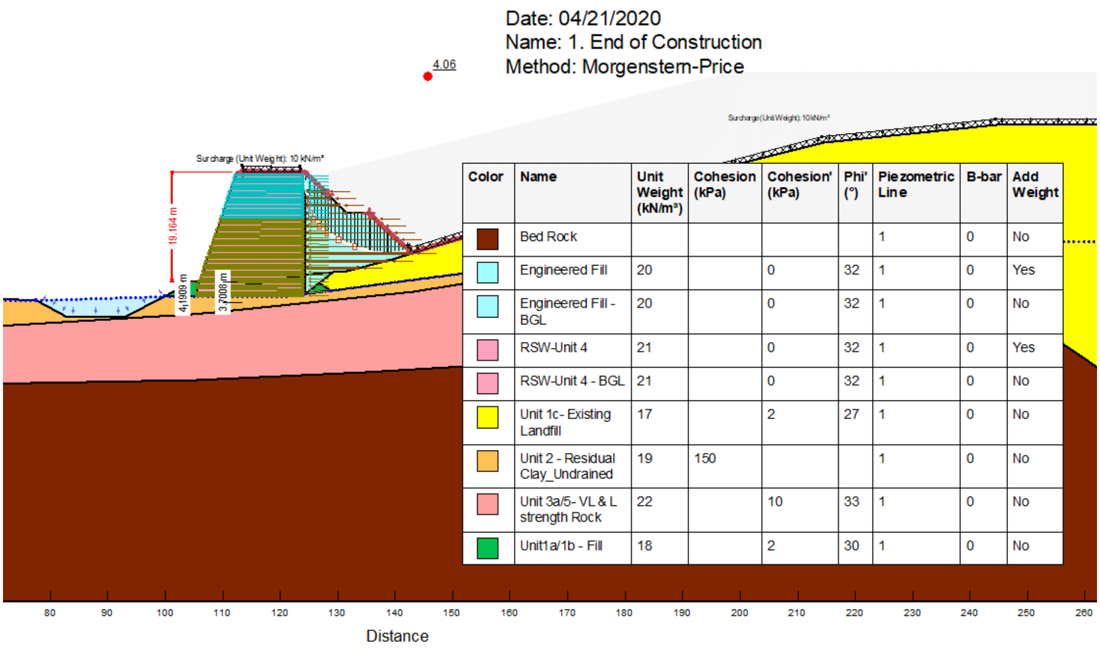


Figure 11: FoS of the MSE wall (Ch 650m) with reinforcement with Entire Wall Width and Liner Supporting Fill

Figure 11 below shows the FoS (FoS = 4.06) of the liner supporting fill at Ch650 at the end of construction. This result indicates that the liner supporting fill has also adequate FoS with this reinforcement arrangement.



Color	Name	Unit Weight (kN/m³)	Cohesion (kPa)	Cohesion' (kPa)	Phi' (°)	Piezometric Line	B-bar	Add Weight
	Bed Rock					1	0	No
	Engineered Fill	20		0	32	1	0	Yes
	Engineered Fill - BGL	20		0	32	1	0	No
	RSW-Unit 4	21		0	32	1	0	Yes
	RSW-Unit 4 - BGL	21		0	32	1	0	No
	Unit 1c- Existing Landfill	17		2	27	1	0	No
	Unit 2 - Residual Clay_Undrained	19	150			1	0	No
	Unit 3a/5- VL & L strength Rock	22		10	33	1	0	No
	Unit1a/1b - Fill	18		2	30	1	0	No

Figure 12: FoS of Liner Supporting Fill (Ch 650m) at End of Construction

The minimum reinforcement within the liner support fill between Ch520 and Ch700 is proposed below and shown in Figure 13.

- Bottom bench – geogrid reinforcement with ultimate tensile strength 200 kN/m at 1200 mm vertical spacing
- Top bench – geogrid reinforcement with ultimate tensile strength 80 kN/m at 1200 mm vertical spacing. Minimum length of reinforcement is 4.0 m.
- The reinforcement within the liner support fill will be separate from reinforcement within reinforced fill (not continuous) as the liner support fill is expected to settle more than the MSE wall.

This is referred to as primary reinforcement for the liner support fill. The extent of primary reinforcement within liner support fill between Ch520 and Ch700 has been assessed for the design assuming the geotechnical properties of the liner support fill (similar properties as reinforced fill) as detailed in Table 3.

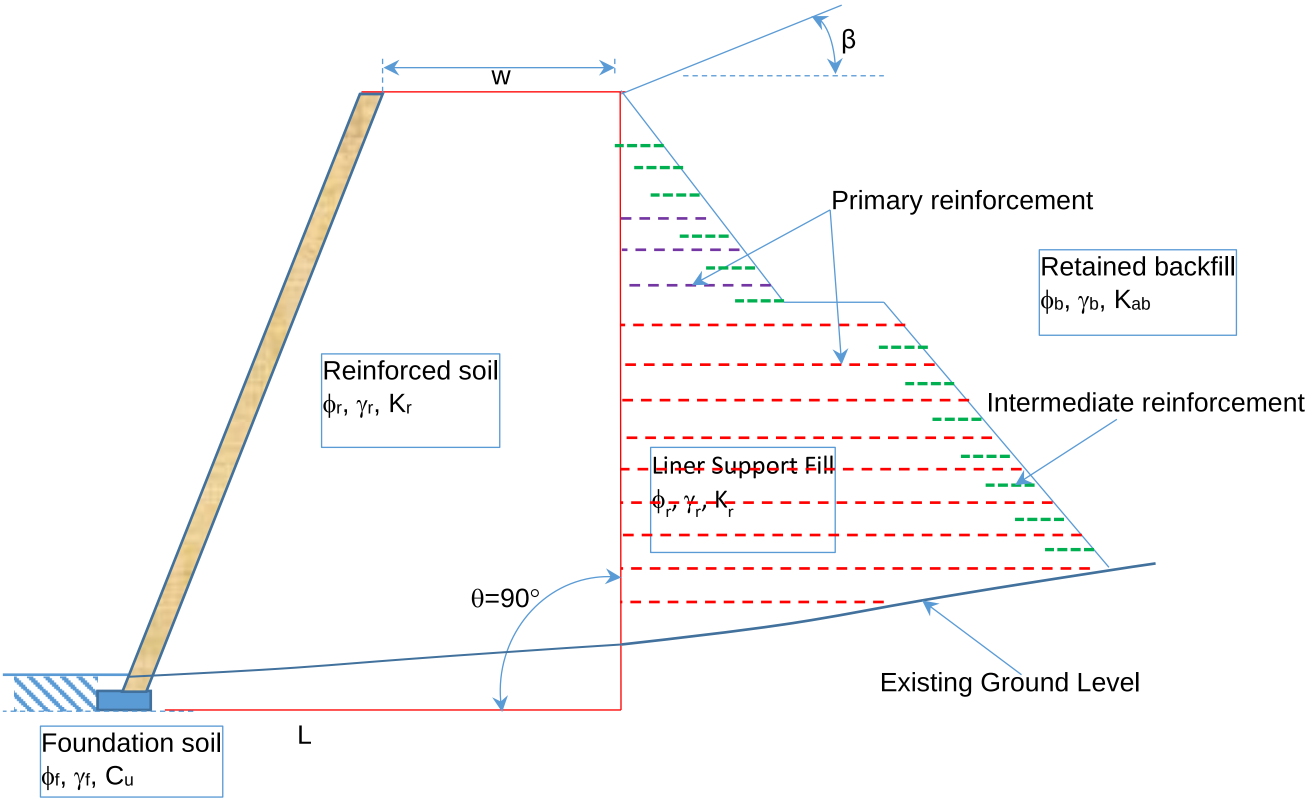


Figure 13: Typical Arrangement of Geogrid within Liner Support Fill

6.2 Intermediate Reinforcement

The liner support fill at all wall chainages will include intermediate reinforcement layers at 300 mm vertical spacing, and generally 1.5 m long, in order to provide local support to allow 1H:1V batter construction and batter surface preparation for liner material placement. This is a temporary stability requirement. The intermediate reinforcement will comprise relatively low strength biaxial geogrid.

7.0 DESIGN OUTCOMES

7.1 Reinforcement Details

Table 14 shows the details of reinforcement within MSE wall and liner supporting fill considering the internal stability, external stability and overall stability. This reinforcement requirement considered the long-term and short-term stability of the the stability of MSE wall and liner supporting fill.

Table 14: Details of the Reinforcement within the MSE Wall and Liner Supporting Fill

H (m)	Location	Reinforcement Details (Note 1)				Liner Supporting Fill
		T_{al} (kN/m)	S_v (m)	L (m)	n	
22 - 21	Bottom	121	0.3	Note 2	30	Note 4
	Middle	121	0.6		10	
	Top	72	0.6		Note 3	
21 - 19	Bottom	121	0.3	Note 2	27	Note 4
	Middle	121	0.6		10	
	Top	72	0.6		Note 3	
19 – 17	Bottom	121	0.3	Note 2	20	Note 5
	Middle	121	0.6	0.7H	10	
	Top	72	0.6	0.7H	Note 3	
17 - 15	Bottom	121	0.3	Note 2	13	Note 5
	Middle	121	0.6	0.7H	10	
	Top	72	0.6	0.7H	Note 3	
15 - 13	Bottom	121	0.3	0.7H	7	Note 5
	Middle	121	0.6		10	
	Top	72	0.6		Note 3	
13 - 11	Bottom	121	0.3	0.7H	5	Note 5
	Middle	121	0.6		5	
	Top	72	0.6		Note 3	
11 - 9	Bottom	121	0.6	0.7H	3	Note 5
	Middle	121			4	
	Top	72			Note 3	

H (m)	Location	Reinforcement Details (Note 1)				Liner Supporting Fill
		T_{al} (kN/m)	S_v (m)	L (m)	n	
9 - 7	Bottom	121	0.6	0.7H	4	Note 5
	Middle	72			4	
	Top	72			Note 3	
7 – 5	Bottom	72	0.6	0.8H	Note 3	Note 5
	Middle					
	Top					
5 - 3	Bottom	72	0.6	5	Note 3	N/A
	Middle					
	Top					
< 3	Bottom	48	0.6	4	Note 3	N/A
	Middle					
	Top					

Note

1. *T_{al} – Long-term reinforcement strength, S_v – Maximum vertical spacing of the reinforcement, L – Minimum length of reinforcement, n – No. of reinforcement*
2. *Reinforcement should be extended from facing to drainage chimney.*
3. *No. of layers varies based on the height of MSE wall.*
4. *Primary reinforcement and intermediate reinforcements are required as detailed in Section 6.0.*
5. *Intermediate reinforcement is only required as detailed in Section 6.0.*

8.0 FOUNDATION TREATMENT

As detailed in Section 2.1, the edge of the waste from Ch650 to Ch850 (approx.) extends partially within the foundation of the MSE wall. The depth of landfill waste below the existing ground level varies and is expected to be up to 2.5 m to 8.5 m within the footprint of the MSE wall. Within these chainages, the wall height varies from approximately 20 m (Ch650) to 10 m (Ch850). The minimum excavation to install the MSE wall (embedment) is approximately 1.2 m to 2.1 m. This will result in landfill below foundation level varying from 0.5 m to 4.5 m.

8.1 Analysis

Figure 14 shows the estimated settlement at top of the wall in 100 years at a cross section (Ch680). This estimation has deducted the settlement induced during the construction. The estimated settlement at the top of wall (underneath the traffic lanes) is approximately 0.35 m. The estimated settlement at top drainage chimney is in the order of 1.2 m.

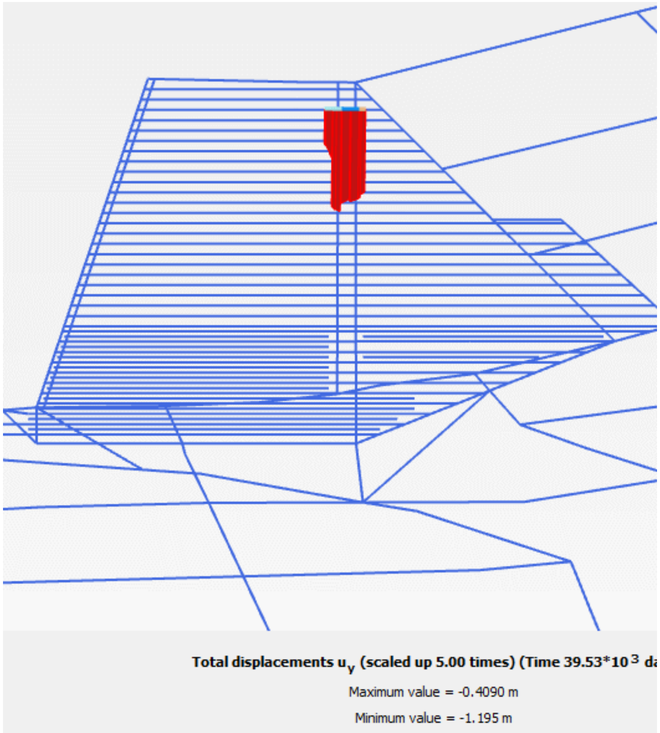


Figure 14: Estimated Settlement at Top of Wall (Ch 680m) in 100 Years without Foundation Treatment

Figure 15 shows the critical potential failure surface which is induced underneath the retaining wall. The minimum Factor of safety is estimated to be 1.8. The Safety Factor is estimated by the approach that the shear strength parameters tan φ and c of the soil as well as the tensile strength are successively reduced until failure of the structure occurs. This failure surface is consistent with the critical failure surface estimated in the limit equilibrium method (SlopeW).

The factor of safety of the wall against the overall stability is expected to be adequate (FoS = 1.8).

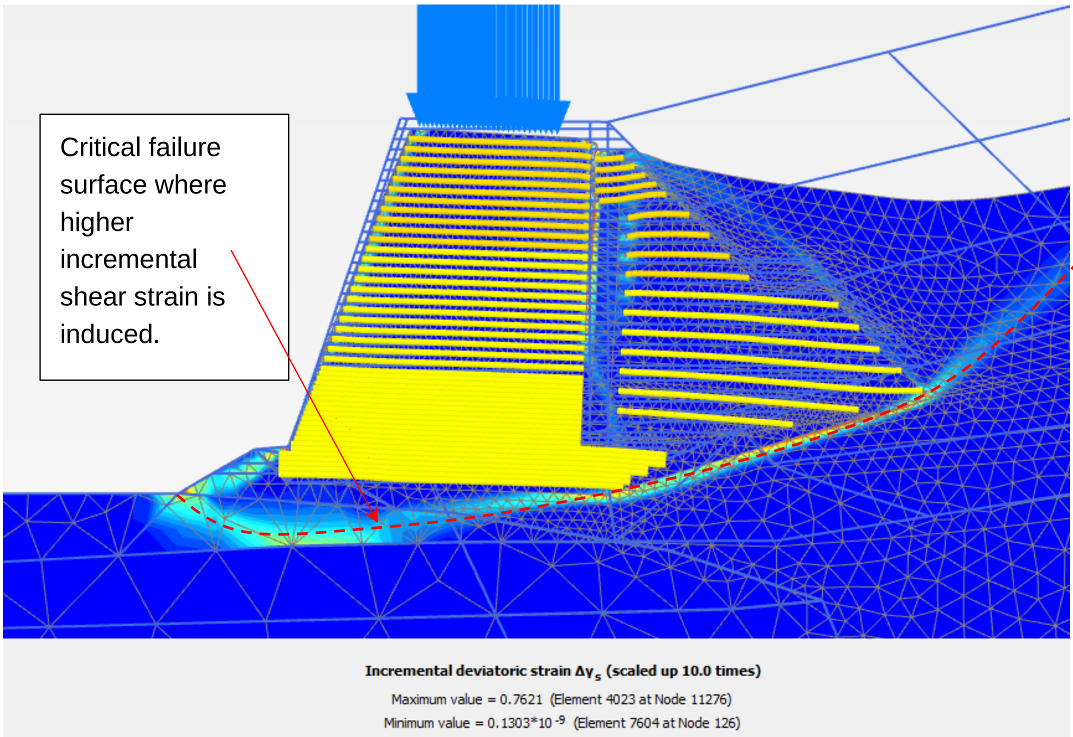


Figure 15: Potential Global Failure Surface from FEM

8.2 Possible Foundation Treatment

As part of the construction process, the MSE wall foundation will be observed to confirm that the foundation is likely to behave as designed. Where inspection of the founding material indicates that there is material that may degrade significantly, then measures may be implemented to reduce the potential settlement of the MSE wall and decrease the geotechnical risk related to excessive settlement induced instability. We recommend the minimum foundation treatment assessment process within the footprint of the MSE wall adopting the following methodologies.

1. Excavate to the design foundation level during construction phase.
2. Carry out a geotechnical inspection to assess the consistency and nature of the founding material and the potential extent of foundation treatment required.
3. Carry out a geotechnical investigation (test pits and trenches) to identify the extent of the landfill waste (depth).
4. Classify the landfill waste material. If the material is predominantly construction waste (Non-putrescible), it may be able to be reused with the appropriate compactive effort.
5. Revisit the geotechnical parameters of landfill waste and ground conditions adopted in the design. If required, carry out additional settlement assessment.
6. If the landfill waste contains a significant amount of decomposable material, the landfill can be excavated and replaced with engineered fill.

Figure 16 shows the estimated settlement at the top of the wall in 100 years at a cross section (Ch680) considering above foundation treatment (excavate and replace). The foundation treatment was modelled with non-creep material underneath the wall foundation. The estimated maximum settlement at top of wall (underneath the traffic lanes) is approximately 0.14 m. The estimated settlement at top drainage chimney is still in the range of 1.2 m. It is noted that in this analysis, the foundation treatment is only considered below footprint of the MSE wall. The settlement at top of drainage chimney may be reduced during detailed design by extending the foundation treatment beyond the drainage chimney.

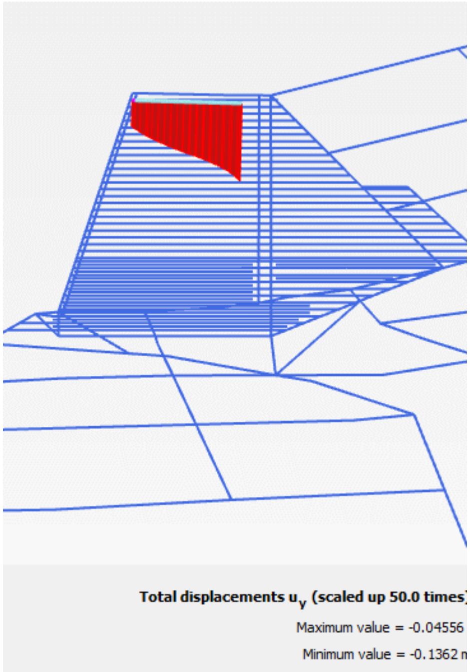


Figure 16: Estimated Settlement at Top of Wall (Ch 680m) in 100 Years for Proposed Foundation Treatment

9.0 RETAINING WALL FACING SYSTEM

9.1 Facing Type

The facing system for the MSE wall will provide physical support for the retained soil adjacent to the wall face to prevent the retained soil from ravelling out between the rows of reinforcement. It is important to note that overall wall stability and soil retention is provided not by the facing system, but by the main reinforcement layers within the fill. The facing system also serves to protect the main reinforcement layers from ultraviolet (uV) exposure to avoid long-term reinforcement degradation. In addition, the facing system promotes safe construction at the wall face because compaction is not required immediately adjacent to steep slopes face. As the MSE wall is permanent, the geogrid within the MSE wall is designed to be the primary face soil retention element and it is wrapped back within the reinforced fill as shown in Figure 17.

The schematic arrangement of the facing system is provided in Figure 17. The component of the system and their functions are detailed below.

- Main geogrid reinforcement wrap-around: The geogrid supports the gravel zone and fill soil laterally. This is the primary long-term facing soil retention measure.
- Steel bar mesh “L-shape”: The steel mesh is proposed as the outer element of the facing and is used as a forming device for the geogrid wrap-around during construction. The steel mesh is left in place after construction. The steel mesh is designed to carry no long-term load in the stability of the MSE wall.
- uV resistant Turf Reinforcement Mat: This material is heavily stabilised against uV attack and will line the inner face of the steel bar mesh. It provides a long-term UV shield for the main geogrid reinforcement and also assists in retaining the gravel. This material will be visible through the steel mesh and will give the wall its primary colour.
- Gravel (or suitably sized rock): Prevents water pressure build up at the face. It provides for facing constructability because the gravel zone can be placed without the need to operate soil compactors immediately adjacent to the wall face.
- Separation geotextile: This material will line the inner face of the geogrid wrap-around. It prevents fill soil migrating into the gravel and assists in retaining the gravel.

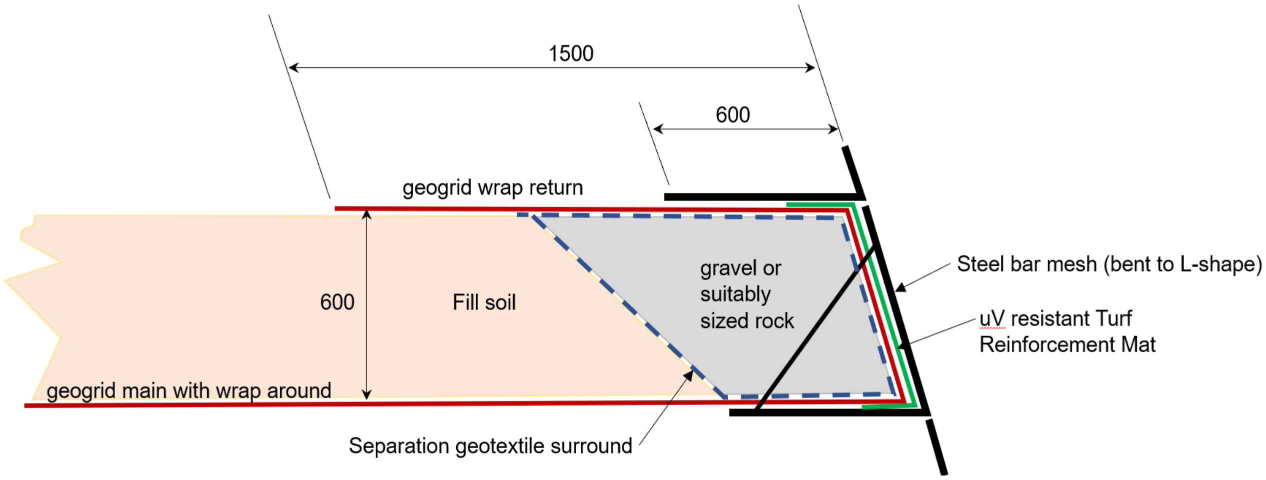


Figure 17: Schematic Arrangement of Facing

9.1.1 Analysis

The geogrid wrap return was assessed considering the force mechanism in each layer of reinforcement as shown in Figure 18. The reinforcement load in the wrap return, T_f is estimated as below and the required length of wrap return, l_w has been checked for the load combinations detailed in Section 4.2. The 0.6 m width (gravel or suitable rockfill width) has been ignored in the estimation of required length of l_w.

For the load combinations, the proposed minimum wrap of 1.5 m is adequate.

$T_f = \sigma_H S_v/2$

Where

σ_H - horizontal stress at any depth Z

S_v – Vertical spacing of the reinforcement layer

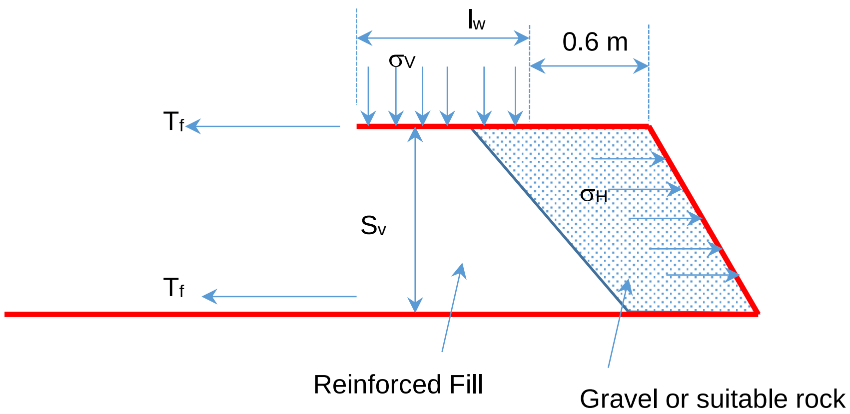


Figure 18: Schematic Force Diagram of Wrap Return

9.2 Visual Appearance

The visual appearance of the wall facing will be governed by the galvanised steel mesh and the uV resistant Turf Reinforcement Mat lining the inside of the steel mesh. A dark green colour is proposed for the uV resistant Turf Reinforcement Mat to enhance the aesthetic appearance of the wall. This colour is expected to be clearly visible through the apertures of the steel mesh.

9.3 Design life and durability

A very long facing design life (approximately 100 years) can be achieved with appropriate selection of the uV resistant Turf Reinforcement Mat. In addition, the steel wire mesh will be heavily galvanised for durability, as in gabion and similar construction, and is likely to achieve a long design life.

10.0 GEOTECHNICAL ASSESSMENT ON REVISED WALL GEOMETRY

10.1 Revised Geometry

We have carried out geotechnical assessment for a revised wall geometry, anticipating wall size reduction during detailed design development. For this assessment, the wall dimensions (wall height, h and width, W) are reduced as below and detailed in Table 15.

- Top width of the wall (W) is reduced by 3 m (i.e 13.5 m to 10.5 m, 12.0 m to 9.0 m)
- Height of the wall (H) reduced by up to 15% from ~Ch200 to ~Ch700 from the height provided in Table 1. Outside of these chainages, the height was not changed.
- The bottom width has been reduced considering the revised top width and revised height.
- Backfill inclination (β) was increased from 1V:4H to 1V:3H and the backfill elevation was limited to 93.5 mAHD.
- Embedment depth (d) of the revised wall was not changes from that in Table 1.

The geotechnical assessment for the revised geometry has included the internal, external and overall stability as per the methodology detailed in Sections 5.1 to 5.3 . Wall settlement analyses were not carried out for the revised wall dimensions as the wall size is reduced from the settlement assessment presented in Section 5.4.

Table 15: Minimum Dimensions Adopted for the Revised Preliminary Wall Design

Ch	RL_{EGL} (mAHD)	Revised W (m))	Revised h (m)	d (m)	Revised H (m)	Bottom width (m)
100	64.2	9.0	4.8	1.2	6.0	10.6
200	66.8	9.0	10.7	3.0	13.7	12.6
300	67.6	9.0	11.1	3.0	14.1	12.7
400	63.1	10.5	13.8	1.8	15.6	15.1
500	61.3	10.5	13.9	1.8	15.7	15.1
600	57.5	10.5	15.8	2.1	17.9	15.8
700	56.5	10.5	15.3	2.1	17.4	15.6
800	62.3	9.9	9.6	1.2	10.8	13.0
850	64.2	9.4	6.1	1.2	7.0	11.5

Note

1. *Refer to* Figure 1 *for wall geometry parameters.*
2. *Embedment depth of the MSE wall provided is based on the wall height (H) and slope in front of the wall. This was not revised in the assessment of the revised geometry.*

10.2 Revised Geotechnical Parameters

The geotechnical parameters were adopted for the geotechnical assessment of the revised wall geometry as provided in Table 3 except the internal friction angles of reinforced fill and liner supporting fill. The internal friction angles of reinforced fill and liner supporting fill have been revised to 34° and 30°, respectively.

10.3 Design Check

The following design checks were carried out to confirm that the revised wall geometry is adequate to meet the minimum design requirement as detailed in FHWA GEC 011.

10.3.1 External Stability

As detailed in FHWA GEC 011, a minimum reinforcement length as shown in Table 16 was initially adopted for the external stability of the MSE walls. The analysis results show that the proposed dimensions of the wall are satisfactory for the sliding, overturning and bearing.

Table 16: External Stability Check except Overall Stability

Ch	h (m)	H (m)	d (m)	w (m) (Note 1)	λ (Note 2)	Sliding Check	Overturning Check	Bearing Check
100	4.77	5.97	1.20	9.0	0.7	OK	OK	OK
200	10.65	13.65	3.00	9.0	0.7			
300	11.13	14.13	3.00	8.4	0.7			
400	13.76	15.56	1.80	9.9	0.7			
500	13.89	15.69	1.80	9.9	0.7			
600	15.83	17.93	2.10	9.9	0.75			
700	15.32	17.42	2.10	9.9	0.75			
800	9.56	10.76	1.20	9.3	0.7			
850	6.10	7.00	1.20	8.8	0.7			

Note

1. *Top width of the reinforced MSE wall is estimated as w = W – width of drainage chimney (taken as 0.6 m).*
2. *Minimum ratio between reinforcement length (L) and wall height (H) as per FHWA – GEC 011 to satisfy the external stability.*

10.3.2 Internal Stability

The minimum reinforcement length was adopted as provided in Table 17 to meet the internal stability of the revised wall geometry.

Table 17: Internal Stability Check

Ch	h (m)	H (m)	Location	Reinforcement Details (Note 1)				Tensile Check	Pull-Out Check
				T_{al} (kN/m)	S_v (m)	L (m)	n		
100	4.77	5.97	Bottom	72	0.6	4.5	3	OK	OK
			Middle	72	0.6	4.5	3		
			Top	72	0.6	6.0	3		
200	10.65	13.65	Bottom	121	0.6	8.4	10	OK	OK
			Middle	72	0.6	8.4	6		
			Top	72	0.6	8.4	6		
300	11.13	14.13	Bottom	121	0.6	8.4	11	OK	OK
			Middle	72	0.6	8.4	6		
			Top	72	0.6	8.4	6		
400	13.76	15.56	Bottom	121	0.3	9.9	6	OK	OK
			Middle	121	0.6	9.9	10		
			Top	72	0.6	9.9	12		
500	13.89	15.69	Bottom	121	0.3	9.9	6	OK	OK
			Middle	121	0.6	9.9	10		
			Top	72	0.6	9.9	12		
600	15.83	17.93	Bottom	121	0.3	9.9	14	OK	OK
			Middle	121	0.6	9.9	10		
			Top	72	0.6	9.9	12		
700	15.32	17.42	Bottom	121	0.3	9.9	13	OK	OK
			Middle	121	0.6	9.9	10		
			Top	72	0.6	9.9	12		
800	9.56	10.76	Bottom	121	0.6	7.5	6	OK	OK
			Middle	72	0.6	7.5	6		
			Top	72	0.6	7.5	5		

Ch	h (m)	H (m)	Location	Reinforcement Details (Note 1)				Tensile Check	Pull-Out Check
				T_{al} (kN/m)	S_v (m)	L (m)	n		
850	6.10	7.30	Bottom	72	0.6	5.5	4	OK	OK
			Middle	72	0.6	5.5	4		
			Top	72	0.6	6.0	3		

Note

1. *T_{al} – Long-term reinforcement strength, S_v – Maximum vertical spacing of the reinforcement, L – Minimum length of reinforcement, n – No. of reinforcement*
2. *Minimum reinforcement length (L) was assessed as per FHWA – GEC 011 to satisfy the internal stability.*

10.3.3 Overall Stability

Overall stability is assessed at two critical sections for the revised geometry as shown in Table 18 below. The assessment indicates that the MSE wall with the revised geometry achieves the target minimum factor of safety against overall stability.

Table 18: Summary of Overall Factor of Safety for the Revised Wall Geometry

Cross section	Factor of Safety					
	Case 1 (1.35)	Case 2 (1.35)	Case 3 (1.50)	Case 4 (1.50)	Case 5 (1.35)	Case 6 (1.10)
CS5 (Ch375)	1.75	1.48	1.51	1.50	1.51	1.39
CS7 (Ch650)	2.30	1.47	1.52	1.50	1.43	1.35

Note

2. *The factor provided in the bracket refers to the target FoS.*

The above design checks for the revised wall geometry shows that the reinforcement within the revised wall as detailed in Table 17 and external dimensions of the revised wall as detailed in Table 15 are adequate for the internal stability, external stability and overall stability.

11.0 OTHER CONSIDERATIONS

11.1 Unforeseen Ground Conditions

The internal and external stability, overall stability and settlement of the MSE wall system was assessed based on the existing geological information, existing site condition and design requirements. The foundation treatment proposed in Section 8.0 is based on the extent of the landfill that was interpreted based on existing geotechnical information. If the extent of the landfill (deeper landfill) is higher than interpreted, additional foundation treatment other than “excavate and replace” may be required.

11.2 Future works

The future works will include placement of landfill waste behind the MSE wall. This design package has assumed that future work will commence once the installation of the MSE wall including liner support fill and liner system is completed to an appropriate level to facilitate the placement of landfill waste and appropriate instrumentation are in place to monitor the wall performance.

11.3 Constructability

Typical construction of the MSE wall comprises the excavation to foundation level, assessment of foundation material, foundation preparation including foundation treatment, placement of reinforced fill with designed reinforcement layers. In addition, the construction of the MSE wall includes the installation of a drainage chimney and its components and installation of liner supporting fill slope with proposed reinforcement.

This geotechnical design has not addressed the requirement of plant platform and access, and stability of the temporary excavation slope and placement of the landfill. The temporary work requirement should be assessed based on the construction methodology to be adopted for construction of the MSE wall.

11.4 Instrumentation and Monitoring

Instrumentation will be installed prior to commencement of construction and during construction of the MSE wall to verify that the observed ground and wall behaviour is consistent with the predicted effects in the design. Instrumentation may include but will not be limited to the use of inclinometers, survey targets, settlement plates, settlement pins and piezometers.

A trigger response system will be developed to identify appropriate actions to be implemented, should certain amount of movement be observed during and immediately after construction. The detailed design will identify location and types of the instrumentation, timing of the installation, monitoring frequencies, trigger levels, and the appropriate actions to be undertaken for the trigger levels.

For example, alarms are generally set at specific percentages of the anticipated or tolerable movement and typically represent an Alert Level, a Response Level and a Stop Work Level.

At the Alert Level, actions that could be implemented include:

- Check that the movement is commensurate with the amount of construction that has occurred.
- Increase frequency of monitoring, if required.

At the Response Level, actions that could be implemented include:

- Increase frequency of monitoring to daily for survey.
- Review rate of movement occurring with time.
- Stop work, if required and review the ground model and the analysis.

- Supplement the monitoring regime.
- Prepare measures to enact should excessive movements continue.

At the Stop Work Level, actions that could be implemented include:

- Continue daily monitoring.
- Stop work and review the ground model and the analysis.
- Revisit the design
- Implement additional short-term measures to immediately reduce adverse behaviour (such as placement of a toe berm in front of a wall)
- Assess and implement remedial measures to reduce ongoing adverse movement in the long term.

By adopting such a system, it provides a verification loop during construction that the observed movements of the MSE wall are consistent with the design. It also provides additional confidence that the MSE wall will behave as designed and will not pose a risk to the neighbouring properties. It is anticipated that the monitoring program would be required during construction and for a period of up to 6 months after construction (typical). The monitoring period may be adjusted following review of collected instrumentation data.

12.0 CLOSURE

The preliminary design of the MSE wall detailed in this technical memorandum indicates that the wall is technically feasible, considering the site-specific geotechnical information and design methodology proposed above. On this basis, we recommend that the project be taken through to detailed design.

If you require additional information, please do not hesitate to contact the undersigned.

Signature Page

Golder Associates Pty Ltd

Loges Paramaguru
Principal Geotechnical Engineer

Tristan McWilliam
Principal Geotechnical Engineer

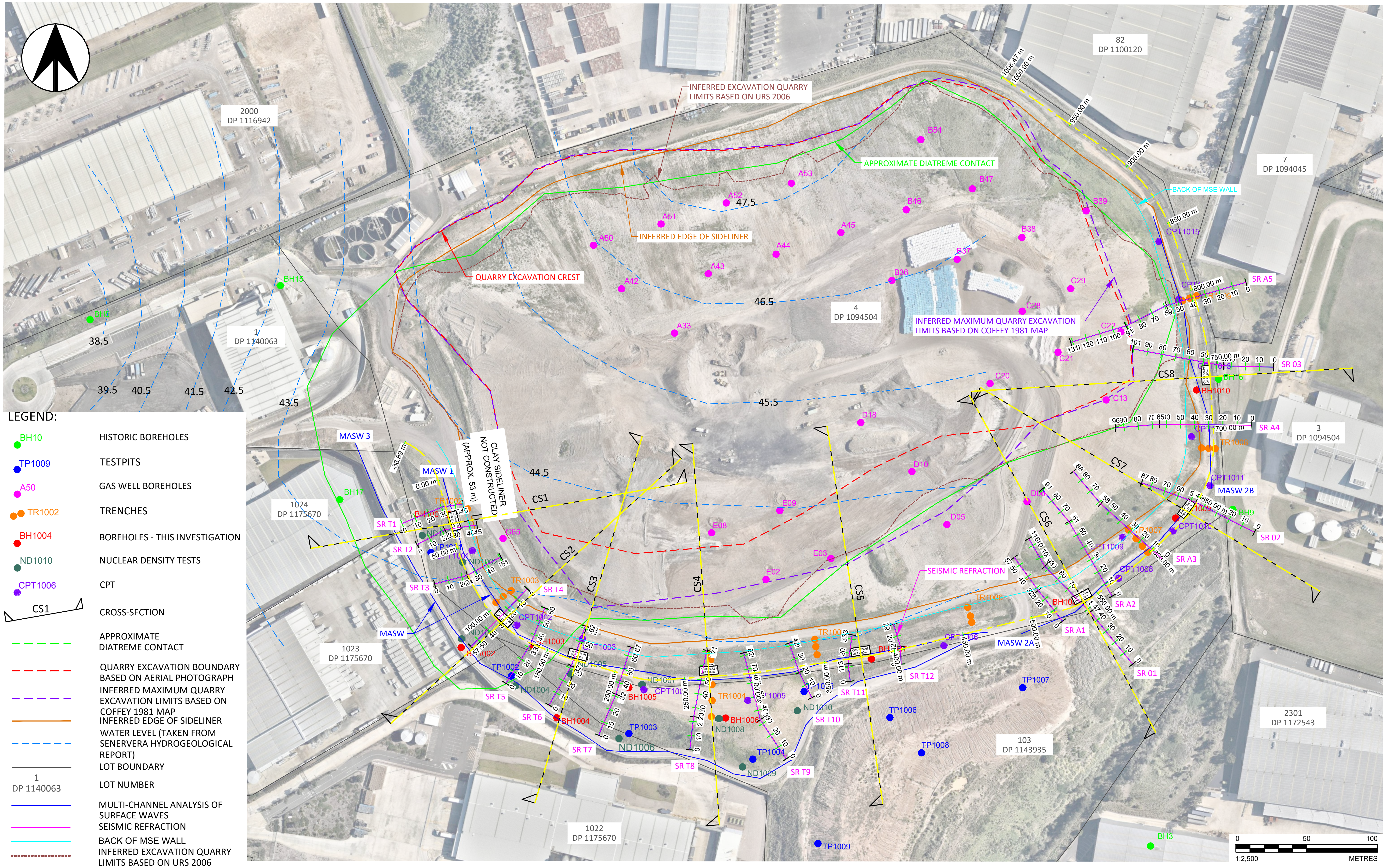
LP/TM/GS

A.B.N. 64 006 107 857

https://golderassociates.sharepoint.com/sites/120150/project files/6 deliverables/19135652-021-r-geotechnical design details/rev2/19135652 -021-r-rev2 geotechnical design of mse wall.docx

APPENDIX A

Geotechnical Long Sections and Cross Sections



REV. YYYY-MM-DD DESCRIPTION DESIGNED PREPARED REVIEWED APPROVED

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PROJECT
PROPOSED MSE RETAINING WALL
85-87 QUARRY ROAD, ERSKINE PARK NSW 2759

TITLE
INFERRED SUBSURFACE

PROJECT NO.	CONTROL	REV.	of	FIGURE
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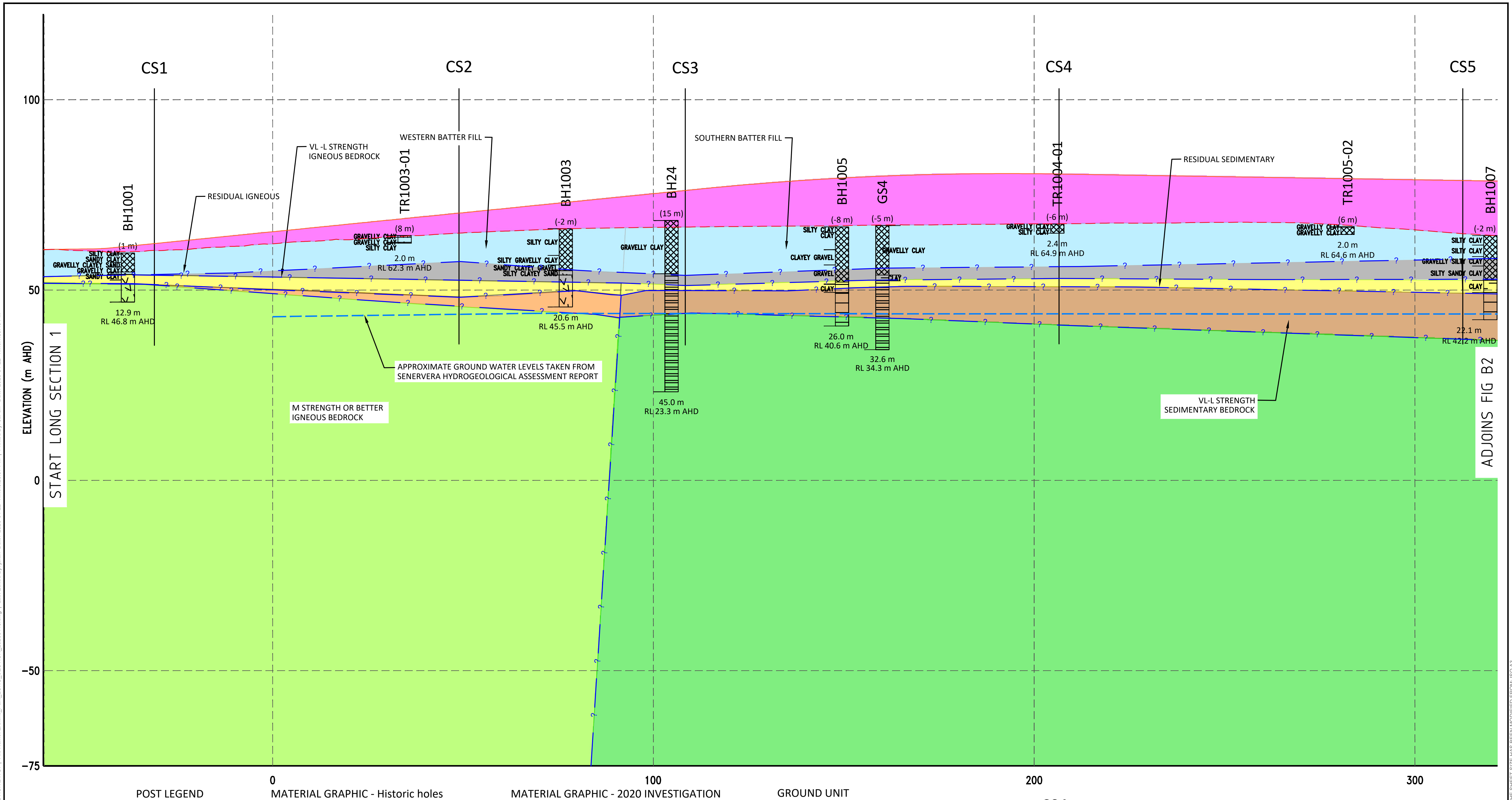
Document Set ID: 9209113
Version: 1, Version Date: 09/07/2020



POST LEGEND

Baseline Offset
Material Name
MUDSTONE
BH #
(0 m)
A
Geology Unit 1
Depth and Elevation at EOH

MATERIAL GRAPHIC - Historic holes

- FILL
- SILTSTONE
- TOPSOIL
- Sandy CLAY
- CLAY
- Clayey GRAVEL
- DOLERITE
- SHALE
- WATER LEVEL
- EG-SURVEY_2019-12-04_FULL_SIMPLIFIED_1M

MATERIAL GRAPHIC - 2020 INVESTIGATION

- FILL Material
- Very Stiff Clay
- Very low and low strength Dolerite
- Medium strength or better Dolerite
- Very Low and Low strength Siltstone bedrock
- Medium strength Siltstone bedrock
- Very Low Strength Volcanic Breccia
- Medium Strength Volcanic Breccia

GROUND UNIT

- 1a - Fill
- 1b - Controlled Fill
- 1c - Landfill Waste
- 2 - Residual
- 3a - Very Low to Low Strength Igneous Bedrock
- 3b - Medium Strength Igneous Bedrock or better
- 4a - Very Low to Low Strength Sedimentary Bedrock
- 4b - Medium Strength Sedimentary Bedrock or Better
- 5 - Proposed MSE Retaining Wall by Golder

CS4
CROSS SECTION LOCATION

0 25 50
1:1,000 VERTICAL METRES

0 25 50
1:1,000 HORIZONTAL METRES

REV.	YYYY-MM-DD	DESCRIPTION	DESIGNED	PREPARED	REVIEWED	APPROVED
A	2020-02-28	DRAFT	FY	FY	XX	XX

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PROJECT
PROPOSED MSE RETAINING WALL
85-87 QUARRY ROAD, ERSKINE PARK NSW 2759

TITLE
INFERRED SUBSURFACE LONG SECTION - LS1
0m - 315m

PROJECT NO.	CONTROL	REV.	of	FIGURE
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25 mm
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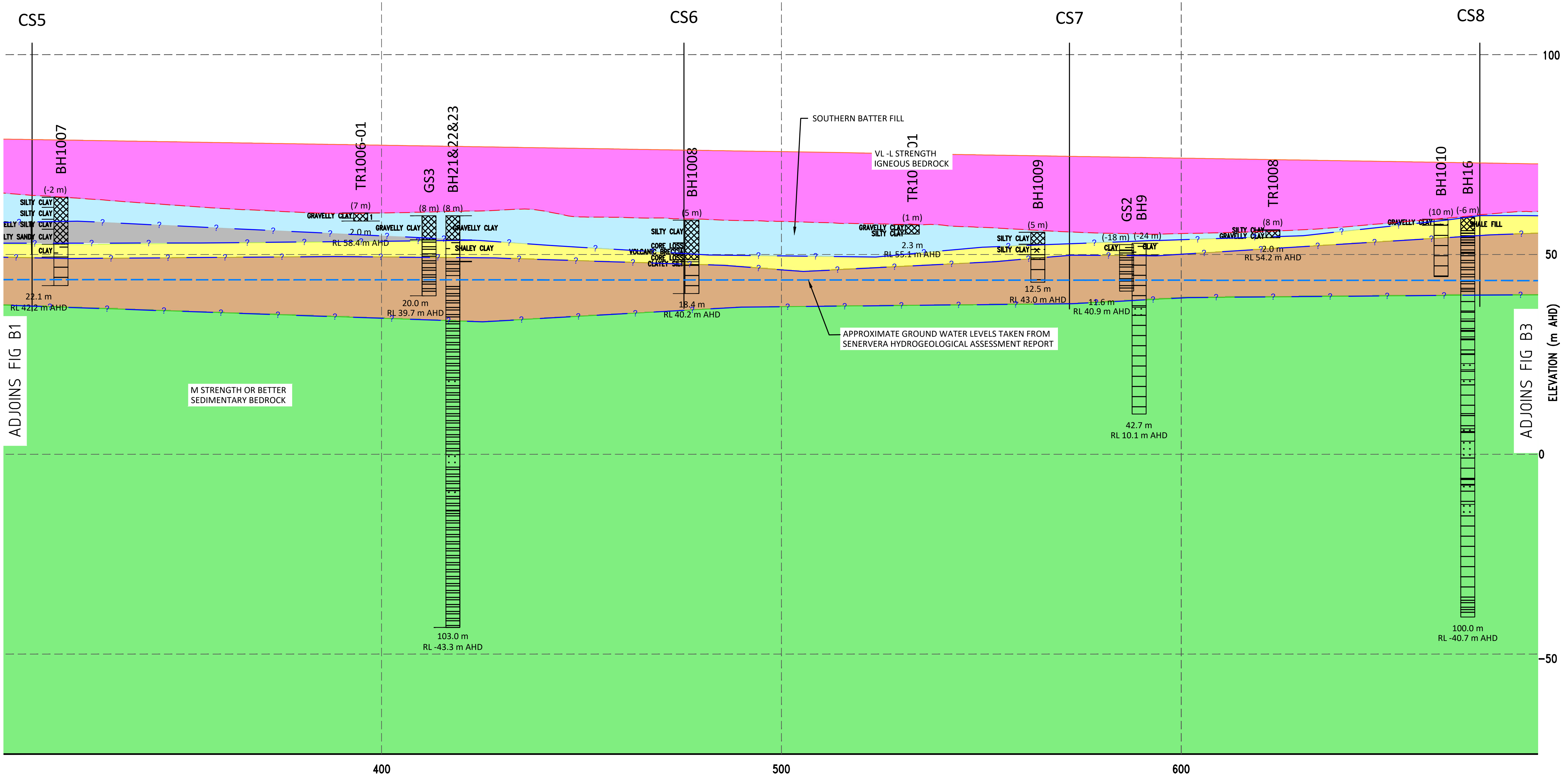

POST LEGEND

BH #
Baseline Offset — (0 m)
Material Name — MUDSTONE
Geology Unit 1 — A
Depth and Elevation at EOH

MATERIAL GRAPHIC - Historic holes

- FILL
- SILTSTONE
- TOPSOIL
- Sandy CLAY
- CLAY
- Clayey GRAVEL
- DOLERITE
- SHALE
- WATER LEVEL
- EG-SURVEY_2019-12-04_FULL_SIMPLIFIED_1M

MATERIAL GRAPHIC - 2020 INVESTIGATION

- FILL Material
- Very Stiff Clay
- Very low and low strength Dolerite
- Medium strength or better Dolerite
- Very Low and Low strength Siltstone bedrock
- Medium strength Siltstone bedrock
- Very Low Strength Volcanic Breccia
- Medium Strength Volcanic Breccia

GROUND UNIT

- 1a - Fill
- 1b - Controlled Fill
- 1c - Landfill Waste
- 2 - Residual
- 3a - Very Low to Low Strength Igneous Bedrock
- 3b - Medium Strength Igneous Bedrock or better
- 4a - Very Low to Low Strength Sedimentary Bedrock
- 4b - Medium Strength Sedimentary Bedrock or Better
- 5 - Proposed MSE Retaining Wall by Golder

CS4 — CROSS SECTION LOCATION

0 25 50
1:1,000 VERTICAL METRES

0 25 50
1:1,000 HORIZONTAL METRES

REV.	YYYY-MM-DD	DESCRIPTION	DESIGNED	PREPARED	REVIEWED	APPROVED
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TITLE
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315m - 675m

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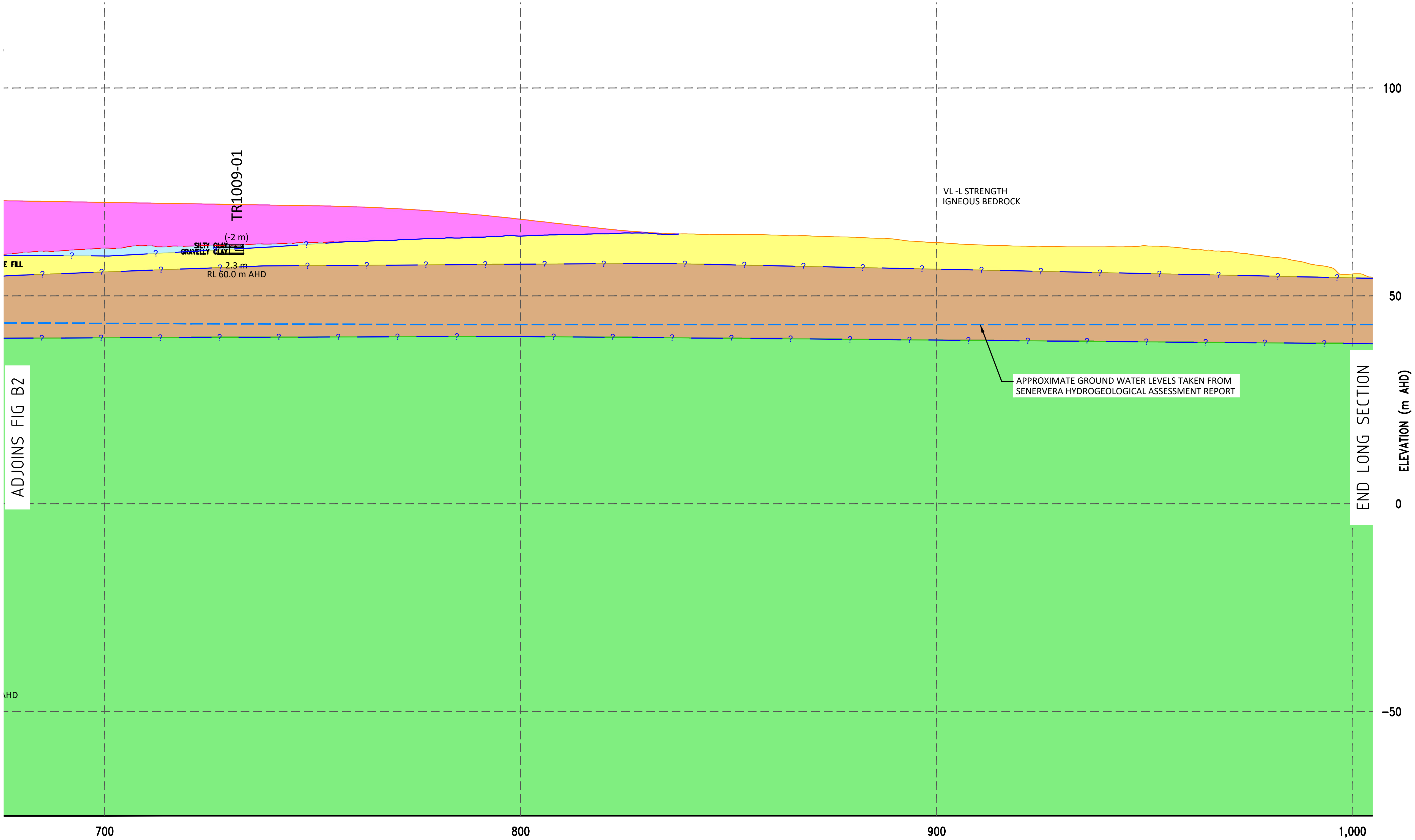

POST LEGEND

BH #
Baseline Offset
Material Name
(0 m)
MUDSTONE
A
Geology Unit 1
Depth and Elevation at EOH

MATERIAL GRAPHIC - Historic holes

- FILL
- SILTSTONE
- TOPSOIL
- Sandy CLAY
- CLAY
- Clayey GRAVEL
- DOLERITE
- SHALE
- WATER LEVEL
- EG-SURVEY_2019-12-04_FULL_SIMPLIFIED_1M

MATERIAL GRAPHIC - 2020 INVESTIGATION

- FILL Material
- Very Low and Low strength Siltstone bedrock
- Very Stiff Clay
- Medium strength Siltstone bedrock
- Very low and low strength Dolerite
- Very Low Strength Volcanic Breccia
- Medium strength or better Dolerite
- Medium Strength Volcanic Breccia

GROUND UNIT

- 1a - Fill
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- 4b - Medium Strength Sedimentary Bedrock or Better
- 5 - Proposed MSE Retaining Wall by Golder

CS4
CROSS SECTION LOCATION

0 25 50
1:1,000 VERTICAL METRES

0 25 50
1:1,000 HORIZONTAL METRES

REV.	YYYY-MM-DD	DESCRIPTION	DESIGNED	PREPARED	REVIEWED	APPROVED
A	2020-02-28	DRAFT	FY	FY	XX	XX XXX

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TITLE
INFERRED SUBSURFACE LONG SECTION - LS1
675m - 1,000m

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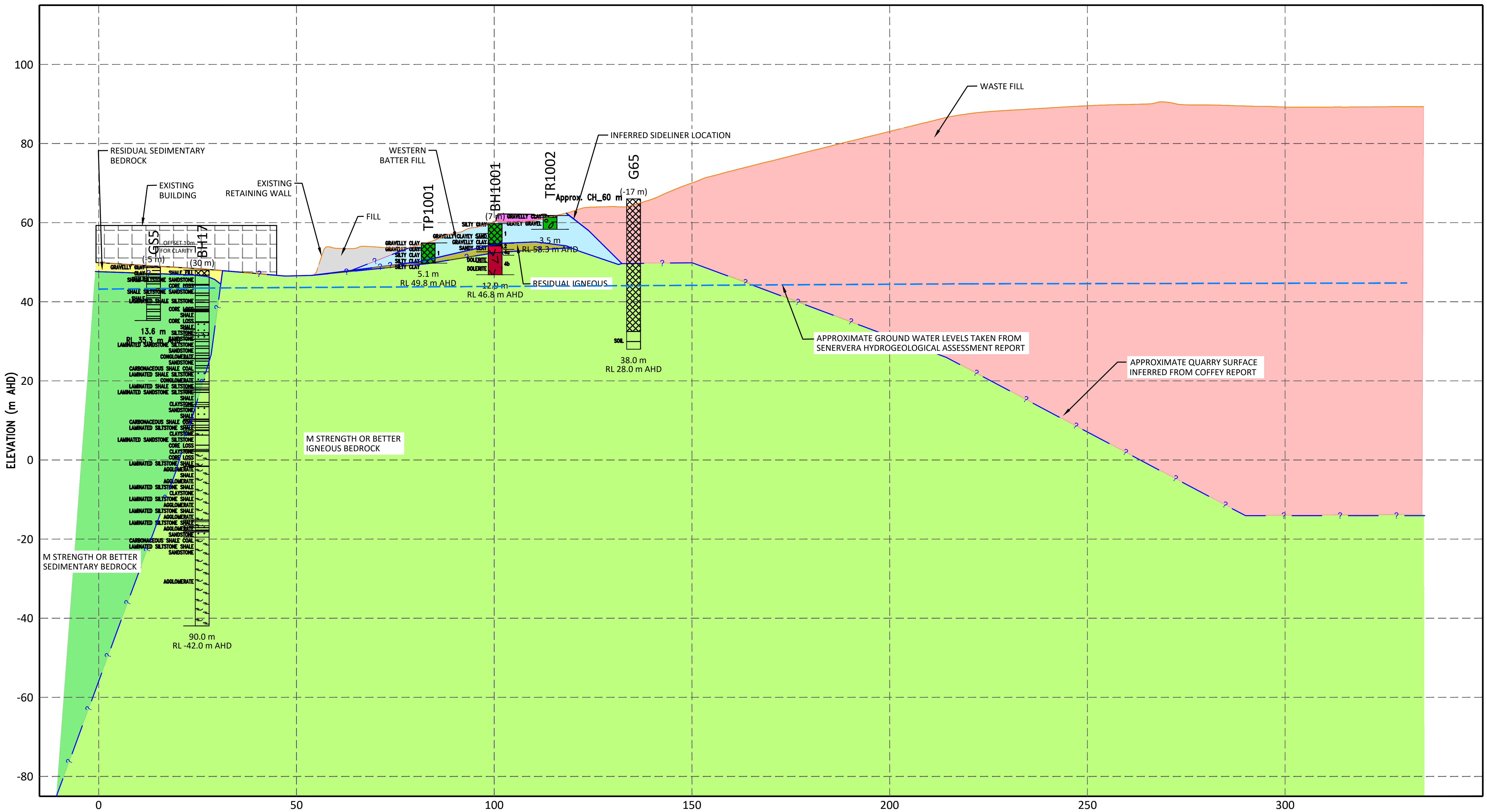

POST LEGEND

BH #

Baseline Offset (0 m)

Material Name MUDSTONE

Geology Unit 1

A

Depth and Elevation at EOH

MATERIAL GRAPHIC - Historic holes

- FILL
- SILTSTONE
- TOPSOIL
- Sandy CLAY
- CLAY
- Clayey GRAVEL
- DOLERITE
- SHALE
- WATER LEVEL
- EG-SURVEY_2019-12-04_FULL_SIMPLIFIED_1M

MATERIAL GRAPHIC - 2020 INVESTIGATION

- FILL Material
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- Medium strength or better Dolerite
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- Medium strength Siltstone bedrock
- Very Low Strength Volcanic Breccia
- Medium Strength Volcanic Breccia

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0 25 50
1:1,000 VERTICAL METRES

0 25 50
1:1,000 HORIZONTAL METRES

REV.	YYYY-MM-DD	DESCRIPTION	DESIGNED	PREPARED	REVIEWED	APPROVED
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TITLE
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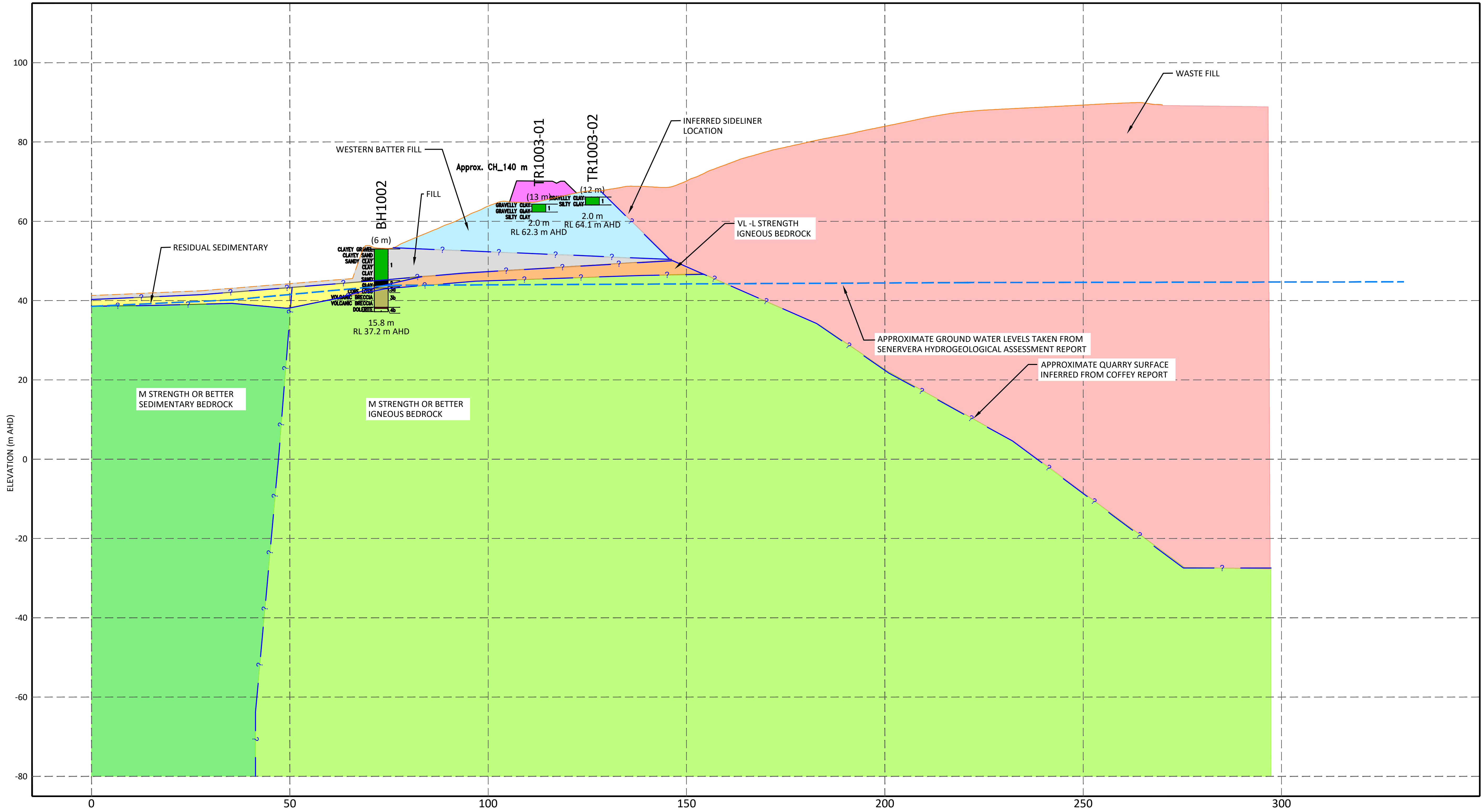

POST LEGEND

BH #
Baseline Offset
Material Name
(0 m)
MUDSTONE
Geology Unit 1
Depth and Elevation at EOH

MATERIAL GRAPHIC - Historic holes

FILL, SILTSTONE, TOPSOIL, Sandy CLAY, CLAY, Clayey GRAVEL, DOLERITE, SHALE

WATER LEVEL

EG-SURVEY_2019-12-04_FULL_SIMPLIFIED_1M

MATERIAL GRAPHIC - 2020 INVESTIGATION

- FILL Material
- Very Stiff Clay
- Very low and low strength Dolerite
- Medium strength or better Dolerite
- Very Low and Low strength Siltstone bedrock
- Medium strength Siltstone bedrock
- Very Low Strength Volcanic Breccia
- Medium Strength Volcanic Breccia

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- 3b - Medium Strength Igneous Bedrock or better
- 4a - Very Low and Low Sedimentary Bedrock
- 4b - Medium Strength Sedimentary Bedrock or better
- 5 - Proposed MSE Retaining Wall by Golder

0 25 50
1:1,000 VERTICAL METRES
0 25 50
1:1,000 HORIZONTAL METRES

REV.	YYYY-MM-DD	DESCRIPTION	DESIGNED	PREPARED	REVIEWED	APPROVED
A	2020-02-28	DRAFT	FY	FY	XX	XX	XXX

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PROJECT
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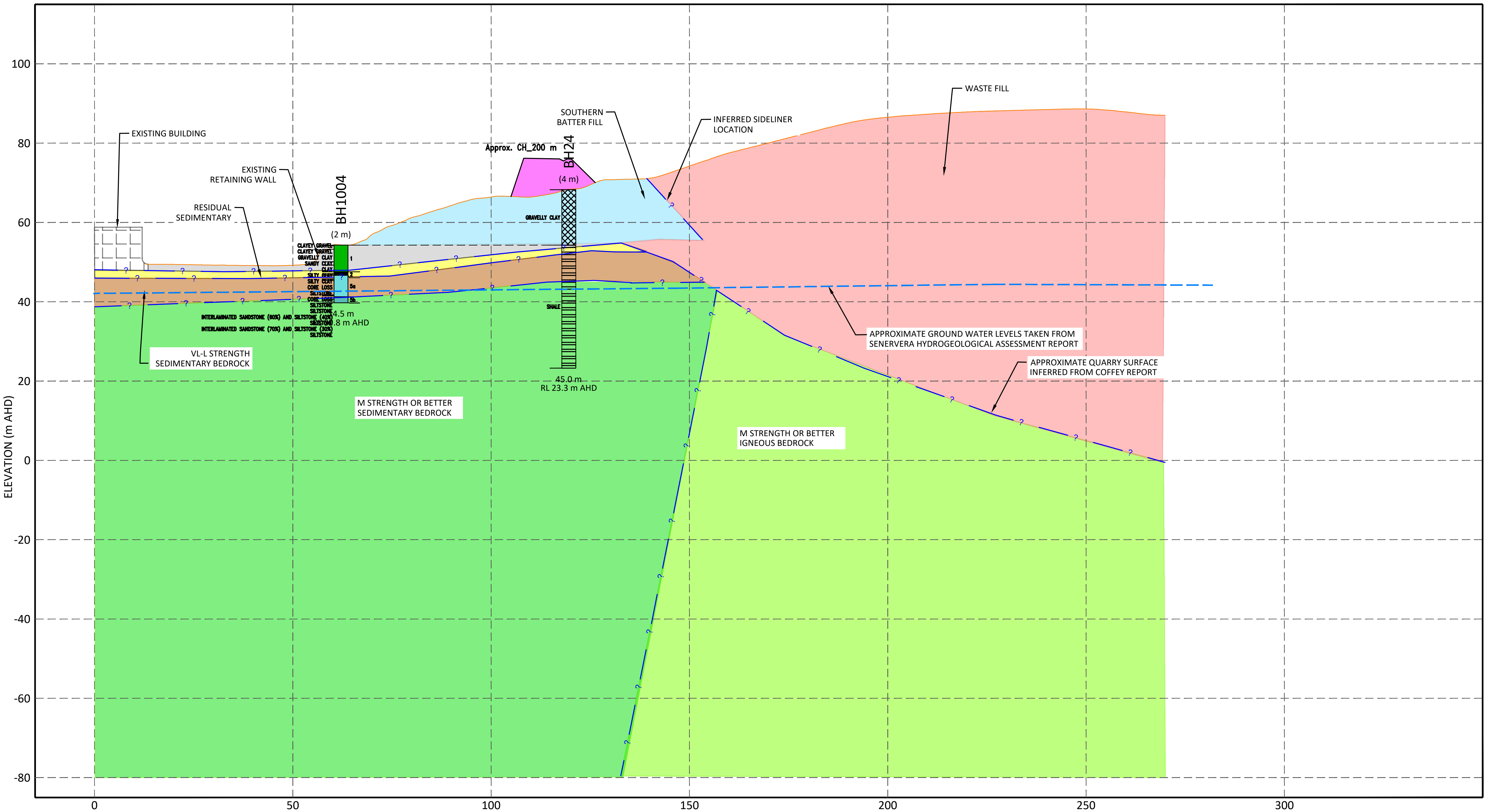

POST LEGEND

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Baseline Offset (0 m)
Material Name
Geology Unit 1
MUDSTONE A
Depth and Elevation at EOH

MATERIAL GRAPHIC - Historic holes

- FILL
- SILTSTONE
- TOPSOIL
- Sandy CLAY
- CLAY
- Clayey GRAVEL
- DOLERITE
- SHALE
- WATER LEVEL
- EG-SURVEY_2019-12-04_FULL_SIMPLIFIED_1M

MATERIAL GRAPHIC - 2020 INVESTIGATION

- FILL Material
- Very Stiff Clay
- Very low and low strength Dolerite
- Medium strength or better Dolerite
- Very Low and Low strength Siltstone bedrock
- Medium strength Siltstone bedrock
- Very Low Strength Volcanic Breccia
- Medium Strength Volcanic Breccia

GROUND UNIT

- 1a - Fill
- 1b - Controlled Fill
- 1c - Landfill Waste
- 2 - Residual
- 3a - Very Low and Low Igneous Bedrock
- 3b - Medium Strength Igneous Bedrock or better
- 4a - Very Low and Low Sedimentary Bedrock
- 4b - Medium Strength Sedimentary Bedrock or better
- 5 - Proposed MSE Retaining Wall by Golder

0 25 50
1:1,000 VERTICAL METRES

0 25 50
1:1,000 HORIZONTAL METRES

REV.	YYYY-MM-DD	DESCRIPTION	DESIGNED	PREPARED	REVIEWED	APPROVED
A	2020-02-28	DRAFT	FY	FY	XX	XX	XXX

CLIENT
ENVIROGUARD PTY LTD

PROJECT
PROPOSED MSE RETAINING WALL
85-87 QUARRY ROAD, ERSKINE PARK NSW 2759

CONSULTANT
GOLDER

SYDNEY OFFICE
LEVEL 8, 40 MOUNT STREET
NORTH SYDNEY, NSW 2060
AUSTRALIA
[+61] (02) 9478 3900
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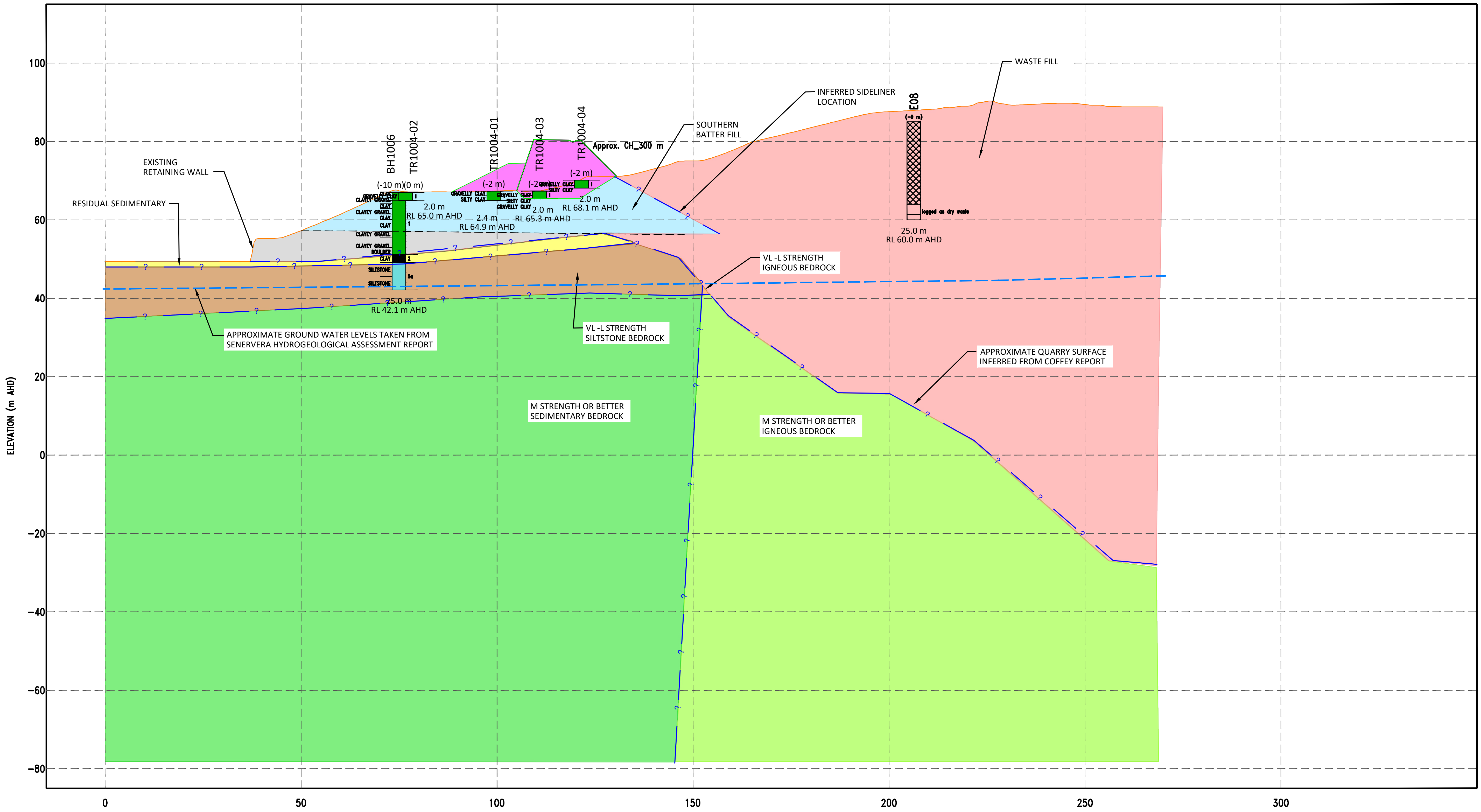
TITLE
INFERRED SUBSURFACE SECTION - CS3

PROJECT NO.	CONTROL	REV.	of	FIGURE
19135652	002	A		B-3



POST LEGEND

BH #
Baseline Offset
Material Name
(0 m)
MUDSTONE
A
Geology Unit 1
Depth and Elevation at EOH

MATERIAL GRAPHIC - Historic holes

- FILL
- SILTSTONE
- TOPSOIL
- Sandy CLAY
- CLAY
- Clayey GRAVEL
- DOLERITE
- SHALE
- WATER LEVEL
- EG-SURVEY_2019-12-04_FULL_SIMPLIFIED_1M

MATERIAL GRAPHIC - 2020 INVESTIGATION

- FILL Material
- Very Stiff Clay
- Very low and low strength Dolerite
- Medium strength or better Dolerite
- Very Low and Low strength Siltstone bedrock
- Medium strength Siltstone bedrock
- Very Low Strength Volcanic Breccia
- Medium Strength Volcanic Breccia

GROUND UNIT

- 1a - Fill
- 1b - Controlled Fill
- 1c - Landfill Waste
- 2 - Residual
- 3a - Very Low and Low Igneous Bedrock
- 3b - Medium Strength Igneous Bedrock or better
- 4a - Very Low and Low Sedimentary Bedrock
- 4b - Medium Strength Sedimentary Bedrock or better
- 5 - Proposed MSE Retaining Wall by Golder

0 25 50
1:1,000 VERTICAL METRES

0 25 50
1:1,000 HORIZONTAL METRES

REV.	YYYY-MM-DD	DESCRIPTION	DESIGNED	PREPARED	REVIEWED	APPROVED
A	2020-02-28	DRAFT	FY	FY	XX	XX

XXX

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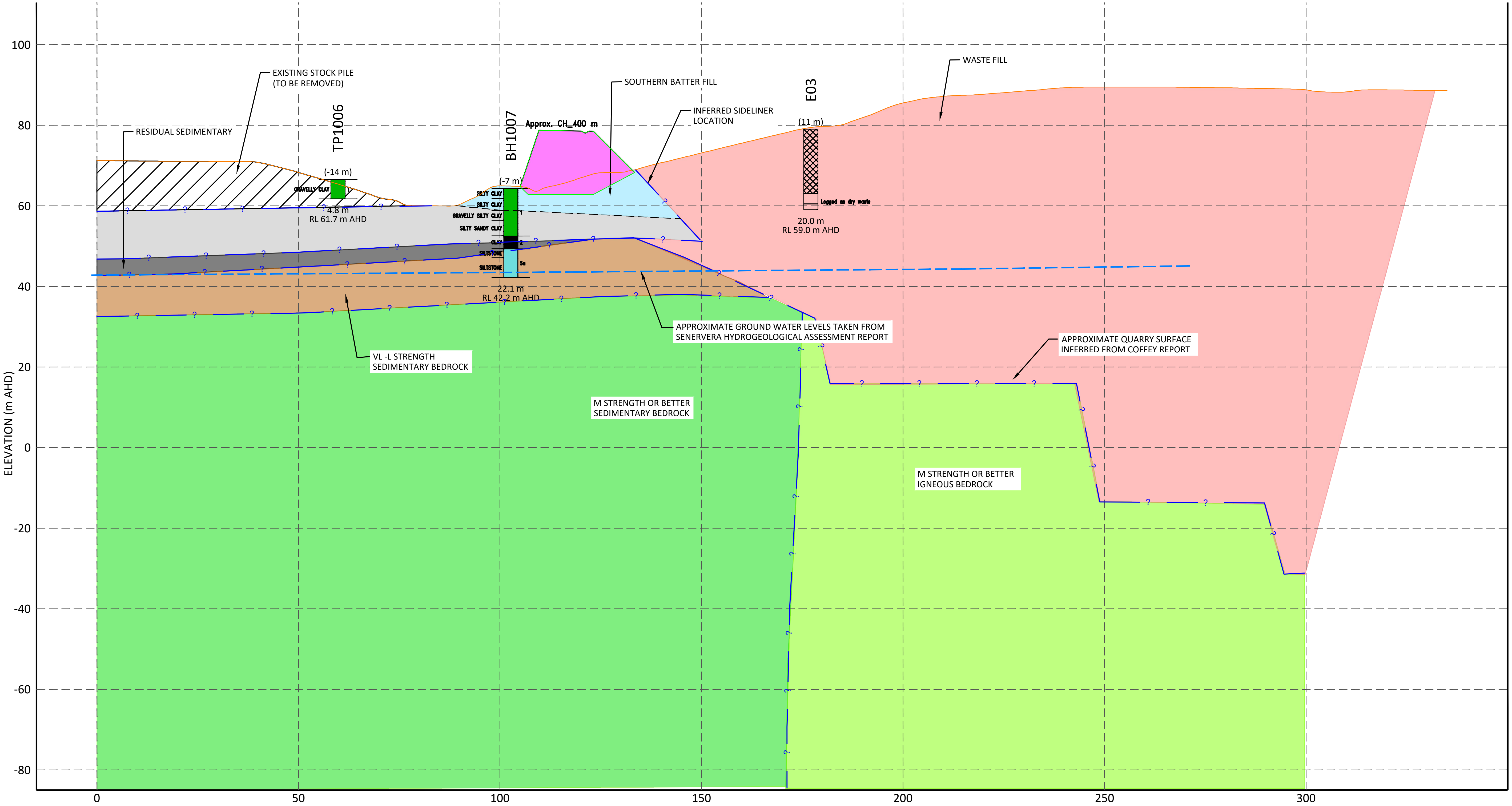
PROJECT
PROPOSED MSE RETAINING WALL
85-87 QUARRY ROAD, ERSKINE PARK NSW 2759

TITLE
INFERRED SUBSURFACE SECTION - CS4

PROJECT NO.	CONTROL	REV.	of	FIGURE
1913562	002	A		B-4

25 mm
IF THIS MEASUREMENT DOES NOT MATCH WHAT IS SHOWN, THE SHEET SIZE HAS BEEN MODIFIED FROM: ISO A3



POST LEGEND

Baseline Offset
Material Name
MUDSTONE
BH #
(0 m)
Geology Unit 1
A
Depth and Elevation at EOH

MATERIAL GRAPHIC - Historic holes

- FILL
- SILTSTONE
- TOPSOIL
- Sandy CLAY
- CLAY
- Clayey GRAVEL
- DOLERITE
- SHALE
- WATER LEVEL
- EG-SURVEY_2019-12-04_FULL_SIMPLIFIED_1M

MATERIAL GRAPHIC - 2020 INVESTIGATION

- FILL Material
- Very Stiff Clay
- Very low and low strength Dolerite
- Medium strength or better Dolerite
- Very Low and Low strength Siltstone bedrock
- Medium strength Siltstone bedrock
- Very Low Strength Volcanic Breccia
- Medium Strength Volcanic Breccia

GROUND UNIT

- 1a - Fill
- 1b - Controlled Fill
- 1c - Landfill Waste
- 2 - Residual
- 3a - Very Low and Low Igneous Bedrock
- 3b - Medium Strength Igneous Bedrock or better
- 4a - Very Low and Low Sedimentary Bedrock
- 4b - Medium Strength Sedimentary Bedrock or better
- 5 - Proposed MSE Retaining Wall by Golder

0 25 50
1:1,000 VERTICAL METRES

0 25 50
1:1,000 HORIZONTAL METRES

REV.	YYYY-MM-DD	DESCRIPTION	DESIGNED	PREPARED	REVIEWED	APPROVED
A	2020-02-28	DRAFT	FY	FY	XX	XX

XXX

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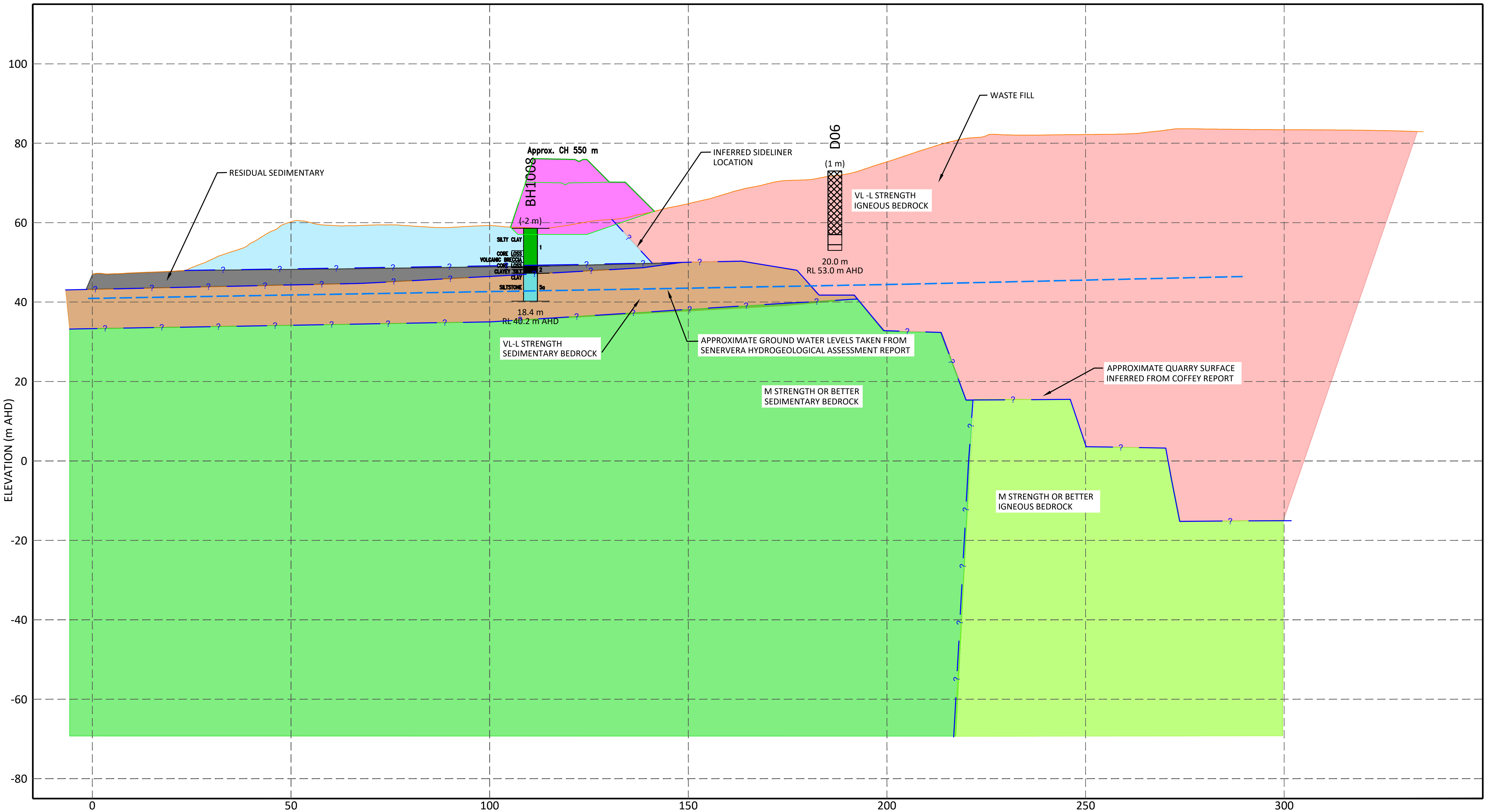
PROJECT
PROPOSED MSE RETAINING WALL
85-87 QUARRY ROAD, ERSKINE PARK NSW 2759

TITLE
INFERRED SUBSURFACE SECTION - CS5

PROJECT NO.	CONTROL	REV.	of	FIGURE
19135652	002	A		B-5



POST LEGEND

BH #
Baseline Offset
Material Name
(0 m)
Geology Unit 1
MUDSTONE
A
Depth and Elevation at EOH

MATERIAL GRAPHIC - Historic holes

- FILL
- SILTSTONE
- TOPSOIL
- Sandy CLAY
- CLAY
- Clayey GRAVEL
- DOLERITE
- SHALE
- WATER LEVEL
- EG-SURVEY_2019-12-04_FULL_SIMPLIFIED_1M

MATERIAL GRAPHIC - 2020 INVESTIGATION

- FILL Material
- Very Stiff Clay
- Very low and low strength Dolerite
- Medium strength or better Dolerite
- Very Low and Low strength Siltstone bedrock
- Medium strength Siltstone bedrock
- Very Low Strength Volcanic Breccia
- Medium Strength Volcanic Breccia

GROUND UNIT

- 1a - Fill
- 1b - Controlled Fill
- 1c - Landfill Waste
- 2 - Residual
- 3a - Very Low and Low Igneous Bedrock
- 3b - Medium Strength Igneous Bedrock or better
- 4a - Very Low and Low Sedimentary Bedrock
- 4b - Medium Strength Sedimentary Bedrock or better
- 5 - Proposed MSE Retaining Wall by Golder

0 25 50
1:1,000 VERTICAL METRES

0 25 50
1:1,000 HORIZONTAL METRES

REV.	YYYY-MM-DD	DESCRIPTION	DESIGNED	PREPARED	REVIEWED	APPROVED
A	2020-02-28	DRAFT	FY	FY	XX	XX

XXX

CLIENT
ENVIROGUARD PTY LTD

PROJECT
PROPOSED MSE RETAINING WALL
85-87 QUARRY ROAD, ERSKINE PARK NSW 2759

CONSULTANT
GOLDER

SYDNEY OFFICE
LEVEL 8, 40 MOUNT STREET
NORTH SYDNEY, NSW 2060
AUSTRALIA
[+61] (02) 9478 3900
www.golder.com

TITLE
INFERRED SUBSURFACE SECTION - CS6

PROJECT NO.	CONTROL	REV.	of	FIGURE
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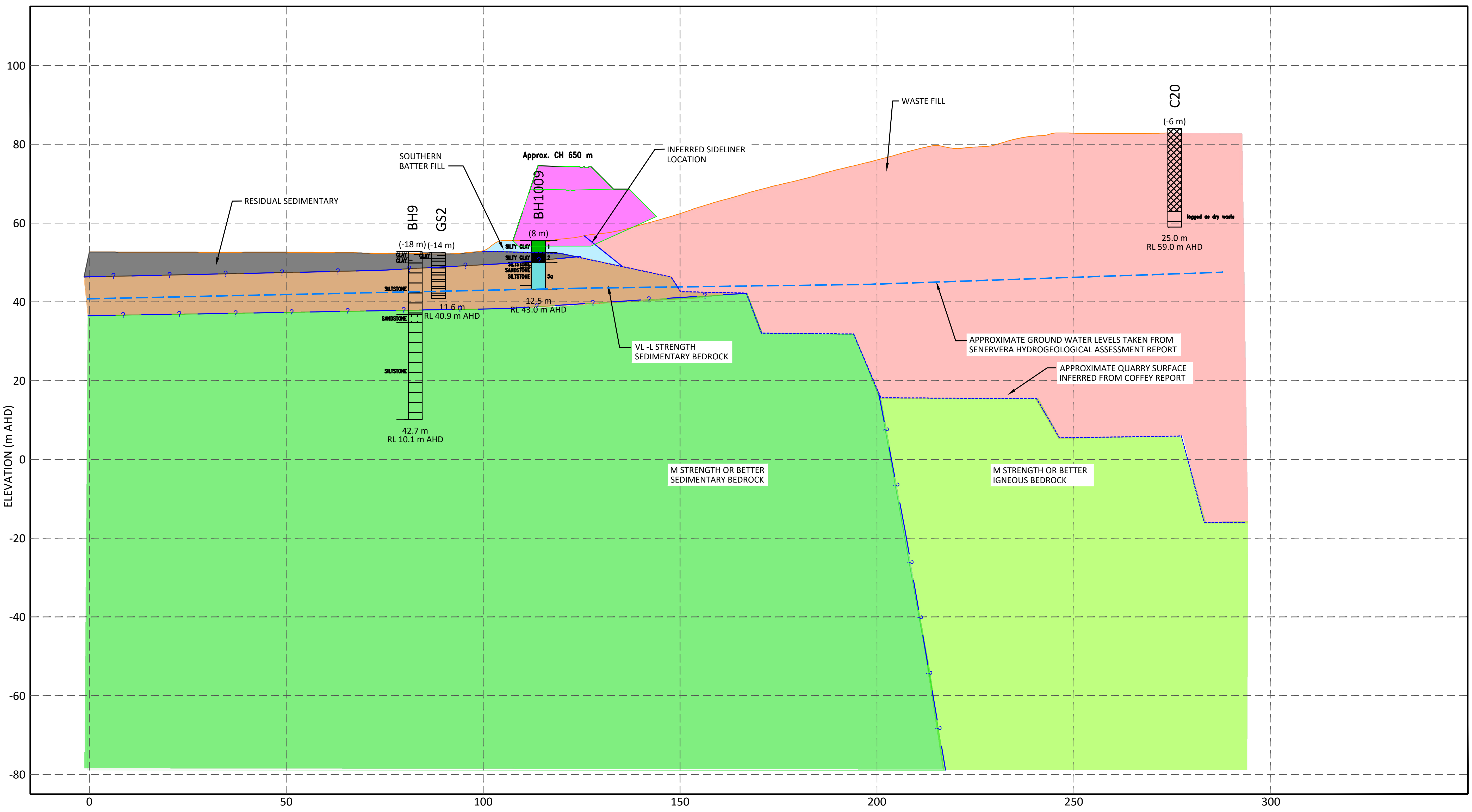

POST LEGEND

BH #
Baseline Offset
Material Name
(0 m)
MUDSTONE
Geology Unit 1
A
Depth and Elevation at EOH

MATERIAL GRAPHIC - Historic holes

- FILL
- SILTSTONE
- TOPSOIL
- Sandy CLAY
- CLAY
- Clayey GRAVEL
- DOLERITE
- SHALE
- WATER LEVEL
- EG-SURVEY_2019-12-04_FULL_SIMPLIFIED_1M

MATERIAL GRAPHIC - 2020 INVESTIGATION

- FILL Material
- Very Stiff Clay
- Very low and low strength Dolerite
- Medium strength or better Dolerite
- Very Low and Low strength Siltstone bedrock
- Medium strength Siltstone bedrock
- Very Low Strength Volcanic Breccia
- Medium Strength Volcanic Breccia

GROUND UNIT

- 1a - Fill
- 1b - Controlled Fill
- 1c - Landfill Waste
- 2 - Residual
- 3a - Very Low and Low Igneous Bedrock
- 3b - Medium Strength Igneous Bedrock or better
- 4a - Very Low and Low Sedimentary Bedrock
- 4b - Medium Strength Sedimentary Bedrock or better
- 5 - Proposed MSE Retaining Wall by Golder

0 25 50
1:1,000 VERTICAL METRES

0 25 50
1:1,000 HORIZONTAL METRES

REV.	YYYY-MM-DD	DESCRIPTION	DESIGNED	PREPARED	REVIEWED	APPROVED
A	2020-02-28	DRAFT	FY	FY	XX	XX	XXX

CLIENT
ENVIROGUARD PTY LTD

CONSULTANT
GOLDER
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AUSTRALIA
[+61] (02) 9478 3900
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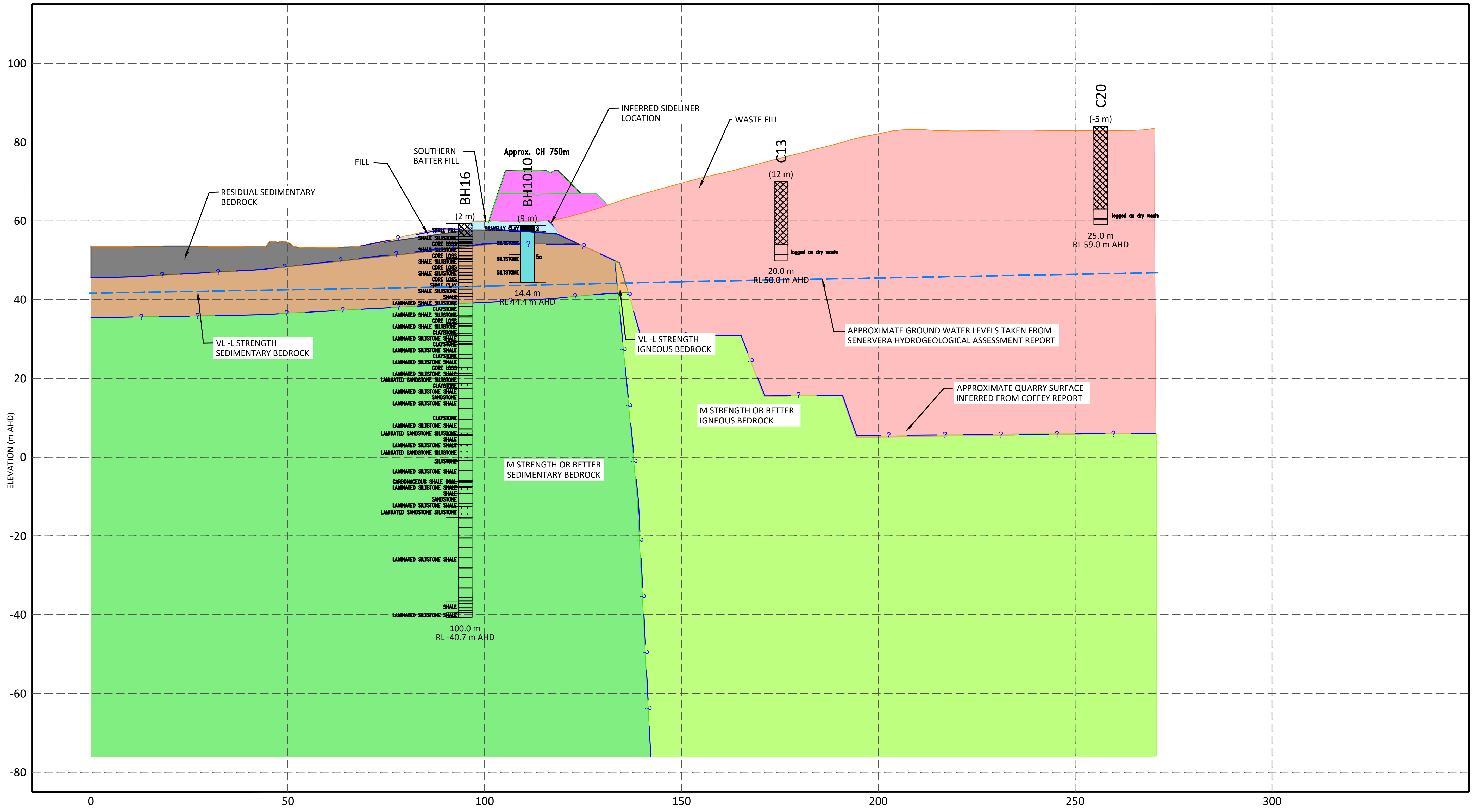
PROJECT
PROPOSED MSE RETAINING WALL
85-87 QUARRY ROAD, ERSKINE PARK NSW 2759

TITLE
INFERRED SUBSURFACE SECTION - CS7

PROJECT NO. 19135652
CONTROL 002
REV. A
of
FIGURE B-7



POST LEGEND

BH #

Baseline Offset — Material Name — MUDSTONE — (0 m) — A — Geology Unit 1

Depth and Elevation at EOH

MATERIAL GRAPHIC - Historic holes

- FILL
- SILTSTONE
- TOPSOIL
- Sandy CLAY
- CLAY
- Clayey GRAVEL
- DOLERITE
- SHALE
- WATER LEVEL
- EG-SURVEY_2019-12-04_FULL_SIMPLIFIED_1M

MATERIAL GRAPHIC - 2020 INVESTIGATION

- FILL Material
- Very Low and Low strength Siltstone bedrock
- Very Stiff Clay
- Medium strength Siltstone bedrock
- Very low and low strength Dolerite
- Very Low Strength Volcanic Breccia
- Medium strength or better Dolerite
- Medium Strength Volcanic Breccia

GROUND UNIT

- 1a - Fill
- 1b - Controlled Fill
- 1c - Landfill Waste
- 2 - Residual
- 3a - Very Low and Low Igneous Bedrock
- 3b - Medium Strength Igneous Bedrock or better
- 4a - Very Low and Low Sedimentary Bedrock
- 4b - Medium Strength Sedimentary Bedrock or better
- 5 - Proposed MSE Retaining Wall by Golder

0 25 50
1:1,000 VERTICAL METRES

0 25 50
1:1,000 HORIZONTAL METRES

REV.	YYYY-MM-DD	DESCRIPTION	DESIGNED	PREPARED	REVIEWED	APPROVED
A	2020-02-28	DRAFT	FY	FY	XX	XX

XXX

CLIENT
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GOLDER

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AUSTRALIA
[+61] (02) 9478 3900
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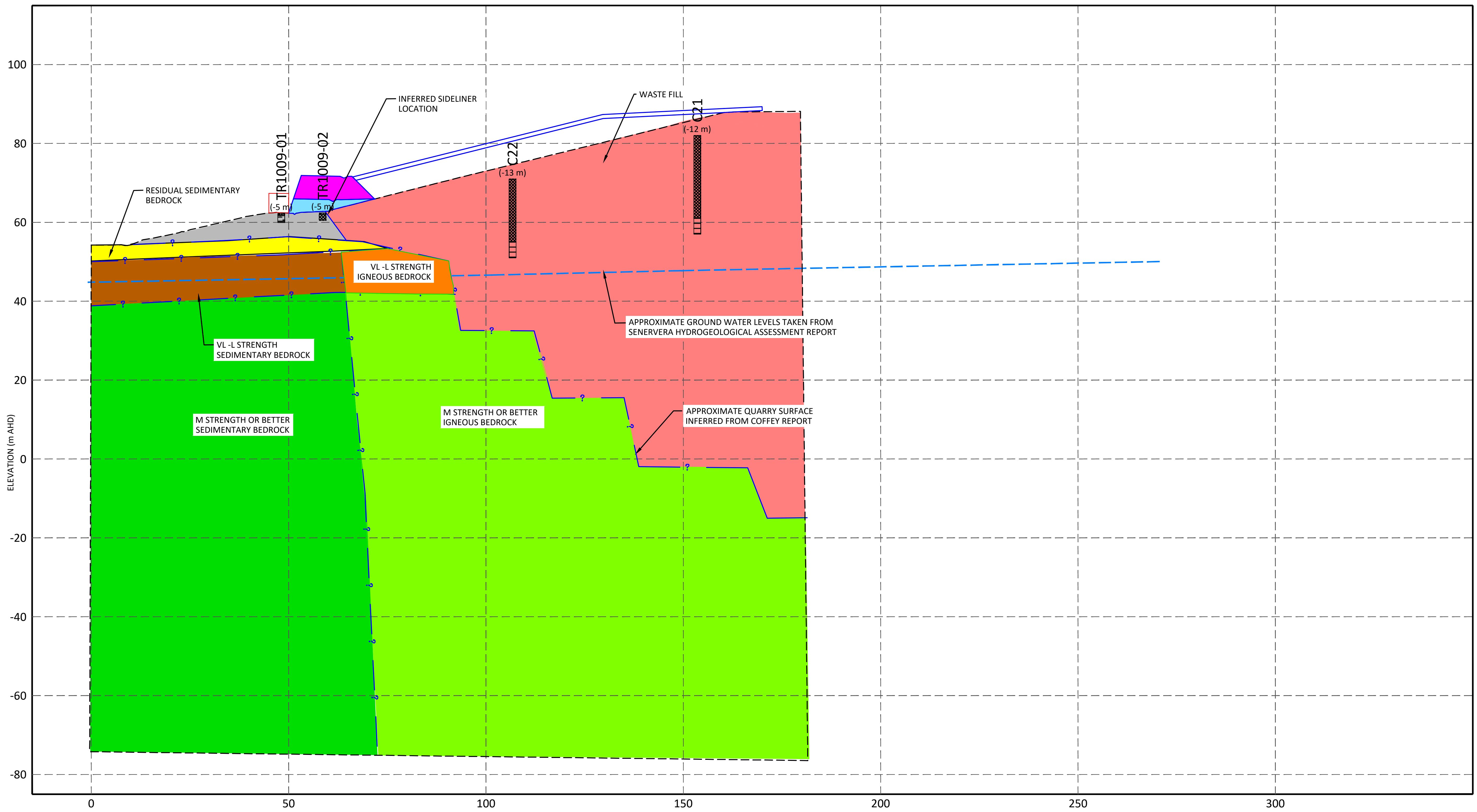
PROJECT
PROPOSED MSE RETAINING WALL
85-87 QUARRY ROAD, ERSKINE PARK NSW 2759

TITLE
INFERRED SUBSURFACE SECTION - CS8

PROJECT NO.	CONTROL	REV.	of	FIGURE
19135652	002	A		B-8



BH # POST LEGEND

Baseline Offset (0 m)
Material Name MUDSTONE
Geology Unit 1 A
Depth and Elevation at EOH

MATERIAL GRAPHIC - Historic holes

FILL
SILTSTONE
TOPSOIL
Sandy CLAY
CLAY
Clayey GRAVEL
DOLERITE
SHALE
WATER LEVEL
EG-SURVEY_2019-12-04_FULL_SIMPLIFIED_1M

MATERIAL GRAPHIC - 2020 INVESTIGATION

FILL Material
Very Low and Low strength Siltstone bedrock
Very Stiff Clay
Medium strength Siltstone bedrock
Very low and low strength Dolerite
Very Low Strength Volcanic Breccia
Medium strength or better Dolerite
Medium Strength Volcanic Breccia

GROUND UNIT

1a - Fill
1b - Controlled Fill
1c - Landfill Waste
2 - Residual
3a - Very Low and Low Igneous Bedrock
3b - Medium Strength Igneous Bedrock or better
4a - Very Low and Low Sedimentary Bedrock
4b - Medium Strength Sedimentary Bedrock or better
5 - Proposed MSE Retaining Wall by Golder

0 25 50
1:1,000 VERTICAL METRES
0 25 50
1:1,000 HORIZONTAL METRES

H 1:1000
0 10 20 30 40 50m
V 1:1000
0 10 20 30 40 50m
Coord.System: Refer site plan Height Datum: AHD

CLIENT
Enviroguard Pty Ltd
CONSULTANT
YYYY-MM-DD
PREPARED FY
DESIGN FY
REVIEW
APPROVED

PROJECT
Proposed MSE Retaining Wall
85-87 Quarry Road, Erskine Park NSW 2759
TITLE
Inferred Subsurface Section Ch800

PROJECT No.	CONTROL	Rev.	FIGURE
1935652a1	2	A	xxx

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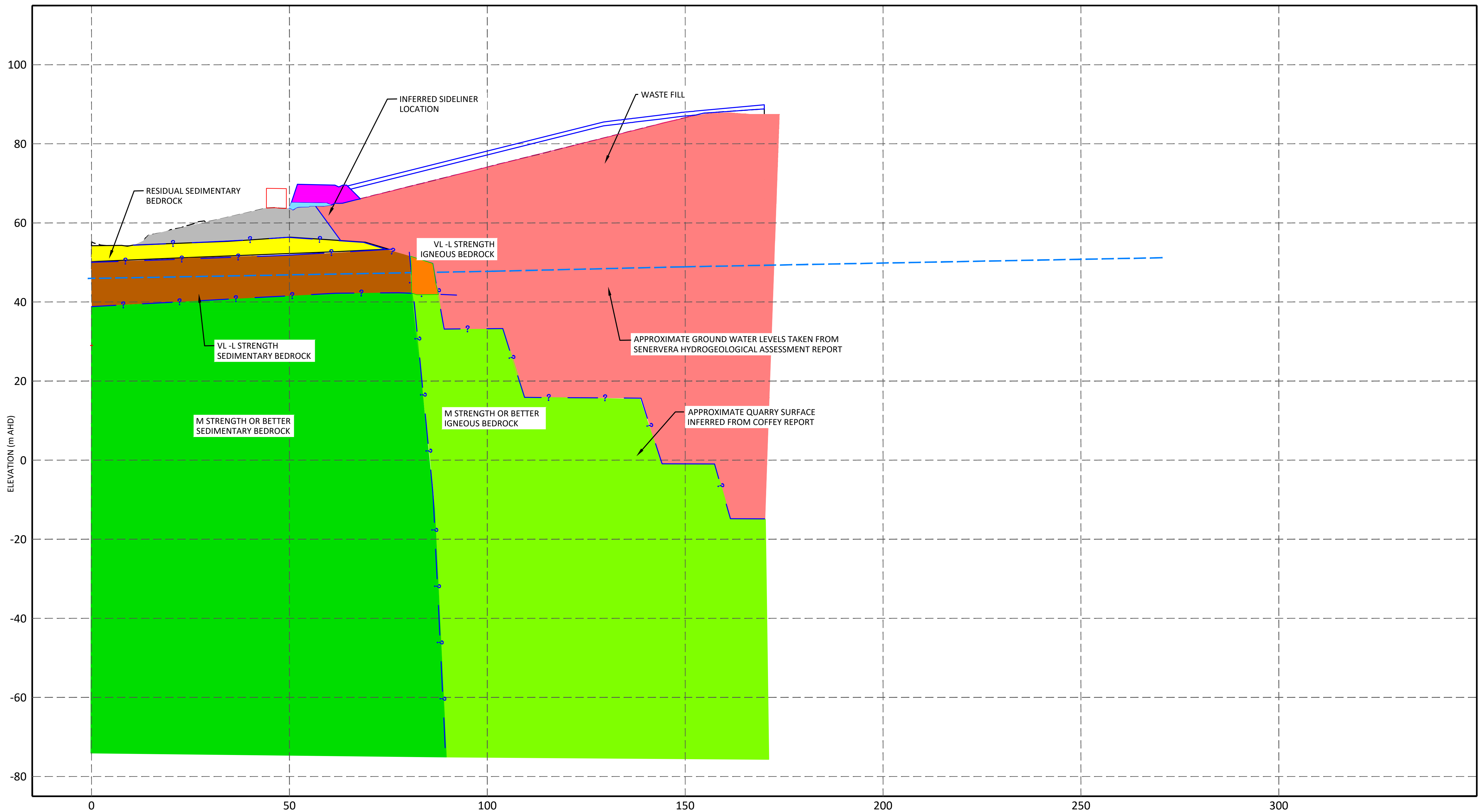


POST LEGEND

BH #
Baseline Offset
Material Name
(0 m)
Geology Unit 1
MUDSTONE
A
Depth and Elevation at EOH

MATERIAL GRAPHIC - Historic holes

FILL, SILTSTONE, TOPSOIL, Sandy CLAY, CLAY, Clayey GRAVEL, DOLERITE, SHALE

WATER LEVEL

EG-SURVEY_2019-12-04_FULL_SIMPLIFIED_1M

MATERIAL GRAPHIC - 2020 INVESTIGATION

- FILL Material
- Very Stiff Clay
- Very low and low strength Dolerite
- Medium strength or better Dolerite
- Very Low and Low strength Siltstone bedrock
- Medium strength Siltstone bedrock
- Very Low Strength Volcanic Breccia
- Medium Strength Volcanic Breccia

GROUND UNIT

- 1a - Fill
- 1b - Controlled Fill
- 1c - Landfill Waste
- 2 - Residual
- 3a - Very Low and Low Igneous Bedrock
- 3b - Medium Strength Igneous Bedrock or better
- 4a - Very Low and Low Sedimentary Bedrock
- 4b - Medium Strength Sedimentary Bedrock or better
- 5 - Proposed MSE Retaining Wall by Golder

0 25 50
1:1,000 VERTICAL METRES

0 25 50
1:1,000 HORIZONTAL METRES

H 1:1000
0 10 20 30 40 50m
V 1:1000
0 10 20 30 40 50m
Coord.System: Refer site plan Height Datum: AHD

CLIENT	Enviroguard Pty Ltd
CONSULTANT	
YYYY-MM-DD	
PREPARED	FY
DESIGN	FY
REVIEW	
APPROVED	
PROJECT	Proposed MSE Retaining Wall 85-87 Quarry Road, Erskine Park NSW 2759
TITLE	Inferred Subsurface Section Ch850
PROJECT No.	1935652a1
CONTROL	2
Rev.	A
FIGURE	xxx

GAP 8_16.8 LIB.GLB Fence FENCE A3L NO PLAN 1935652-MASTER-NEWSECTIONS.GPJ <<DrawingFile>> 09/03/2020 11:21 8.30.004 Datgel Tools

APPENDIX B

Geogrid Datasheet

Global Synthetics
Australian Company – Global Expertise

WOVEN POLYESTER HIGH PERFORMANCE GEOGRID ACEGrid® the proven choice for:

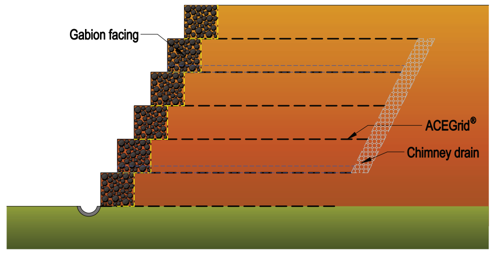
LONG TERM SOIL REINFORCEMENT IN APPLICATIONS OF

- STEEPENED REINFORCED SLOPES
- RETAINING WALLS
- VENEER REINFORCEMENT FOR LANDFILLS
- SOIL REINFORCEMENT OVER PILED FOUNDATIONS
- SUPPORT OVER VOIDS
- CAPPING OF TAILINGS PONDS
- BASAL REINFORCEMENT OF SOFT SOILS

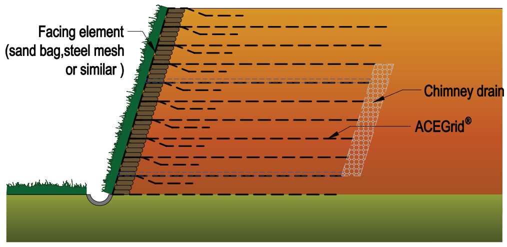
and other applications where soil will benefit from the inclusion of a tensile element for additional load carrying capacity.

Significant ACEGrid® soil reinforced structure for highway reconstruction works.

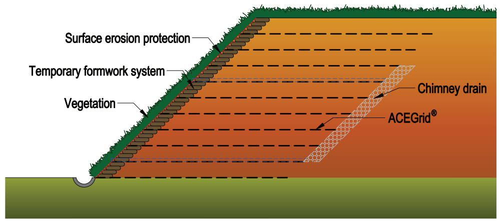
Global Synthetics proudly brings you ACEGrid®



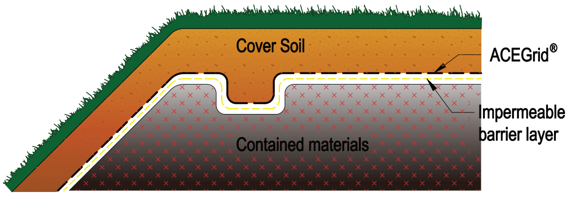
Retaining wall with ACEGrid® - Gabion facing



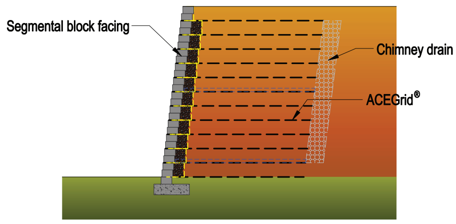
Reinforced slope with ACEGrid® - Wrapped facing
(Maximum slope angle is 70°)



Reinforced slope with ACEGrid®
(Maximum slope angle is 45°)



Veneer application in a landfill with ACEGrid®



Retaining wall with ACEGrid® - Segmental block facing

ACEGrid® is an engineered woven geogrid that has exceptionally high strength characteristics at low levels of strain. The product is additionally coated with a polymer that provides high resistance to degradation in soil environments as well as providing additional UV and mechanical damage protection to the fibres. The ACEGrid® geogrids may be constructed with tensile strengths up to 900 kN/m. Strains generated at ultimate tensile strength are typically less than 10%. Being composed of high tenacity polyester fibres they deliver low creep strains when subject to high tensile loads. Creep strains of less than 1% at design loads of 40% of the initial characteristic tensile strength at 120 year design life, are obtained.

ACEGrid® is made from high tenacity polyester fibres that have a demonstrated history of performance both here in Australia and around the world. ACEGrid® has the ability to carry significant loads imposed upon the product from a range of soil reinforcement applications including slopes and walls and provide innovative solutions in applications that will benefit from the inclusion of a tensile element within the soil structure.

ACEGrid® is suitable for use in short term as well as very long term ground support applications with design lives in excess of 120 years. The product may be manufactured for site specific requirements. The rolls are available in wide widths up to 5.0m to minimise wastage from overlap requirements.

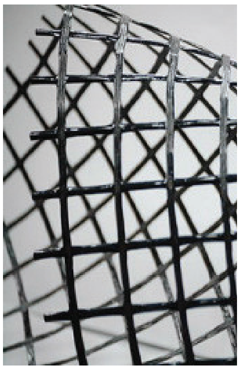
The Challenge

Road and geotechnical design engineers are facing an increasing challenge throughout Australia of building roads, retaining structures and related structures with a wide range of soil types and over problematic soil conditions. Relatively low shear capacity soils benefit significantly with the inclusion of an appropriate geosynthetic. ACEGrid® provides engineers with a cost effective, proven alternative to the importation of expensive fills and allows structures to be built relatively economically than would normally be possible.

Meeting the Challenge

Quality

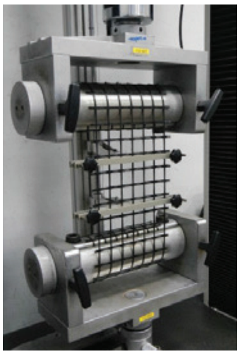
Ace Geosynthetics have a commitment to using the latest technology in weaving processes that delivers highest strengths possible at low soil compatible strains. Quality control within the manufacturing process ensures consistency of manufacture at all times. Ace Geosynthetics hold a number of internationally recognised accreditation approvals for their manufacturing processes.



Materials

Ace Geosynthetics use the best available polymers and the highest tenacity yarn to make the ACEGrid® product. Of importance is the choice of polymer used to make the ACEGrid® structural geogrid. Polyester polymer, in such applications of structural reinforcement, is the most resistant to loss of strength through creep effects over very long periods of time. The use of low carboxyl end group, high molecular weight, base polymer, has been proven to withstand the effects of hydrolysis and subsequent loss of strength in alkaline environments. Polyester polymer is the least susceptible to long term temperature effects.

Testing



Ace Geosynthetics have a commitment to fully understand the short term and long term behavior of their product. Significant internal and external testing has been carried out at some of the world's most well recognised research and test facilities to independently verify product performance when subjected to physical damage, chemical resistance, load and temperature effects. Both real time and accelerated test methods have been performed to ensure that the ACEGrid® product performance is understood over design lives in excess of 120 years.

History

Ace Geosynthetics high performance geogrid has been used for years on many Australian soil reinforcement projects with outstanding success. ACEGrid® geogrid is stocked locally with larger requirements made to order with speedy lead times to suit construction requirements. ACEGrid® can be custom manufactured to suit specific project demands such as roll width or length. ACEGrid® product is supported in Australia, New Zealand and the South Pacific by Global Synthetics engineers. ACEGrid® geogrid has been approved for use under the NSW RMS R57 Specification process. Similarly this approval is accepted by the Queensland Department of Transport and Main Roads. International approvals are held with the product accredited with BBA (British Board of Agreement) for applications of basal and slope reinforcement. Product evaluations have been carried out in the USA through the AASHTO- NTPEP programme.

The use of ACEGrid® as a front wrapped reinforcement treatment and after completion showing the vegetated structure.

1. General

The ACEGrid® high performance geogrid range, are engineered products for applications of short term and long soil reinforcement. The product is woven with strength in both the roll length direction (commonly called the machine direction-MD) and with strength manufactured in the cross roll direction (commonly called the cross direction-CD). Generally the strength of the product will be dominant in one direction of the roll (normally the MD) with sufficient strength in the other direction of the fabric (normally CD) such that the fibres are dimensionally stable and the roll may be easily deployed.

In applications of soil reinforcement the use of ACEGrid® engineered geogrids allows significant tensile strength to be imparted to soils. Soils are very weak in tension. The use of soil reinforcement techniques has proven to be a very cost effective method of construction. ACEGrid® engineered geogrids are manufactured from high tenacity polyester (PET) fibres with high molecular weights and low carboxyl end groups such that the product is suitable for use in normally occurring soil types, for design lives in excess of 120 years. ACEGrid® high performance geogrids are available in a range of strengths from 40kN/m to 900 kN/m tensile strength.

2. Load assessment of ACEGrid®

The use of ACEGrid® high performance geogrid, in long term soil reinforcement applications, requires an assessment of the long term load carrying capabilities of the product.

The procedure adopted for ACEGrid® high performance geogrid follows a partial factor approach that accounts for influences of time, temperature, environment and load.

The assessment procedures for ACEGrid® geogrids are compatible with US Federal Highway of Administration (FHWA), British Code of Practice BS8006:2010, EN ISO 20432:2007 and Australian Standard AS 4678. Australian Standards Handbook HB154- Geosynthetics-Guidelines on Durability may be read in conjunction with this data sheet. There may be additional considerations in some design situations such as the need to satisfy appropriate connection criteria. Additional guidance is given in Section 9 of this document, for further reference.

The following procedure is an accepted method for determining the long term design strength of the reinforcement at differing design lives.

$$T_d = \frac{T_c}{f_c \cdot f_d \cdot f_e \cdot f_{m11} \cdot f_{m12}}$$

where,

T_d is the long term design strength of the reinforcement at the required design life.

T_c is the characteristic short term tensile strength of the reinforcement.

f_c is the partial factor relating to creep effects over the required design life of the reinforcement.

f_d is the partial factor relating to damage effects on the reinforcement.

f_e is the partial factor relating to environmental effects on the reinforcement.

f_{m11} is the partial factor relating to consistency of manufacture of the reinforcement.

f_{m12} is the partial factor relating to extrapolation of test data.

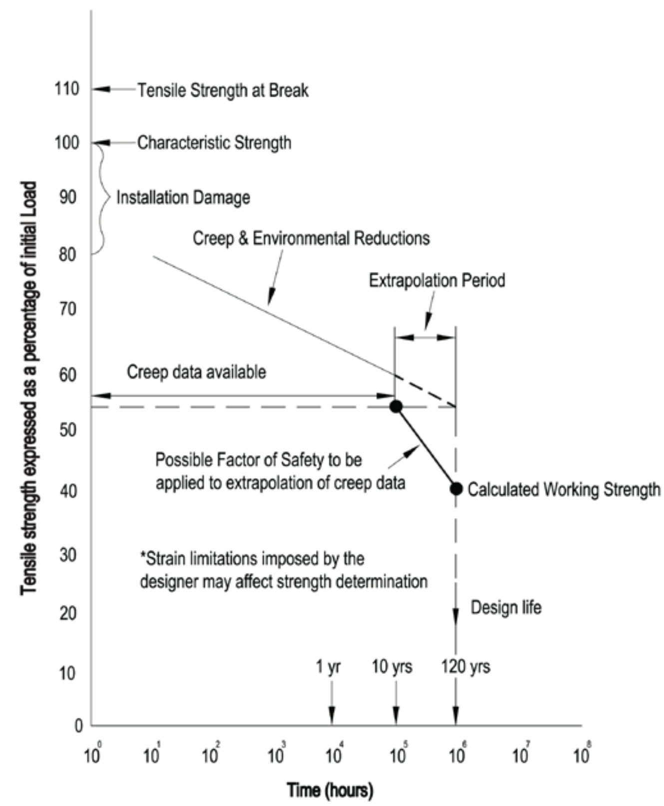


Fig. 1 Partial Factor Reductions to be considered in long term strength derivation

3. Partial factor relating to creep, f_c

In any assessment of the partial factor for creep, f_c, the creep rupture characteristics of the reinforcement must be known.

Significant independent testing has been carried out using both conventional creep rupture testing under long term loading conditions as well as accelerated test methods. From Fig.2 the values of fc can be obtained for different design lives. For example, at 60 years design life the ACEGrid® geogrid shows a 71 % strength retention which equates to a partial factor of f_c= 1.41. The published value of f_c for a 120 year design life is 1.45.

ACEGrid®, being composed of high tenacity polyester fibres exhibit very low creep strains even at high tensile load levels. Creep strains of less than 1% over a 120 year design life at a design load of 40% of initial tensile strength are obtained.

The treatment of long term total and creep strains is referenced in Section 8 of this document. The reader is encouraged to carefully consider strain requirements and the effects on the allowable design strength of the geogrid.

Manufacturers of these products must be able to demonstrate creep testing of the manufactured product rather than simple creep testing of the yarn only.

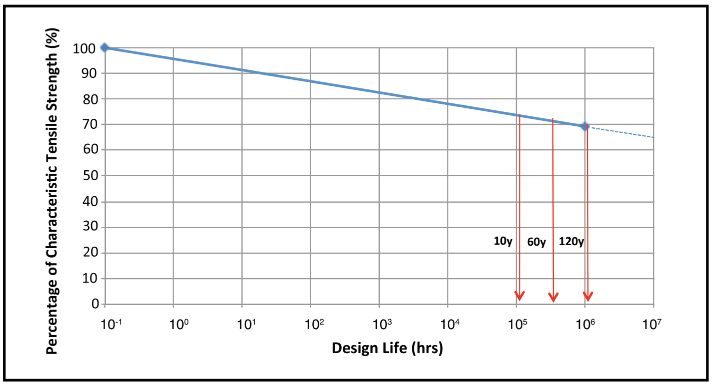


Fig. 2 Creep Rupture Curve
ACEGrid® Geogrid

4. Partial factor relating to installation damage, f_d

The magnitude of damage, f_d, imposed upon the ACEGrid® geogrid is a function of the structure of the reinforcement, the aggressiveness of the fill placed either side of the reinforcement, the method of placement of the fill and the level of compaction performed.

The damage factors used for ACEGrid® geogrid are derived from independent field and large scale laboratory tests. Values of f_d for ACEGrid® geogrid placed in varying soil environments may be obtained from Global Synthetics.

5. Partial factor relating to environmental effects, f_e

The magnitude of the partial factor, f_e, is a function of the polymers used as well as the structure of the reinforcement used. ACEGrid® geogrids are manufactured from virgin, high tenacity polyester fibres. Polyester fibres have over many years demonstrated high resistance to strength loss when buried in soil environments for long periods of time. The ACEGrid® geogrid range is made of high molecular weight, low carboxyl end group fibres that are very stable in a range of pH environments. A range of partial factors, f_d, are given in the data sheet for a range of design lives.

6. Partial factor relating to consistency of manufacture, f_{m11}

ACEGrid® geogrids are manufactured according to independently audited Quality Control and Assurance standards to meet a confidence level of 95% of the published tensile strengths.

The partial factor adopted for ACEGrid® geogrid for consistency of manufacture, **f_{m11}, has a value of 1.0 for design lives up to 120 years** in accordance with BS 8006: 2010.

7. Partial factor relating to extrapolation of creep data, f_{m12}

ACEGrid® geogrids have been extensively tested both in real time creep testing and using time temperature shifting curves to account for long period of time. Both methods are carried out using ASTM and ISO test protocols. The examination of creep data and the suitability of use to extrapolate such data is referenced to BS8006:2010 and EN ISO 20432:2007. The partial factor based on the validity of the statistical envelope between real time testing and time, temperature shifting methods (SIM) **allows f_{m12} to be assigned a value of 1.0 for design lives up to 120 years.**

8. Tensile strength strain properties

8.1 Short term tensile strength and strain with time = 0 hours

The short term tensile strength relationship to strain of ACEGrid® geogrid is shown as a master curve in Fig.3. The graph shows, as the "y" ordinate, the strength of the ACEGrid® geogrid as a percentage of the characteristic short term tensile strength. Thus one master curve may be used to represent all ACEGrid® grades available by converting the percentage values into actual strength values for individual grades. It is important to note that a relationship exists between strength, strain and time for all geosynthetic reinforcement products.

Isochronous stress curves (refer to Fig. 4) must be used to calculate the long term design strength that will limit design strain for a given design life. Some manufacturers do not provide such information on their data sheets which may lead to an over estimation of achievable geogrid strength for a long term design strain requirement.

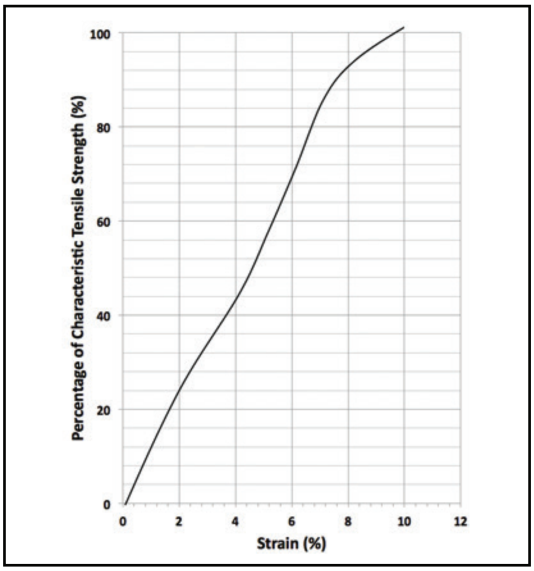


Fig. 3 Short term tensile strength-strain relationship for ACEGrid® Geogrid.

8.2 Long term tensile strength and strain with time dependency to 120 years

The long term tensile strength relationship to **strain** with the influence of **time dependency** for ACEGrid® geogrid is shown as a master curve in Fig.4. The graph shows, as the "y" ordinate, the strength of the ACEGrid® geogrid as a percentage of the characteristic tensile strength. The "x" axis is the strain component that is appropriate to long term loading conditions. This is theoretically any time greater than t=0 mins. Superimposed upon the curves is the time relationship. A number of long term design lives have been plotted that allow the designer to limit the load within the ACEGrid® geogrid such that a design strain limit is not exceeded for the structure to be constructed. Thus one master curve may be used to represent all ACEGrid® grades by converting the percentage values into actual strength values for individual grades.

Shown at Fig.5 are the components of strain that are necessary to understand when specifying any structural soil reinforcement geosynthetic.

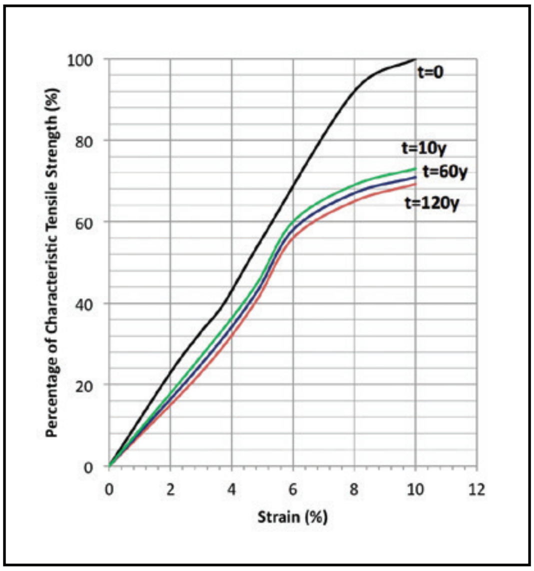


Fig. 4 Long term tensile strength-strain- time relationship for ACEGrid® Geogrid Isochronous curves.

9.0 Other Design Considerations and Benefits

9.1 Designing with Gabion Facing and ACEGrid®

A comprehensive design manual – "Link Gabions and Mattresses" details the use of gabions as the facing element in combination with ACEGrid® soil reinforcement techniques. Contact Global Synthetics.

9.2 Segmental Block Facing and ACEGrid®

Software is available for a range of proprietary facing options such as Keystone® and Anchor® Wall Systems. Contact Global Synthetics.

9.3 RMS (NSW) and TMR (QLD) Approval and ACEGrid®

Full approval details may be downloaded from the RMS (ex RTA NSW) website. Specifications RMS R57 and TMR 11.06 apply.

9.4 BBA Certification for Applications of Slopes and Basal Reinforcement

Full documentation available for design to BBA certification-contact Global Synthetics.

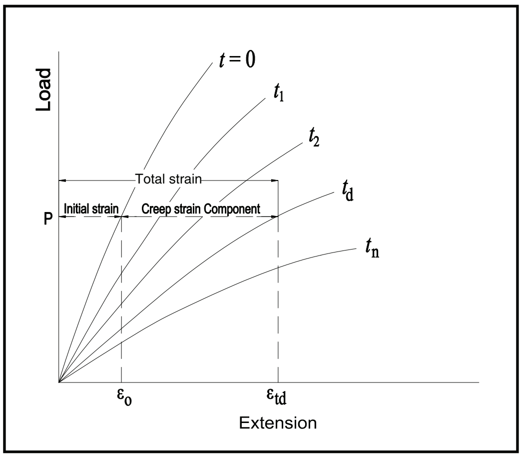


Fig. 5 Method of determining the various components of strain.

PROPERTIES OF ACEGrid® HIGH PERFORMANCE UNIAXIAL GEOGRID

PROPERTY		UNITS	GG40	GG60	GG80	GG100	GG120	GG150	GG200
MECHANICAL PROPERTIES									
Mean ultimate tensile strength ISO 10319	MD	kN/m	45	70	90	110	130	165	219
Characteristic ultimate tensile strength ISO 10319	MD	kN/m	42	65	84	106	121	157	206
Strain at short term strength ISO 10319	MD	%	10	10	10	10	10	10	10
Partial factor - creep rupture - f_c									
at 10 years design life			1.37	1.37	1.37	1.37	1.37	1.37	1.37
at 60 years design life			1.41	1.41	1.41	1.41	1.41	1.41	1.41
at 120 years design life			1.45	1.45	1.45	1.45	1.45	1.45	1.45
Creep limited strength									
at 10 years design life	MD	kN/m	30.7	47.4	61.3	77.4	88.3	114.6	150.4
at 60 years design life	MD	kN/m	29.8	46.1	59.6	75.2	85.8	111.3	146.1
at 120 years design life	MD	kN/m	29.0	44.8	57.9	73.1	83.4	108.3	142.1
Partial factor - construction damage - f_d in coarse gravel less than 50mm			1.12	1.1	1.1	1.1	1.1	1.1	1.05
Partial factor - environmental effects in soil environment 2 < soil pH < 10 - f_e									
not exceeding 10 years design life			1.0	1.0	1.0	1.0	1.0	1.0	1.0
at 60 years design life			1.03	1.03	1.03	1.03	1.03	1.03	1.03
at 120 years design life			1.05	1.05	1.05	1.05	1.05	1.05	1.05
Long term design strengths - t_d in coarse gravel less than 50mm									
at 10 years design life	MD	kN/m	27	43	56	70	80	104	143
at 60 years design life	MD	kN/m	26	41	53	66	76	98	135
at 120 years design life	MD	kN/m	25	39	50	63	72	94	129
Nominal roll width		m	4	4	4	4	4	4	4
Nominal roll length		m	50	50	50	50	50	50	50
Nominal roll mass		kg	55	60	65	80	95	108	140

NOTE:

1. The characteristic short term strength is the statistical 95% confidence limit.
2. All creep testing has been carried out at 20^0 C.
3. Roll widths to 5m are available.
4. The cross direction (C.D.) strength is 30kN/m
5. Long term design strength are characteristic values.

afaq PAS 2050 Greenhouse Gas Emissions

afaq ISO 9001 Qualité AFNOR CERTIFICATION

Long term design strength is determined by compounding the reduction factors for creep, installation, and environmental effects. ACEGrid® is made from polyester yarn with high molecular weight, Mn > 30,000 and a Carboxyl End Group, CEG of <14 mmol/kg. ACEGrid® is resistant to all naturally occurring soil acids and alkalines, pH 2 - 10. Values quoted are statistically 95% confident and are described as the characteristic value. Testing on the product is carried out in a credited testing laboratories within factory and at third party accredited testing laboratories and institutions.

Design assistance provided

Contact Global Synthetics for assistance using the ACEGrid® high performance geogrids reinforcement solution.

More about ACE Geosynthetics

ACE Geosynthetics are a specialist manufacturer of a wide range of geosynthetic products including ACETex® PET structural geotextiles and ACEGrid® soil reinforcement geogrids. ACE Geosynthetics are fully accredited to international quality standards and a commitment to their customers worldwide. ACE Geosynthetics are a market innovator and are continually striving in their product development.

More about Global Synthetics

Global Synthetics is a 100% Australian-owned company, proud to offer a complete range of high-quality geosynthetic products backed by over 100 years of combined staff experience in the industry. We have supplied products to some of the largest recent infrastructure works in Australia. Global Synthetics provides major benefits to any geotechnical engineering project with the right products and our technical expertise.

Stay on top of the latest Global Synthetics information and visit our website at **www.globalsynthetics.com.au**

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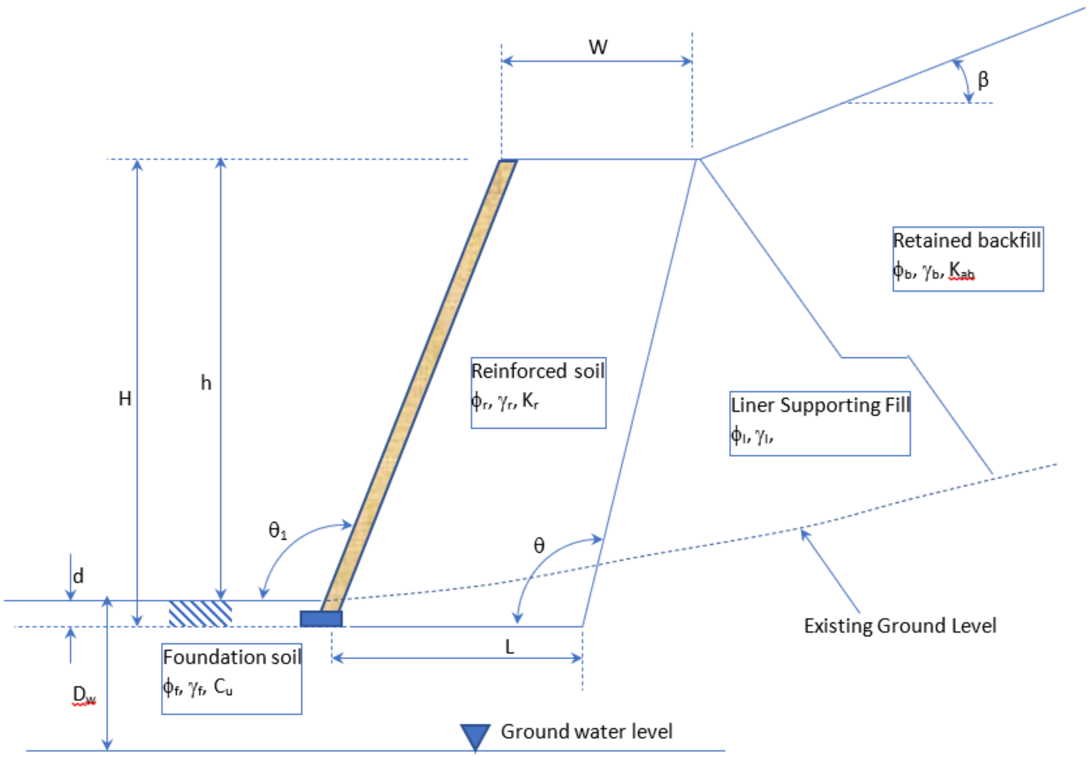
August 2018

APPENDIX C

External Stability

DESIGN OF MECHNICALLY STABILISED EARTH (MSE) WALLS



References:

1. Design and Construction of Mechanically Stabilized Earth Walls and Reinforced Soil Slopes – Volume I, Publication No. FHWA-NHI-10-024, Federal Highway Administration FHWA GEC 011 – Volume I.

2. AASHTO LRFD Bridge Design Specifications, Seventh. Edition, 2014.

3. AS 4678-2002 - Earth-retaining structures.

4. AS1170.4-1993 - Minimum Design Loads on Structures, Part 4: Earthquake Loads.

1. External Stability Design Paramaters

Wall location Ch200

1.1. Wall Geometry

Wall Height above the existing ground level, h $h := 11.83\ m$

Embedment depth $d := 3.0\ m$

Top width of wall $W := 12\ m$

Slope of backfill behind wall $\beta := 14\ deg$

Slope in Front of Structure	Minimum Embedment Depth
Horizontal	H/20.0
3.0H : 1.0V	H/10.0
2.0H : 1.0V	H/7.0
1.5H : 1.0V	H/5.0

Face inclination from horizontal $\theta_1 := 108.4\ deg$ Wall slope: 3V:1H

Depth of groundwater below the existing ground level $D_w := 11.0\ m$

1.2. Reinforced Soil Block Parameters

Eff. Friction of reinforced block $\phi_r := 32\ deg$

Unit weight of reinforced block $\gamma_r := 20\ \frac{kN}{m^3}$

1.3. Retained backfill Parameters

Note

1. Retained backfill friction angle was assumed to be friction angle of the landfill, cohesion was assumed zero.
2. Unit weight of retained backfill was assumed to be equal to unit weight of the liner supporing fill.

Eff. Frcition of retained backfill $\phi_b := 27\ deg$

Unit weight of backfill $\gamma_b := 20\ \frac{kN}{m^3}$

1.4. Foundation Soil Parameters

Drained friction angle of foundation soil $\phi_f := 28\ deg$

Undrained shear strength of foundation soil $C_u := 100\ kPa$ Enter "NA", if Cu is not applicable to foundation soil

Unit weight of foundation soil $\gamma_f := 18\ \frac{kN}{m^3}$

Note

1. For the estimation of top width of wall, width of draingae chimney is reduced.
2. facing was considered as part of the MSE wall as geogrid was used to wrap back.

Wall Height $H := h + d = 14.83\ m$

Top width of MSE wall $w := W - 1.0\ m$

Ratio between MSE wall height and reinforcement length, L/H $\lambda := 0.7$

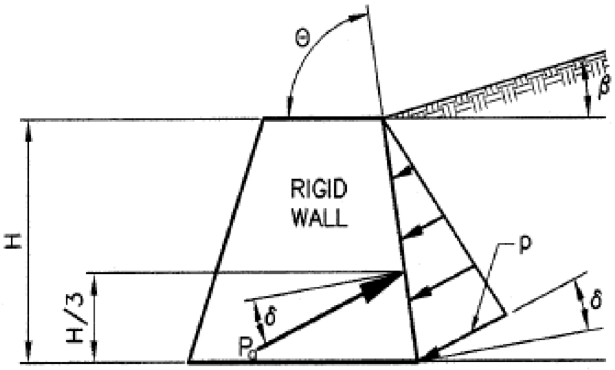
Min. bottom width of reinforced MSE wall $L := \text{if}\left(\left(w + \frac{h}{3}\right) \le \lambda \cdot H, w + \frac{h}{3}, \lambda \cdot H\right) = 10.381\ m$

2. Loads

Angle of fric between retained backfill and rein. Soil $\quad \delta := \beta \quad$ Assumed Equal to β

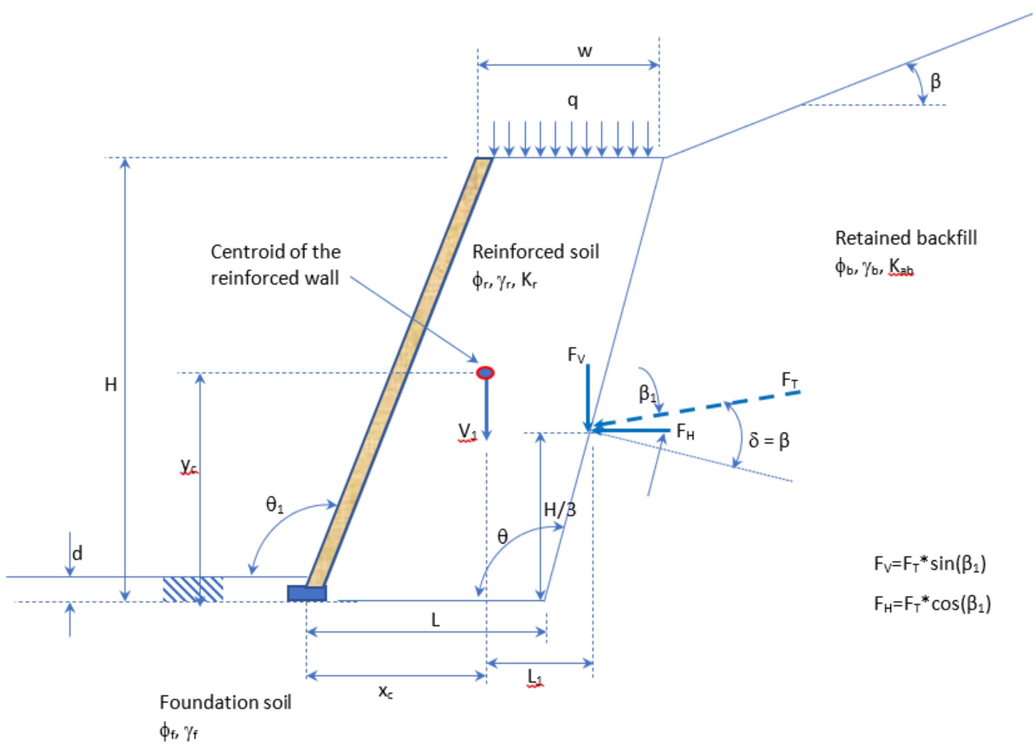
Wall batter $\quad \theta := 180.\ deg - \operatorname{atan}\left(\dfrac{H}{\dfrac{H}{\tan(180.\ deg - \theta_1)} + w - L}\right)$

$$\theta := \text{if}\left(\theta \geq 180.\ deg, \theta - 180\ deg, \theta\right)$$

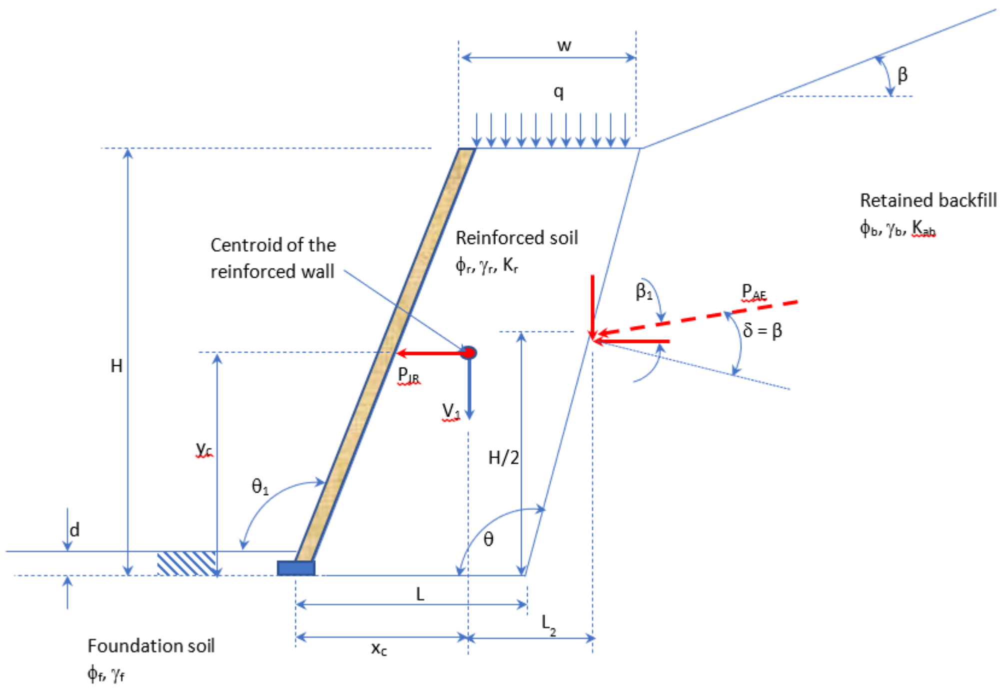


2.1. Load Combinations

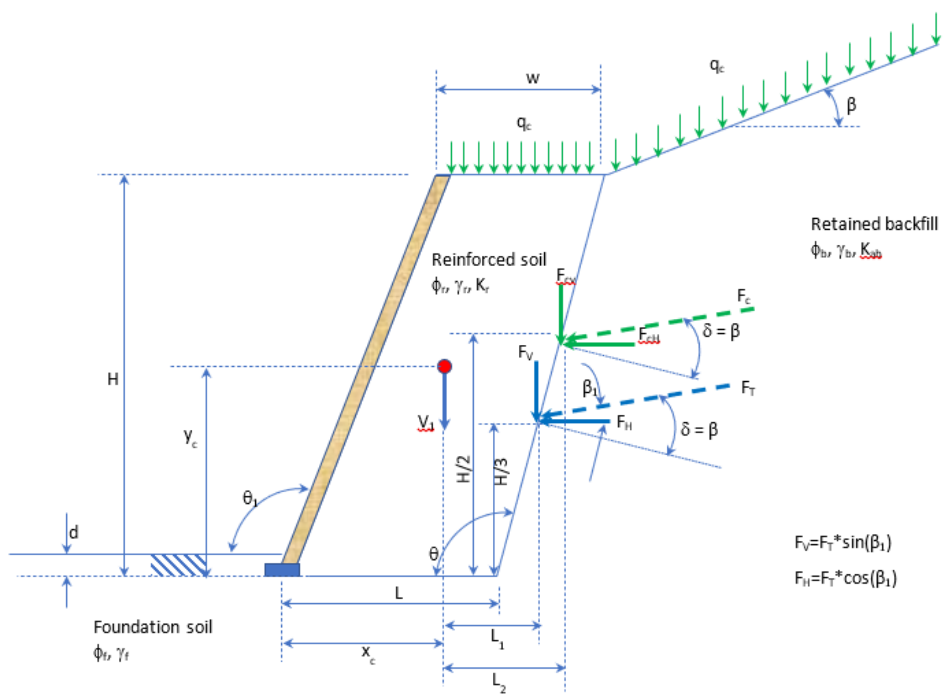
Load Case 1 (Strength): $\quad$ "$\gamma_p \cdot EV$" + "$\gamma_p \cdot EH$" + "$\gamma_{LS} \cdot LS$"



Load Case 2 (Earth quake): $"\gamma_p \cdot EV" + "\gamma_p \cdot EH" + "\gamma_{EQ} \cdot LS" + "1.0 \cdot EQ"$



Load Case 3 (Construction): $"\gamma_p \cdot EV" + "\gamma_p \cdot EH" + "\gamma_{LS} \cdot LS"$



Load Case 4 (Vehicular Impact):

Vehicular impact on traffic barrier tends to affect only the internal stability of MSE walls (reinforcement). So, the external stability assessment has not been carried for this load case.

2.2. Applicable loads

Horizontal earth pressure (EH) Vertcial earth pressure (EV) Earthquake Load (EQ)

Live load (traffic) surcharge (LS) $q_L := 20\ kPa$

Live load (construction) surcharge (LS) $q_c := 10\ kPa$

$$\Gamma := \left(1 + \sqrt{\frac{\sin(\phi_b + \delta) \cdot \sin(\phi_b - \beta)}{\sin(\theta - \delta) \cdot \sin(\theta + \beta)}}\right)^2$$

Cofficient of active earth pressure

$$K_{ab} := \frac{\left(\sin(\theta + \phi_b)\right)^2}{\Gamma \cdot (\sin(\theta))^2 \cdot \sin(\theta - \delta)}$$

Assumption: live loads due to traffic and construction have been considered as a surcharge in the external stability estimation.

2.3. Load and Resistance factors

Maximum horizontal earth pressure factor $\gamma_{EHmax} := 1.50$

Minimum horizontal earth pressure factor $\gamma_{EHmin} := 0.90$

Minimum vertical earth pressure factor $\gamma_{EVmin} := 1.00$

Maximum vertcial earth pressure factor $\gamma_{EVmax} := 1.35$

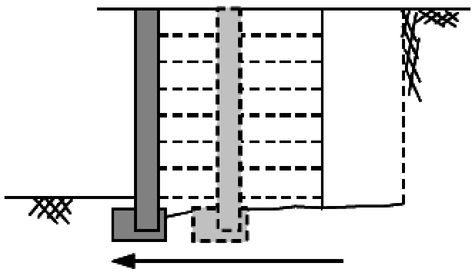
Live load factor $\gamma_{LS} := 1.75$

Load factor for live load for load case 2 $\gamma_{EQ} := 1.00$

Resistance factor for shear resistance between soil and foundation $\phi_\tau := 1.00$

Resistance factor for bearing $\phi := 0.65$

3. Sliding Stability



3.1. Load Case 1

Note

1. Inclination of retained backfill force resultant to normal of the back wall face (δ) is assumed to be β ($\delta=\beta$).
2. Inclination of retained backfill force resultant to horizontal (β_1) will be function of back wall inclination θ and δ.
3. β_1 is estimated for three different cases
   a) $\theta \geq 90\ deg \wedge (\theta - 90.\ deg) \geq \delta$
   b) $\theta \geq 90\ deg \wedge (\theta - 90.\ deg) < \delta$
   C) $\theta < 90\ deg$

$\beta_a := (\theta - 90\ deg) - \delta$ $\qquad$ $\beta_b := \delta - (\theta - 90\ deg)$ $\qquad$ $\beta_c := 90\ deg - \theta + \delta$

$\beta_1 := \text{if}\left(\theta \geq 90\ deg \wedge (\theta - 90.\ deg) \geq \delta, \beta_a, \text{if}\left(\theta \geq 90\ deg \wedge (\theta - 90.\ deg) < \delta, \beta_b, \beta_c\right)\right)$

Retained backfill force resultant per unit width $\qquad$ $F_T := \frac{1}{2} K_{ab} \cdot \gamma_b \cdot H^2$

Horizontal driving force per unit width $\qquad$ $F_H := F_T \cdot \cos(\beta_1)$

Vertical force per unit width $\qquad$ $F_V := F_T \cdot \sin(\beta_1)$

Factored horizontal driving force per unit width $\qquad$ $P_{d1} := \gamma_{EHmax} \cdot F_H$

Weight of reinforced block $\qquad$ $V_1 := \gamma_r \cdot H \cdot \frac{(L + w)}{2}$

Minimum soil friction angle $\qquad$ $\mu := min(\tan(\phi_r), \tan(\phi_f))$

Notes:
1. Live load is excluded as it increases the sliding stability.

Sliding resistance between rein. soil and foundation - Drained

$$R_{\tau\_drained} := \left(\gamma_{EVmin} \cdot V_1 + \gamma_{EHmin} \cdot F_V\right) \cdot \mu$$

Sliding resistance between rein. soil and foundation -Undrained

$$R_{\tau\_undrained} := \textbf{if}\left(C_u = \text{“NA”}, \text{“NA”}, C_u \cdot L\right)$$

Note:
If ground water is below the foundation level (Dw-d > 2.0 m below foundation level), sliding resistance (undrained) was not considered in the external stability.

$$R_{\tau\_undrained} := \textbf{if}\left(D_w - d \geq 2.0\ m, \text{“NA”}, R_{\tau\_undrained}\right)$$

Factored Sliding resistance

$$R_r := \textbf{if}\left(R_{\tau\_undrained} = \text{“NA”}, \phi_\tau \cdot R_{\tau\_drained}, \mathit{min}\left(\phi_\tau \cdot R_{\tau\_drained}, \phi_\tau \cdot R_{\tau\_undrained}\right)\right)$$

Sliding Check

$$\textbf{SlidingCheckCase1} := \textbf{if}\left(R_r \geq P_{d1}, \text{“OK”}, \text{“Not OK”}\right)$$

$$\textbf{SlidingCheckCase1} = \text{“OK”}$$

3.2. Load Case 2

AS4678 is adopted for the estimation of acceleration coefficient.

Live load (traffic) has been assumed to an equivalent live load surcharge of 20 kPa as it increases the horizontal force.

Peak ground acceleration coefficient $a := 0.08$

Horizontal coefficient of acceleration $a_h := 0.5 \cdot a$

Horizontal coefficient of acceleration with amplification of motion

$$a_{mh} := \left(1.45 - a_h\right) \cdot a_h \qquad k_v := 0.00$$

$$\xi := \operatorname{atan}\left(\frac{a_{mh}}{1 - k_v}\right)$$

$$\delta_1 := \mathit{min}\left(\phi_r, \phi_b\right)$$

$$I := \beta$$

$$\chi := 90.\ deg$$

Mononobe-Okabe (M-O) formulation

$$K_{AE} := \frac{\cos\left(\phi_b - \xi - \chi + \theta\right)^2}{\cos(\xi) \cdot \cos(\chi - \theta)^2 \cdot \cos\left(\delta_1 + \chi - \theta + \xi\right) \cdot \left(1 + \sqrt{\frac{\sin\left(\phi_b + \delta_1\right) \cdot \sin\left(\phi_b - \xi - I\right)}{\cos\left(\delta_1 + \chi - \theta + \xi\right) \cdot \cos\left(I - \chi + \theta\right)}}\right)^2}$$

Total (static + dynamic) thrust $P_{AE} := 0.5 \cdot K_{AE} \cdot \gamma_b \cdot H^2$

Horizontal inertial force $P_{IR} := 0.5 \cdot \left(a_{mh} \cdot V_1 + \gamma_{EQ} \cdot q_L \cdot w \cdot a_{mh}\right)$

Total horizontal force $T_{HF} := P_{AE} \cdot \cos\left(\delta_1\right) + P_{IR}$

Notes:
1. Live load was considered as part of the reinforced soil mass

Sliding Check $\mathbf{SlidingCheckCase2} := \mathbf{if}\left(R_r \geq T_{HF}, \text{“OK”}, \text{“Not OK”}\right)$

$\mathbf{SlidingCheckCase2} = \text{“OK”}$

3.3. Load Case 3

Uniform construction surcharge resultant per unit width $F_C := K_{ab} \cdot q_c \cdot H$

Horizontal component of Fc $F_{CH} := F_C \cdot \cos\left(\beta_1\right)$

Vertical component of Fc $F_{CV} := F_C \cdot \sin\left(\beta_1\right)$

Factored horizontal driving force per unit width $P_{d3} := \gamma_{EHmax} \cdot F_H + \gamma_{LS} \cdot F_{CH}$

Weight of reinforced block $V_1 := \gamma_r \cdot H \cdot \frac{(L + w)}{2}$

Minimum soil friction angle $\mu := \mathit{min}\left(\tan\left(\phi_r\right), \tan\left(\phi_f\right)\right)$

Notes:
1. Live load surcharge immidiately above the reinforced fill is excluded as it increases the sliding stability.

Sliding resistance between rein. soil and foundation - Drained $R_{\tau\_drained} := \left(\gamma_{EVmin} \cdot V_1 + \gamma_{EHmin} \cdot F_V + \gamma_{EHmin} \cdot F_{CV}\right) \cdot \mu$

Sliding resistance between rein. soil and foundation -Undrained $R_{\tau\_undrained\_con} := \mathbf{if}\left(C_u = \text{“NA”}, \text{“NA”}, C_u \cdot L\right)$

Note:
If ground water is below the foundation level (if Dw-d > 2.0 m below foundation level), sliding resistance (undrained) was not considered in the external stability.

$$R_{\tau\_undrained\_con} := \text{if}\left(D_w - d \geq 2.0\ m, \text{“NA”}, R_{\tau\_undrained}\right)$$

Factored Sliding resistance

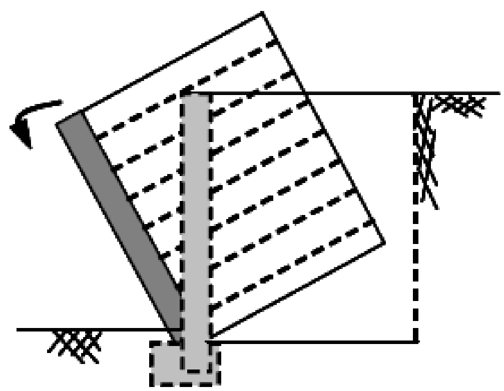
$$R_r := \text{if}\left(R_{\tau\_undrained\_con} = \text{“NA”}, \phi_\tau \cdot R_{\tau\_drained}, min\left(\phi_\tau \cdot R_{\tau\_drained}, \phi_\tau \cdot R_{\tau\_undrained\_con}\right)\right)$$

Sliding Check

$$\textbf{SlidingCheckCase3} := \text{if}\left(R_r \geq P_{d3}, \text{“OK”}, \text{“Not OK”}\right)$$

$$\textbf{SlidingCheckCase3} = \text{“OK”}$$

4. Rotational/Overturning Stability



4.1. Load Case 1

Notes:
1. Weight and width of the facing is neglected in this calculation
2. Traffic surcharge was ignored as it contribute to reduce the eccentricity
3. Moment is estimated about the middle of the bottom width of wall.

Estimation of Centeroid of the wall from the toe

$$l_1 := \frac{H}{\tan\left(180.\ deg - \theta_1\right)}$$

$$l_2 := L - l_1 - w = -5.552\ m$$

X distance to centeroid of the reforced wall from toe of the wall

$$x_c := \frac{\frac{{l_1}^2}{3} + w \cdot \left(l_1 + \frac{w}{2}\right) + \frac{1}{2} \cdot l_2 \cdot \left(l_1 + w + \frac{1}{3} \cdot l_2\right)}{\frac{(L + w)}{2}}$$

Y distance to centeroid of the reforced wall from toe of the wall

$$y_c := \frac{H}{3} \cdot \frac{(L + 2 \cdot w)}{L + w}$$

$$L_1 := \textbf{if}\left(l_2 \le 0, l_1 + w - x_c + \frac{2}{3} \cdot l_2, L - x_c - \frac{2}{3} \cdot l_2\right)$$

Eccentricity

$$e_1 := \frac{\gamma_{EHmax} \cdot F_H \cdot \frac{H}{3} - \gamma_{EHmax} \cdot F_V \cdot L_1}{\gamma_{EVmin} \cdot V_1 + \gamma_{EHmax} \cdot F_V}$$

Rotation Check

$$\textbf{RotationCheckCase1} := \textbf{if}\left(\frac{L}{4} \ge e_1, \text{“OK”}, \text{“Not OK”}\right)$$

$$\textbf{RotationCheckCase1} = \text{“OK”}$$

4.2. Load Case 2

$$L_2 := \textbf{if}\left(l_2 \le 0, l_1 + w - x_c + \frac{1}{2} \cdot l_2, L - x_c - \frac{1}{2} \cdot l_2\right)$$

Notes:

1. Weight and width of the facing is neglected in this calculation
2. Traffic surcharge was ignored as it contribute to reduce the eccentricity
3. Moment is estimated about the middle of the bottom width of wall.

Eccentricity

$$e_2 := \frac{P_{IR} \cdot y_c + P_{AE} \cdot \cos(\beta_1) \cdot 0.5 \cdot H - P_{AE} \cdot \sin(\beta_1) \cdot L_2}{\gamma_{EVmin} \cdot V_1 + P_{AE} \cdot \sin(\beta_1)}$$

Rotation Check

$$\textbf{RotationCheckCase2} := \textbf{if}\left(\frac{2 \cdot L}{5} \ge e_2, \text{“OK”}, \text{“Not OK”}\right)$$

$$\textbf{RotationCheckCase2} = \text{“OK”}$$

4.3. Load Case 3

Notes:

1. Weight and width of the facing is neglected in this calculation
2. Traffic surcharge was ignored as it contribute to reduce the eccentricity
3. Moment is estimated about the middle of the bottom width of wall.

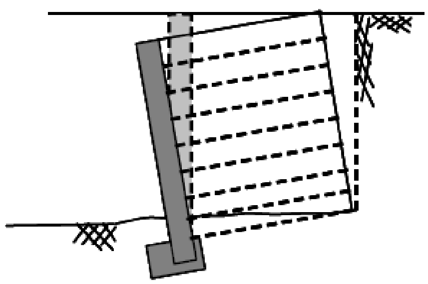
Eccentricity

$$e_3 := \frac{\gamma_{EHmax} \cdot F_H \cdot \frac{H}{3} - \gamma_{EHmax} \cdot F_V \cdot L_1 + \gamma_{LS} \cdot F_{CH} \cdot \frac{H}{2} - \gamma_{LS} \cdot F_{CV} \cdot L_2}{\gamma_{EVmin} \cdot V_1 + \gamma_{EHmax} \cdot F_V + \gamma_{LS} \cdot F_{CV}}$$

Rotation Check $\mathbf{RotationCheckCase3} := \textbf{if}\left(\frac{L}{4} \geq e_3, \text{“OK”}, \text{“Not OK”}\right)$

$\mathbf{RotationCheckCase3} = \text{“OK”}$

5. Bearing Capacity of Foundation Soil



Live load (traffic) has been assumed as an equivalent live load surcharge of 20 kPa

5.1. Load Case 1

Eccentricity $e_{B1} := \dfrac{\gamma_{EHmax} \cdot F_H \cdot \frac{H}{3} - \gamma_{EHmax} \cdot F_V \cdot L_1}{\gamma_{EVmax} \cdot V_1 + \gamma_{EHmax} \cdot F_V + \gamma_{LS} \cdot q_L \cdot w}$

Effective foundation width $L' := \textbf{if}\left(\left(L - 2 \cdot e_{B1}\right) \leq 0, L, L - 2 \cdot e_{B1}\right)$

Factored vertical stress $q_{vf1} := \dfrac{\gamma_{EVmax} \cdot V_1 + \gamma_{LS} \cdot q_L \cdot w + \gamma_{EHmax} \cdot F_V}{L'}$

Estimation of bearing capacity of the foundation

$$N_q := e^{\pi \cdot \tan\left(\phi_f\right)} \cdot \tan\left(45.\ deg + \frac{\phi_f}{2}\right)^2$$

Bearing resistance - Undrained $q_{n\_undrained} := \textbf{if}\left(C_u = \text{“NA”}, \text{“NA”}, 5.14\ C_u + N_q \cdot \gamma_f \cdot d\right)$

$$N_\gamma := 2 \cdot \left(N_q + 1\right) \cdot \tan\left(\phi_f\right)$$

Unit weight of water $\gamma_w := 9.81\ \dfrac{kN}{m^3}$

Effective unit weight of soil adjusted to ground water

$$\gamma_{fdw} := \mathbf{if}\left(\left(D_w - d\right) \geq L + d, \gamma_f, \mathbf{if}\left(D_w \leq d, \left(\gamma_f - \gamma_w\right), \frac{\left(L + d - D_w\right) \cdot \left(\gamma_f - \gamma_w\right) + \left(D_w - d\right) \cdot \gamma_f}{L}\right)\right)$$

Bearing resistance - Drained $\quad q_{n\_drained} := 0.5 \cdot L' \cdot N_\gamma \cdot \gamma_{fdw}$

$$q_n := \mathbf{if}\left(C_u = \text{“NA”}, q_{n\_drained}, min\left(q_{n\_undrained}, q_{n\_drained}\right)\right)$$

Factored bearing resistance $\quad q_R := \phi \cdot q_n$

Bearing capacity check $\quad \mathbf{BearingCheckCase1} := \mathbf{if}\left(q_R \geq q_{vf1}, \text{“OK”}, \text{“Not OK”}\right)$

$$\mathbf{BearingCheckCase1} = \text{“OK”}$$

5.2. Load Case 2

Eccentricity $\quad e_{B2} := \dfrac{P_{IR} \cdot y_c + P_{AE} \cdot \cos\left(\beta_1\right) \cdot 0.5 \cdot H - P_{AE} \cdot \sin\left(\beta_1\right) \cdot L_2}{\gamma_{EVmin} \cdot V_1 + P_{AE} \cdot \sin\left(\beta_1\right)}$

Effective foundation width $\quad L'' := \mathbf{if}\left(\left(L - 2 \cdot e_{B2}\right) \leq 0, L, L - 2 \cdot e_{B2}\right)$

Factored vertical stress $\quad q_{vf2} := \dfrac{\gamma_{EVmin} \cdot V_1 + \gamma_{EQ} \cdot q_L \cdot w + P_{AE} \cdot \sin\left(\beta_1\right)}{L''}$

Note:
Resistance factor = 1.0 is recommended for the Load Case: 2

Bearing capacity check $\quad \mathbf{BearingCheckCase2} := \mathbf{if}\left(\dfrac{q_R}{\phi} \geq q_{vf2}, \text{“OK”}, \text{“Not OK”}\right)$

$$\mathbf{BearingCheckCase2} = \text{“OK”}$$

5.1. Load Case 3

Eccentricity

$$e_{B3} := \frac{\gamma_{EHmax} \cdot F_H \cdot \frac{H}{3} - \gamma_{EHmax} \cdot F_V \cdot L_1 + \gamma_{LS} \cdot F_{CH} \cdot \frac{H}{2} - \gamma_{LS} \cdot F_{CV} \cdot L_2}{\gamma_{EVmax} \cdot V_1 + \gamma_{EHmax} \cdot F_V + \gamma_{LS} \cdot q_c \cdot w + \gamma_{LS} \cdot F_{CV}}$$

Effective foundation width $\quad L''' := \mathbf{if}\left(\left(L - 2 \cdot e_{B3}\right) \leq 0, L, L - 2 \cdot e_{B3}\right)$

Factored vertical stress $q_{vf3} := \dfrac{\gamma_{EVmax} \cdot V_1 + \gamma_{LS} \cdot q_c \cdot w + \gamma_{EHmax} \cdot F_V + \gamma_{LS} \cdot F_{CV}}{L'''}$

Bearing resistance - Undrained $q_{n\_undrained\_con} := \textbf{if}\left(C_u = \text{“NA”}, \text{“NA”}, 5.14\ C_u + N_q \cdot \gamma_f \cdot d\right)$

$$q_{n\_con} := \textbf{if}\left(C_u = \text{“NA”}, q_{n\_drained}, min\left(q_{n\_undrained\_con}, q_{n\_drained}\right)\right)$$

Factored bearing resistance $q_R := \phi \cdot q_{n\_con}$

Bearing capacity check $\text{BearingCheckCase3} := \textbf{if}\left(q_R \geq q_{vf3}, \text{“OK”}, \text{“Not OK”}\right)$

$\text{BearingCheckCase3} = \text{“OK”}$

6. Design Check - External Stability

6.1. Load Case 1

$\text{SlidingCheckCase1} = \text{“OK”}$

$\text{RotationCheckCase1} = \text{“OK”}$

$\text{BearingCheckCase1} = \text{“OK”}$

6.2. Load Case 2

$\text{SlidingCheckCase2} = \text{“OK”}$

$\text{RotationCheckCase2} = \text{“OK”}$

$\text{BearingCheckCase2} = \text{“OK”}$

6.3. Load Case 3

$\text{SlidingCheckCase3} = \text{“OK”}$

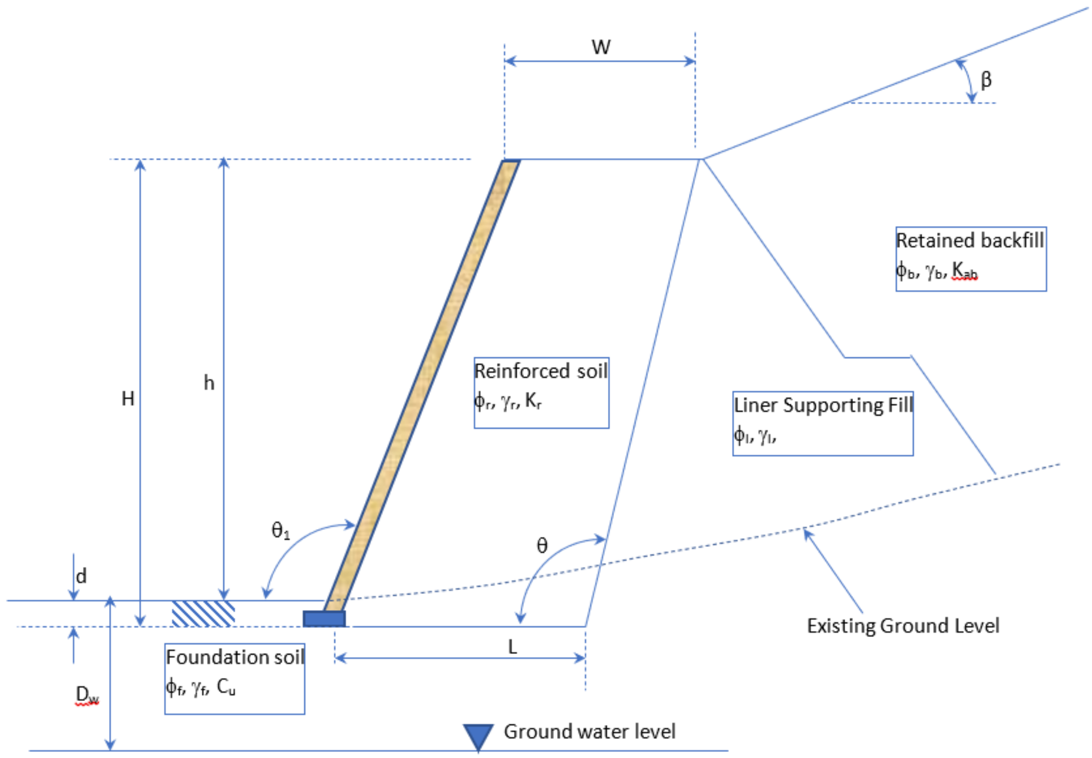
$\text{RotationCheckCase3} = \text{“OK”}$

$\text{BearingCheckCase3} = \text{“OK”}$

6.4. Load Case 4

Vehicular impact on traffic barrier tends to affect only the internal stability of MSE walls (reinforcement). So, the external stability assessment has not been carried for this load case.

DESIGN OF MECHNICALLY STABILISED EARTH (MSE) WALLS



References:

1. Design and Construction of Mechanically Stabilized Earth Walls and Reinforced Soil Slopes – Volume I, Publication No. FHWA-NHI-10-024, Federal Highway Administration FHWA GEC 011 – Volume I.
2. AASHTO LRFD Bridge Design Specifications, Seventh. Edition, 2014.
3. AS 4678-2002 - Earth-retaining structures.
4. AS1170.4-1993 - Minimum Design Loads on Structures, Part 4: Earthquake Loads.

1. External Stability Design Paramaters

Wall location Ch600

1.1. Wall Geometry

Wall Height above the existing ground level, h $h := 17.59\ m$

Embedment depth $d := 2.1\ m$

Top width of wall $W := 13.5\ m$

Slope of backfill behind wall $\beta := 14\ deg$

Slope in Front of Structure	Minimum Embedment Depth
Horizontal	H/20.0
3.0H : 1.0V	H/10.0
2.0H : 1.0V	H/7.0
1.5H : 1.0V	H/5.0

Face inclination from horizontal $\theta_1 := 108.4\ deg$ Wall slope: 3V:1H

Depth of groundwater below the existing ground level $D_w := 1.0\ m$

1.2. Reinforced Soil Block Parameters

Eff. Friction of reinforced block $\phi_r := 32\ deg$

Unit weight of reinforced block $\gamma_r := 20\ \dfrac{kN}{m^3}$

1.3. Retained backfill Parameters

__Note__

1. Retained backfill friction angle was assumed to be friction angle of the landfill, cohesion was assumed zero.
2. Unit weight of retained backfill was assumed to be equal to unit weight of the liner supporing fill.

Eff. Frcition of retained backfill $\phi_b := 27\ deg$

Unit weight of backfill $\gamma_b := 20\ \dfrac{kN}{m^3}$

1.4. Foundation Soil Parameters

Drained friction angle of foundation soil $\phi_f := 29\ deg$

Undrained shear strength of foundation soil $C_u := 150\ kPa$ Enter "NA", if Cu is not applicable to foundation soil

Unit weight of foundation soil $\gamma_f := 18\ \dfrac{kN}{m^3}$

__Note__

1. For the estimation of top width of wall, width of draingae chimney is reduced.
2. facing was considered as part of the MSE wall as geogrid was used to wrap back.

Wall Height $H := h + d = 19.69\ m$

Top width of MSE wall $w := W - 1.0\ m$

Ratio between MSE wall height and reinforcement length, L/H $\lambda := 1.0$

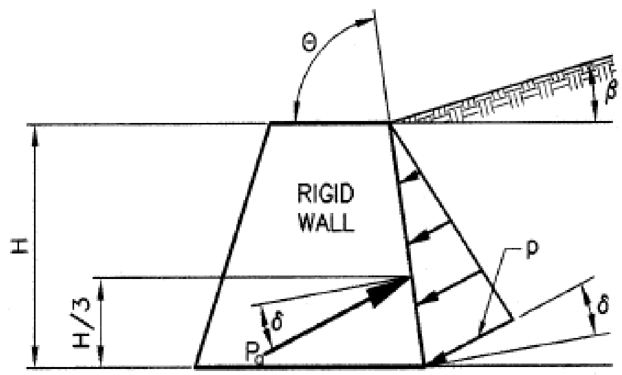
Min. bottom width of reinforced MSE wall $L := \mathbf{if}\left(\left(w + \dfrac{h}{3}\right) \le \lambda \cdot H, w + \dfrac{h}{3}, \lambda \cdot H\right)$

2. Loads

Angle of fric between retained backfill and rein. Soil $\delta := \beta$ Assumed Equal to β

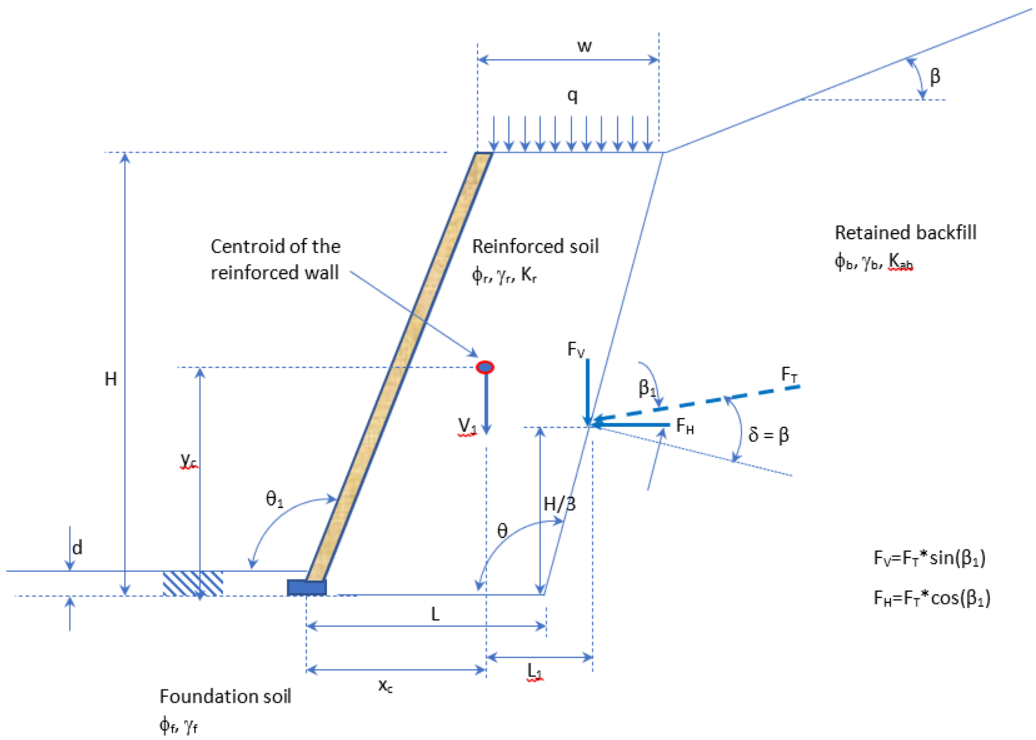
Wall batter $$\theta := 180.\ deg - \operatorname{atan}\left(\frac{H}{\frac{H}{\tan(180.\ deg - \theta_1)} + w - L}\right)$$

$$\theta := \text{if}\left(\theta \geq 180.\ deg, \theta - 180\ deg, \theta\right)$$



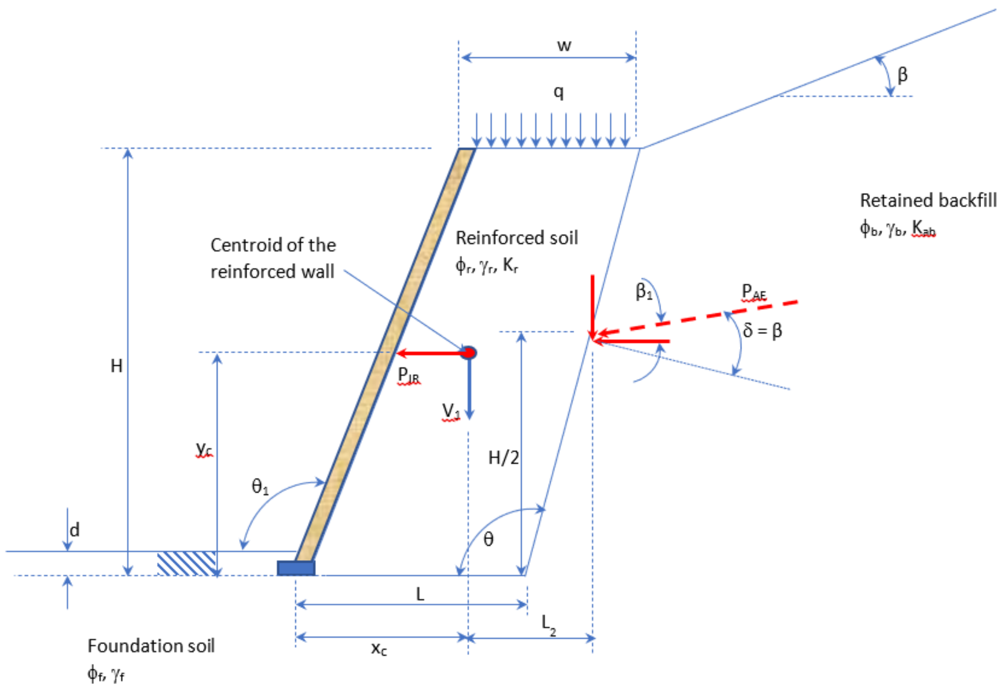
2.1. Load Combinations

Load Case 1 (Strength): $"\gamma_p \cdot EV" + "\gamma_p \cdot EH" + "\gamma_{LS} \cdot LS"$



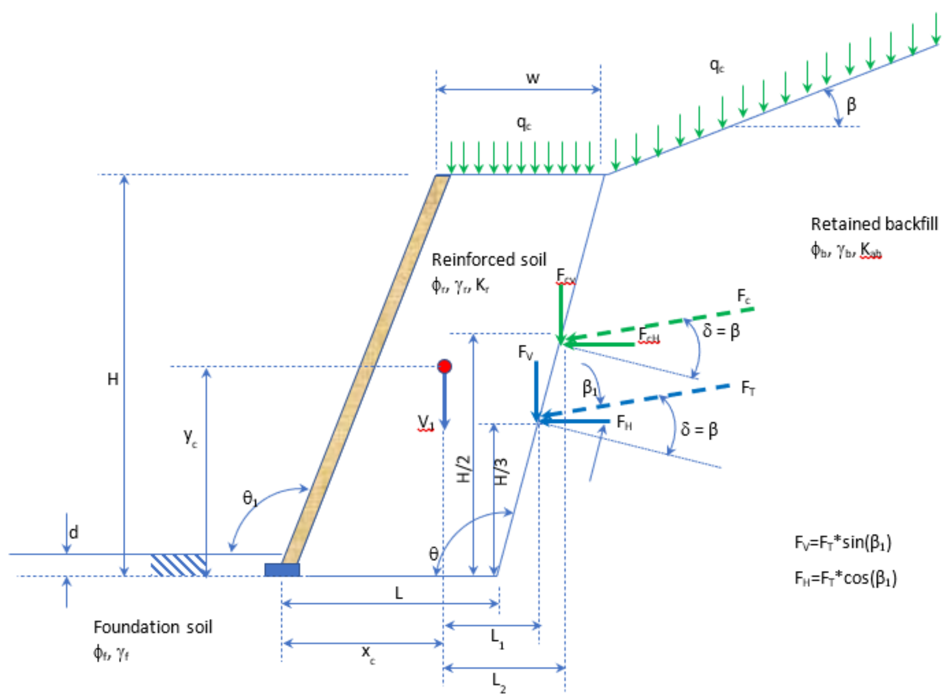
Load Case 2 (Earth quake):

$$"\gamma_p \cdot EV" + "\gamma_p \cdot EH" + "\gamma_{EQ} \cdot LS" + "1.0 \cdot EQ"$$



Load Case 3 (Construction):

$$"\gamma_p \cdot EV" + "\gamma_p \cdot EH" + "\gamma_{LS} \cdot LS"$$



Load Case 4 (Vehicular Impact):

Vehicular impact on traffic barrier tends to affect only the internal stability of MSE walls (reinforcement). So, the external stability assessment has not been carried for this load case.

2.2. Applicable loads

Horizontal earth pressure (EH)   Vertcial earth pressure (EV)   Earthquake Load (EQ)

Live load (traffic) surcharge (LS) $q_L := 20\ kPa$

Live load (construction) surcharge (LS) $q_c := 10\ kPa$

$$\Gamma := \left(1 + \sqrt{\frac{\sin(\phi_b + \delta) \cdot \sin(\phi_b - \beta)}{\sin(\theta - \delta) \cdot \sin(\theta + \beta)}}\right)^2$$

Cofficient of active earth pressure $$K_{ab} := \frac{\left(\sin(\theta + \phi_b)\right)^2}{\Gamma \cdot (\sin(\theta))^2 \cdot \sin(\theta - \delta)}$$

Assumption: live loads due to traffic and construction have been considered as a surcharge in the external stability estimation.

2.3. Load and Resistance factors

Maximum horizontal earth pressure factor $\gamma_{EHmax} := 1.50$

Minimum horizontal earth pressure factor $\gamma_{EHmin} := 0.90$

Minimum vertical earth pressure factor $\gamma_{EVmin} := 1.00$

Maximum vertcial earth pressure factor $\gamma_{EVmax} := 1.35$

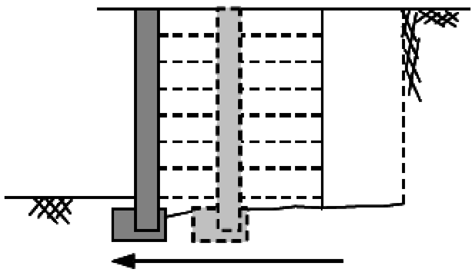
Live load factor $\gamma_{LS} := 1.75$

Load factor for live load for load case 2 $\gamma_{EQ} := 1.00$

Resistance factor for shear resistance between soil and foundation $\phi_\tau := 1.00$

Resistance factor for bearing $\phi := 0.65$

3. Sliding Stability



3.1. Load Case 1

Note

1. Inclination of retained backfill force resultant to normal of the back wall face (δ) is assumed to be β ($\delta=\beta$).
2. Inclination of retained backfill force resultant to horizontal (β_1) will be function of back wall inclination θ and δ.
3. β_1 is estimated for three different cases
   a) $\theta \geq 90\ deg \wedge (\theta - 90.\ deg) \geq \delta$
   b) $\theta \geq 90\ deg \wedge (\theta - 90.\ deg) < \delta$
   C) $\theta < 90\ deg$

$\beta_a := (\theta - 90\ deg) - \delta$ $\qquad$ $\beta_b := \delta - (\theta - 90\ deg)$ $\qquad$ $\beta_c := 90\ deg - \theta + \delta$

$\beta_1 := \text{if}\left(\theta \geq 90\ deg \wedge (\theta - 90.\ deg) \geq \delta, \beta_a, \text{if}\left(\theta \geq 90\ deg \wedge (\theta - 90.\ deg) < \delta, \beta_b, \beta_c\right)\right)$

Retained backfill force resultant per unit width $\qquad F_T := \dfrac{1}{2} K_{ab} \cdot \gamma_b \cdot H^2$

Horizontal driving force per unit width $\qquad F_H := F_T \cdot \cos(\beta_1)$

Vertical force per unit width $\qquad F_V := F_T \cdot \sin(\beta_1)$

Factored horizontal driving force per unit width $\qquad P_{d1} := \gamma_{EHmax} \cdot F_H$

Weight of reinforced block $\qquad V_1 := \gamma_r \cdot H \cdot \dfrac{(L + w)}{2}$

Minimum soil friction angle $\qquad \mu := min\left(\tan(\phi_r), \tan(\phi_f)\right)$

Notes:
1. Live load is excluded as it increases the sliding stability.

Sliding resistance between rein. soil and foundation - Drained

$$R_{\tau\_drained} := \left(\gamma_{EVmin} \cdot V_1 + \gamma_{EHmin} \cdot F_V\right) \cdot \mu$$

Sliding resistance between rein. soil and foundation -Undrained

$$R_{\tau\_undrained} := \textbf{if}\left(C_u = \text{“NA”}, \text{“NA”}, C_u \cdot L\right)$$

Note:
If ground water is below the foundation level (Dw-d > 2.0 m below foundation level), sliding resistance (undrained) was not considered in the external stability.

$$R_{\tau\_undrained} := \textbf{if}\left(D_w - d \geq 2.0\ m, \text{“NA”}, R_{\tau\_undrained}\right)$$

Factored Sliding resistance

$$R_r := \textbf{if}\left(R_{\tau\_undrained} = \text{“NA”}, \phi_\tau \cdot R_{\tau\_drained}, min\left(\phi_\tau \cdot R_{\tau\_drained}, \phi_\tau \cdot R_{\tau\_undrained}\right)\right)$$

Sliding Check

$$\textbf{SlidingCheckCase1} := \textbf{if}\left(R_r \geq P_{d1}, \text{“OK”}, \text{“Not OK”}\right)$$

$$\textbf{SlidingCheckCase1} = \text{“OK”}$$

3.2. Load Case 2

AS4678 is adopted for the estimation of acceleration coefficient.

Live load (traffic) has been assumed to an equivalent live load surcharge of 20 kPa as it increases the horizontal force.

Peak ground acceleration coefficient $a := 0.08$

Horizontal coefficient of acceleration $a_h := 0.5 \cdot a$

Horizontal coefficient of acceleration with amplification of motion

$$a_{mh} := \left(1.45 - a_h\right) \cdot a_h \qquad k_v := 0.00$$

$$\xi := \mathrm{atan}\left(\frac{a_{mh}}{1 - k_v}\right)$$

$$\delta_1 := min\left(\phi_r, \phi_b\right)$$

$$I := \beta$$

$$\chi := 90.\ deg$$

Mononobe-Okabe (M-O) formulation

$$K_{AE} := \frac{\cos\left(\phi_b - \xi - \chi + \theta\right)^2}{\cos(\xi) \cdot \cos(\chi - \theta)^2 \cdot \cos\left(\delta_1 + \chi - \theta + \xi\right) \cdot \left(1 + \sqrt{\frac{\sin\left(\phi_b + \delta_1\right) \cdot \sin\left(\phi_b - \xi - I\right)}{\cos\left(\delta_1 + \chi - \theta + \xi\right) \cdot \cos(I - \chi + \theta)}}\right)^2}$$

Total (static + dynamic) thrust $P_{AE} := 0.5 \cdot K_{AE} \cdot \gamma_b \cdot H^2$

Horizontal inertial force $P_{IR} := 0.5 \cdot \left(a_{mh} \cdot V_1 + \gamma_{EQ} \cdot q_L \cdot w \cdot a_{mh}\right)$

Total horizontal force $T_{HF} := P_{AE} \cdot \cos\left(\delta_1\right) + P_{IR}$

Notes:
1. Live load was considered as part of the reinforced soil mass

Sliding Check $\textbf{SlidingCheckCase2} := \textbf{if}\left(R_r \geq T_{HF}, \text{“OK”}, \text{“Not OK”}\right)$

$\textbf{SlidingCheckCase2} = \text{“OK”}$

3.3. Load Case 3

Uniform construction surcharge resultant per unit width $F_C := K_{ab} \cdot q_c \cdot H$

Horizontal component of Fc $F_{CH} := F_C \cdot \cos\left(\beta_1\right)$

Vertical component of Fc $F_{CV} := F_C \cdot \sin\left(\beta_1\right)$

Factored horizontal driving force per unit width $P_{d3} := \gamma_{EHmax} \cdot F_H + \gamma_{LS} \cdot F_{CH}$

Weight of reinforced block $V_1 := \gamma_r \cdot H \cdot \frac{(L + w)}{2}$

Minimum soil friction angle $\mu := \mathit{min}\left(\tan\left(\phi_r\right), \tan\left(\phi_f\right)\right)$

Notes:
1. Live load surcharge immidiately above the reinforced fill is excluded as it increases the sliding stability.

Sliding resistance between rein. soil and foundation - Drained $R_{\tau\_drained} := \left(\gamma_{EVmin} \cdot V_1 + \gamma_{EHmin} \cdot F_V + \gamma_{EHmin} \cdot F_{CV}\right) \cdot \mu$

Sliding resistance between rein. soil and foundation -Undrained $R_{\tau\_undrained\_con} := \textbf{if}\left(C_u = \text{“NA”}, \text{“NA”}, C_u \cdot L\right)$

Note:
If ground water is below the foundation level (if Dw-d > 2.0 m below foundation level), sliding resistance (undrained) was not considered in the external stability.

$$R_{\tau\_undrained\_con} := \textbf{if}\left(D_w - d \ge 2.0\ m, \text{“NA”}, R_{\tau\_undrained}\right)$$

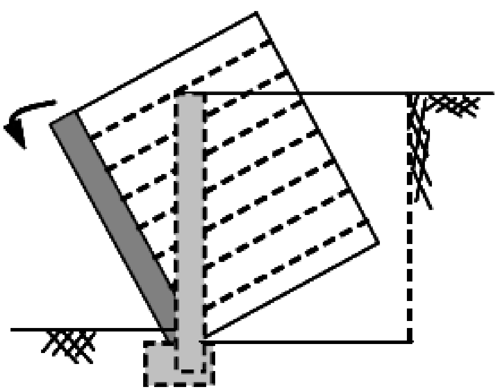
Factored Sliding resistance

$$R_r := \textbf{if}\left(R_{\tau\_undrained\_con} = \text{“NA”}, \phi_\tau \cdot R_{\tau\_drained}, min\left(\phi_\tau \cdot R_{\tau\_drained}, \phi_\tau \cdot R_{\tau\_undrained\_con}\right)\right)$$

Sliding Check $\textbf{SlidingCheckCase3} := \textbf{if}\left(R_r \ge P_{d3}, \text{“OK”}, \text{“Not OK”}\right)$

$$\textbf{SlidingCheckCase3} = \text{“OK”}$$

4. Rotational/Overturning Stability



4.1. Load Case 1

Notes:
1. Weight and width of the facing is neglected in this calculation
2. Traffic surcharge was ignored as it contribute to reduce the eccentricity
3. Moment is estimated about the middle of the bottom width of wall.

Estimation of Centeroid of the wall from the toe

$$l_1 := \frac{H}{\tan\left(180.\ deg - \theta_1\right)} \qquad l_2 := L - l_1 - w = -0.687\ m$$

X distance to centeroid of the reforced wall from toe of the wall

$$x_c := \frac{\frac{l_1^{\ 2}}{3} + w \cdot \left(l_1 + \frac{w}{2}\right) + \frac{1}{2} \cdot l_2 \cdot \left(l_1 + w + \frac{1}{3} \cdot l_2\right)}{\frac{(L + w)}{2}}$$

Y distance to centeroid of the reforced wall from toe of the wall

$$y_c := \frac{H}{3} \cdot \frac{(L + 2 \cdot w)}{L + w}$$

$$L_1 := \textbf{if}\left(l_2 \le 0, l_1 + w - x_c + \frac{2}{3} \cdot l_2, L - x_c - \frac{2}{3} \cdot l_2\right)$$

Eccentricity $$e_1 := \frac{\gamma_{EHmax} \cdot F_H \cdot \frac{H}{3} - \gamma_{EHmax} \cdot F_V \cdot L_1}{\gamma_{EVmin} \cdot V_1 + \gamma_{EHmax} \cdot F_V}$$

Rotation Check $$\mathbf{RotationCheckCase1} := \textbf{if}\left(\frac{L}{4} \ge e_1, \text{“OK”}, \text{“Not OK”}\right)$$

$$\mathbf{RotationCheckCase1} = \text{“OK”}$$

4.2. Load Case 2

$$L_2 := \textbf{if}\left(l_2 \le 0, l_1 + w - x_c + \frac{1}{2} \cdot l_2, L - x_c - \frac{1}{2} \cdot l_2\right)$$

Notes:
1. Weight and width of the facing is neglected in this calculation
2. Traffic surcharge was ignored as it contribute to reduce the eccentricity
3. Moment is estimated about the middle of the bottom width of wall.

Eccentricity $$e_2 := \frac{P_{IR} \cdot y_c + P_{AE} \cdot \cos\left(\beta_1\right) \cdot 0.5 \cdot H - P_{AE} \cdot \sin\left(\beta_1\right) \cdot L_2}{\gamma_{EVmin} \cdot V_1 + P_{AE} \cdot \sin\left(\beta_1\right)}$$

Rotation Check $$\mathbf{RotationCheckCase2} := \textbf{if}\left(\frac{2 \cdot L}{5} \ge e_2, \text{“OK”}, \text{“Not OK”}\right)$$

$$\mathbf{RotationCheckCase2} = \text{“OK”}$$

4.3. Load Case 3

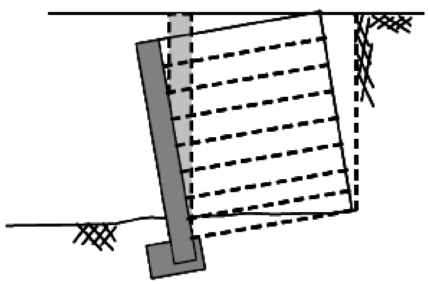
Notes:
1. Weight and width of the facing is neglected in this calculation
2. Traffic surcharge was ignored as it contribute to reduce the eccentricity
3. Moment is estimated about the middle of the bottom width of wall.

Eccentricity $$e_3 := \frac{\gamma_{EHmax} \cdot F_H \cdot \frac{H}{3} - \gamma_{EHmax} \cdot F_V \cdot L_1 + \gamma_{LS} \cdot F_{CH} \cdot \frac{H}{2} - \gamma_{LS} \cdot F_{CV} \cdot L_2}{\gamma_{EVmin} \cdot V_1 + \gamma_{EHmax} \cdot F_V + \gamma_{LS} \cdot F_{CV}}$$

Rotation Check $\mathbf{RotationCheckCase3} := \mathbf{if}\left(\frac{L}{4} \geq e_3, \text{“OK”}, \text{“Not OK”}\right)$

$\mathbf{RotationCheckCase3} = \text{“OK”}$

5. Bearing Capacity of Foundation Soil



Live load (traffic) has been assumed as an equivalent live load surcharge of 20 kPa

5.1. Load Case 1

Eccentricity $$e_{B1} := \frac{\gamma_{EHmax} \cdot F_H \cdot \frac{H}{3} - \gamma_{EHmax} \cdot F_V \cdot L_1}{\gamma_{EVmax} \cdot V_1 + \gamma_{EHmax} \cdot F_V + \gamma_{LS} \cdot q_L \cdot w}$$

Effective foundation width $L' := \mathbf{if}\left(\left(L - 2 \cdot e_{B1}\right) \leq 0, L, L - 2 \cdot e_{B1}\right)$

Factored vertical stress $$q_{vf1} := \frac{\gamma_{EVmax} \cdot V_1 + \gamma_{LS} \cdot q_L \cdot w + \gamma_{EHmax} \cdot F_V}{L'}$$

Estimation of bearing capacity of the foundation

$$N_q := e^{\pi \cdot \tan(\phi_f)} \cdot \tan\left(45.\ deg + \frac{\phi_f}{2}\right)^2$$

Bearing resistance - Undrained $q_{n\_undrained} := \mathbf{if}\left(C_u = \text{“NA”}, \text{“NA”}, 5.14\ C_u + N_q \cdot \gamma_f \cdot d\right)$

$$N_\gamma := 2 \cdot \left(N_q + 1\right) \cdot \tan\left(\phi_f\right)$$

Unit weight of water $\gamma_w := 9.81\ \frac{kN}{m^3}$

Effective unit weight of soil adjusted to ground water

$$\gamma_{fdw} := \text{if}\left(\left(D_w - d\right) \ge L + d, \gamma_f, \text{if}\left(D_w \le d, \left(\gamma_f - \gamma_w\right), \frac{\left(L + d - D_w\right) \cdot \left(\gamma_f - \gamma_w\right) + \left(D_w - d\right) \cdot \gamma_f}{L}\right)\right)$$

Bearing resistance - Drained $\quad q_{n\_drained} := 0.5 \cdot L' \cdot N_\gamma \cdot \gamma_{fdw}$

$$q_n := \text{if}\left(C_u = \text{"NA"}, q_{n\_drained}, \min\left(q_{n\_undrained}, q_{n\_drained}\right)\right)$$

Factored bearing resistance $\quad q_R := \phi \cdot q_n$

Bearing capacity check $\quad \textbf{BearingCheckCase1} := \text{if}\left(q_R \ge q_{vf1}, \text{"OK"}, \text{"Not OK"}\right)$

$$\textbf{BearingCheckCase1} = \text{"OK"}$$

5.2. Load Case 2

Eccentricity $\quad e_{B2} := \dfrac{P_{IR} \cdot y_c + P_{AE} \cdot \cos\left(\beta_1\right) \cdot 0.5 \cdot H - P_{AE} \cdot \sin\left(\beta_1\right) \cdot L_2}{\gamma_{EVmin} \cdot V_1 + P_{AE} \cdot \sin\left(\beta_1\right)}$

Effective foundation width $\quad L'' := \text{if}\left(\left(L - 2 \cdot e_{B2}\right) \le 0, L, L - 2 \cdot e_{B2}\right)$

Factored vertical stress $\quad q_{vf2} := \dfrac{\gamma_{EVmin} \cdot V_1 + \gamma_{EQ} \cdot q_L \cdot w + P_{AE} \cdot \sin\left(\beta_1\right)}{L''}$

Note:
Resistance factor = 1.0 is recommended for the Load Case: 2

Bearing capacity check $\quad \textbf{BearingCheckCase2} := \text{if}\left(\dfrac{q_R}{\phi} \ge q_{vf2}, \text{"OK"}, \text{"Not OK"}\right)$

$$\textbf{BearingCheckCase2} = \text{"OK"}$$

5.1. Load Case 3

Eccentricity

$$e_{B3} := \frac{\gamma_{EHmax} \cdot F_H \cdot \frac{H}{3} - \gamma_{EHmax} \cdot F_V \cdot L_1 + \gamma_{LS} \cdot F_{CH} \cdot \frac{H}{2} - \gamma_{LS} \cdot F_{CV} \cdot L_2}{\gamma_{EVmax} \cdot V_1 + \gamma_{EHmax} \cdot F_V + \gamma_{LS} \cdot q_c \cdot w + \gamma_{LS} \cdot F_{CV}}$$

Effective foundation width $\quad L''' := \text{if}\left(\left(L - 2 \cdot e_{B3}\right) \le 0, L, L - 2 \cdot e_{B3}\right)$

Factored vertical stress $$q_{vf3} := \frac{\gamma_{EVmax} \cdot V_1 + \gamma_{LS} \cdot q_c \cdot w + \gamma_{EHmax} \cdot F_V + \gamma_{LS} \cdot F_{CV}}{L'''}$$

Bearing resistance - Undrained $$q_{n\_undrained\_con} := \mathbf{if}\left(C_u = \text{"NA"}, \text{"NA"}, 5.14\ C_u + N_q \cdot \gamma_f \cdot d\right)$$

$$q_{n\_con} := \mathbf{if}\left(C_u = \text{"NA"}, q_{n\_drained}, \min\left(q_{n\_undrained\_con}, q_{n\_drained}\right)\right)$$

Factored bearing resistance $$q_R := \phi \cdot q_{n\_con}$$

Bearing capacity check $$\text{BearingCheckCase3} := \mathbf{if}\left(q_R \geq q_{vf3}, \text{"OK"}, \text{"Not OK"}\right)$$

$$\text{BearingCheckCase3} = \text{"OK"}$$

6. Design Check - External Stability

6.1. Load Case 1

SlidingCheckCase1 = "OK"

RotationCheckCase1 = "OK"

BearingCheckCase1 = "OK"

6.2. Load Case 2

SlidingCheckCase2 = "OK"

RotationCheckCase2 = "OK"

BearingCheckCase2 = "OK"

6.3. Load Case 3

SlidingCheckCase3 = "OK"

RotationCheckCase3 = "OK"

BearingCheckCase3 = "OK"

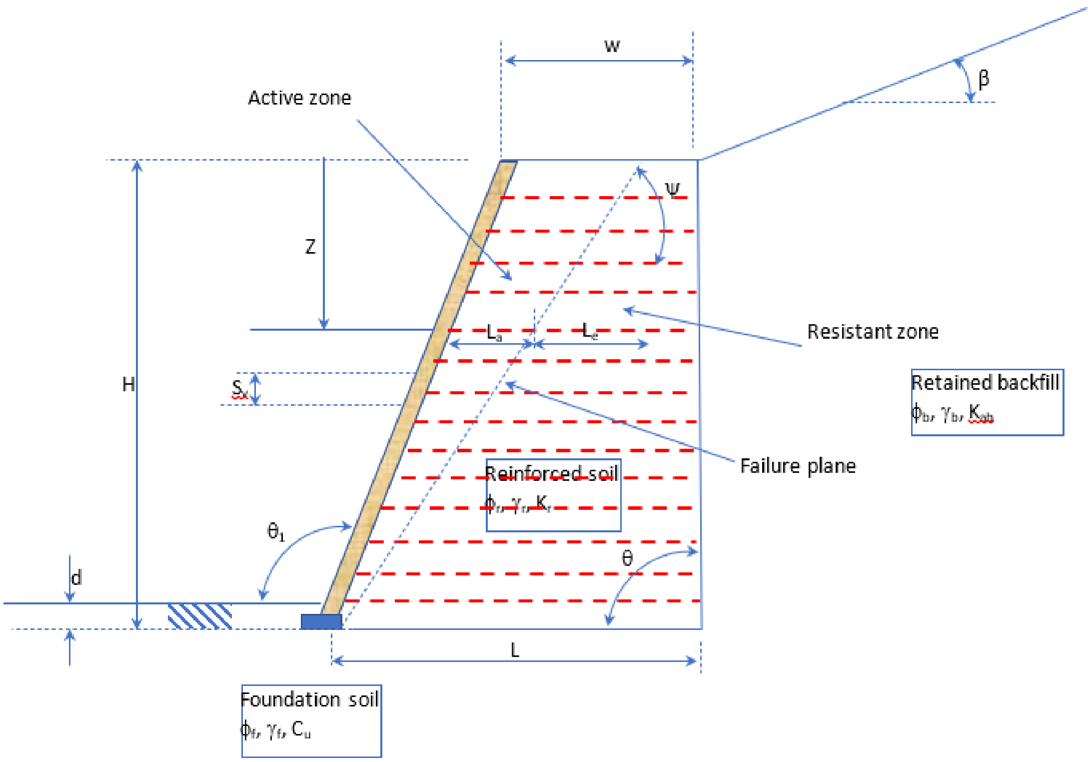
6.4. Load Case 4

Vehicular impact on traffic barrier tends to affect only the internal stability of MSE walls (reinforcement). So, the external stability assessment has not been carried for this load case.

APPENDIX D

Internal Stability

Internal Stability - Sample Calculations



Calculation below shows the sample calcuation of tensile and pull-out failure of reinforcement at a specific depth/layer.

Layer # $n := 20$ Refer to excel sheet below

Depth of reinforcement $Z := 5.5\ m$

Vertical reinforcement spacing/Contributory height $S_v := 0.6\ m$

Ultimate Tensile Strength $T_{ult} := 120\ \frac{kN}{m}$ For GG120

Length of reinforcement $L_r := 10.4\ m$

Total no of reinforcement layers within the wall $N := 28$ To be estimated based on geometry and spacing

Partial factor - creep rupture $RF_{CR} := 1.45$

Partial factor - construction damage $RF_{ID} := 1.1$

Partial factor - environmental effects $RF_D := 1.05$

Combined strength reduction factor $RF := RF_{CR} \cdot RF_{ID} \cdot RF_D$

Pullout resistance factor $F := 0.42$

Scale correction factor $\alpha := 0.8$

Coverage ratio $R_c := 1$ $C := 2$

Rankine active earth pressure coefficient

$$K_a := \frac{\sin(\theta + \phi_r)^2}{\sin(\theta)^3 \cdot \left(1 + \frac{\sin(\phi_r)}{\sin(\theta)}\right)^2}$$

lateral earth pressure coefficient $K_r := K_a$ Extensible reinforcement (geogrid)

$a_1 := \tan(\phi_r - \beta)$ $a_2 := \cot(\phi_r + \theta - 90.\ deg)$ $a_3 := \tan(\delta + 90.\ deg - \theta)$

Inclination of failure plane

$$\psi := \operatorname{atan}\left(\frac{-a_1 + \sqrt{a_1 \cdot (a_1 + a_2)\ (1 + a_3 \cdot a_2)}}{1 + a_3 \cdot (a_1 + a_2)}\right) + \phi_r$$

1. Tensile Failure of Reinforcement

1.1. Load Case 1

Surcharge equivalent height

$$h_{eq1} := \frac{q_L \cdot \gamma_{LS}}{\gamma_r \cdot \gamma_{EVmax}}$$

Horizontal stress at depth Z

$$\sigma_{H1} := K_r \cdot \gamma_r \cdot (Z + h_{eq1}) \cdot \gamma_{EVmax}$$

Maximum factored tension

$$T_{max1} := \sigma_{H1} \cdot S_v$$

Nominal long-term reinforcement strength

$$T_{al1} := \frac{T_{ult}}{RF_{CR} \cdot RF_{ID} \cdot RF_D}$$

Resistance Factors for tensile and pullout resistance $\phi_{GG1} := 0.9$

Factored tensile resistance $T_{r1} := \phi_{GG1} \cdot T_{al1}$

Tensile Check $\mathbf{TensileCheck1} := \mathbf{if}\left(T_{r1} \geq T_{max1}, \text{“OK”}, \text{“Not OK”}\right)$

$\mathbf{TensileCheck1} = \text{“OK”}$

1.2. Load Case 2

Surcharge equivalent height $h_{eq2} := \dfrac{q_L \cdot \gamma_{EQ}}{\gamma_r \cdot \gamma_{EVmax}}$

Horizontal stress at depth Z $\sigma_{H2} := K_r \cdot \gamma_r \cdot \left(Z + h_{eq2}\right) \cdot \gamma_{EVmax}$

Maximum factored tension $T_{max2} := \sigma_{H2} \cdot S_v$

Soil weight of the active zone $W_a := \dfrac{1}{2} \cdot \gamma_r \cdot H^2 \cdot \left(\tan\left(90.\ deg - \psi\right) - \tan\left(\theta_1 - 90.\ deg\right)\right)$

Factored incremental dynamic inertia force $T_{md} := \dfrac{a_{mh} \cdot W_a}{N - 1}$

Resistance Factors for tensile and pullout resistance $\phi_{GG2} := 1.2$

Static component of resistance $S_{rs2} := \dfrac{T_{max2} \cdot RF}{\phi_{GG2} \cdot R_c}$

Dynamic component of resistance $S_{rt2} := \dfrac{T_{md} \cdot RF_{ID} \cdot RF_D}{\phi_{GG2} \cdot R_c}$

Tensile Check $\mathbf{TensileCheck2} := \mathbf{if}\left(T_{ult} \geq \left(S_{rs2} + S_{rt2}\right), \text{“OK”}, \text{“Not OK”}\right)$

$\mathbf{TensileCheck2} = \text{“OK”}$

1.3. Load Case 3

Surcharge equivalent height $h_{eq3} := \dfrac{q_c \cdot \gamma_{LS}}{\gamma_r \cdot \gamma_{EVmax}}$

Horizontal stress at depth Z $\sigma_{H3} := K_r \cdot \gamma_r \cdot \left(Z + h_{eq3}\right) \cdot \gamma_{EVmax}$

Maximum factored tension $T_{max3} := \sigma_{H3} \cdot S_v$

Nominal long-term reinforcement strength $T_{al3} := \dfrac{T_{ult}}{RF_{ID} \cdot RF_D}$ No creep considered during construction

Resistance Factors for tensile and pullout resistance $\phi_{GG3} := 0.9$

Factored tensile resistance $T_{r3} := \phi_{GG3} \cdot T_{al3}$

Tensile Check $\mathbf{TensileCheck3} := \mathbf{if}\left(T_{r3} \geq T_{max3}, \text{“OK”}, \text{“Not OK”}\right)$

$\mathbf{TensileCheck3} = \text{“OK”}$

1.4. Load Case 4

Surcharge equivalent height $h_{eq4} := \dfrac{q_L \cdot 1.0}{\gamma_r}$ Assumed 1 m

Horizontal stress at depth Z $\sigma_{H4} := K_r \cdot \gamma_r \cdot \left(Z + h_{eq4}\right) \cdot \gamma_{EVmax}$

Maximum factored tension $T_{max4} := \sigma_{H4} \cdot S_v$

Note

1. Traffic railing impact events tend to affect only the internal stability of MSE walls

2. The recommended static impact force is assumed 45 kN applied on a barrier with a minimum height of 810 mm above the road surface.

3. As per FHWA NHI-10-024, the static impact force, adds an additional horizontal force to the upper 2 layers of soil reinforcement.

4. The upper layer of soil reinforcement be designed for a rupture impact load equivalent to a static load of 33.5 kN/m of wall.

5. The second layer be designed with a rupture impact load equivalent to a static load of 8.8 kN/m.

Factored impact load $T_I := \mathbf{if}\left(n = N - 1, 33.5 \dfrac{kN}{m}, \mathbf{if}\left(n = N - 2, 8.8 \dfrac{kN}{m}, 0 \dfrac{kN}{m}\right)\right)$

Resistance Factors for tensile and pullout resistance $\phi_{GG4} := 1.0$

Static component of resistance $S_{rs4} := \frac{T_{max4} \cdot RF}{\phi_{GG4} \cdot R_c}$

Dynamic component of resistance $S_{rt4} := \frac{T_I \cdot RF_{ID} \cdot RF_D}{\phi_{GG2} \cdot R_c}$

Tensile Check $\textbf{TensileCheck4} := \textbf{if}\left(T_{ult} \geq \left(S_{rs4} + S_{rt4}\right), \text{"OK"}, \text{"Not OK"}\right)$

$\textbf{TensileCheck4} = \text{"OK"}$

2. Pullout Failure of Reinforcement

2.1. Load Case 1

Nominal vertical stress at depth Z $\sigma_v := \gamma_r \cdot Z$

min. length of embedment in resistant zone $L_{e1} := \textbf{if}\left(\frac{T_{max1}}{\phi_{GG1} \cdot F \cdot \alpha \cdot \sigma_v \cdot C \cdot R_c} \leq 1\ m, 1\ m, \frac{T_{max1}}{\phi_{GG1} \cdot F \cdot \alpha \cdot \sigma_v \cdot C \cdot R_c}\right)$

min. length of embedment in active zone $L_a := (H - Z) \cdot \left(\tan(90.\ deg - \psi) - \tan\left(\theta_1 - 90.\ deg\right)\right)$

Pullout Check $\textbf{PulloutCheck1} := \textbf{if}\left(L_r \geq L_{e1} + L_a, \text{"OK"}, \text{"Not OK"}\right)$

$\textbf{PulloutCheck1} = \text{"OK"}$

2.2. Load Case 2

Total factored load (static = dynamic) $T_{total2} := T_{max2} + T_{md}$

min. length of embedment in resistant zone

$$L_{e2} := \textbf{if}\left(\frac{T_{total2}}{\phi_{GG2} \cdot 0.8 \cdot F \cdot \alpha \cdot \sigma_v \cdot C \cdot R_c} \leq 1\ m, 1\ m, \frac{T_{total2}}{\phi_{GG2} \cdot 0.8 \cdot F \cdot \alpha \cdot \sigma_v \cdot C \cdot R_c}\right)$$

Available length of embedment in resistant zone $L_{ea} := L_r - L_a$

Pullout Check $\textbf{PulloutCheck2} := \textbf{if}\left(L_{ea} \geq L_{e2}, \text{"OK"}, \text{"Not OK"}\right)$

2.3. Load Case 3

min. length of embedment in resistant zone
$$L_{e3} := \text{if}\left(\frac{T_{max3}}{\phi_{GG3} \cdot F \cdot \alpha \cdot \sigma_v \cdot C \cdot R_c} \le 1\ m, 1\ m, \frac{T_{max3}}{\phi_{GG3} \cdot F \cdot \alpha \cdot \sigma_v \cdot C \cdot R_c}\right)$$

min. length of embedment in active zone
$$L_a := (H - Z) \cdot (\tan(90.\ deg - \psi) - \tan(\theta_1 - 90.\ deg))$$

Pullout Check
$$\textbf{PulloutCheck3} := \text{if}\left(L_r \ge L_{e3} + L_a, \text{"OK"}, \text{"Not OK"}\right)$$

$$\textbf{PulloutCheck3} = \text{"OK"}$$

2.4. Load Case 4

Note.

1. Soil reinforcement be designed for a pullout impact load equivalent to a static load of 19.0 kN/m.

2. The second layer be designed with a pullout impact load equivalent to a static load of 8.8 kN/m.

Factored impact load for pullout
$$T_{IP} := \text{if}\left(n = N - 1, 19.0\ \frac{kN}{m}, \text{if}\left(n = N - 2, 8.8\ \frac{kN}{m}, 0\ \frac{kN}{m}\right)\right)$$

Total factored load (static = dynamic)
$$T_{total4} := T_{max4} + T_{IP}$$

min. length of embedment in resistant zone
$$L_{e4} := \text{if}\left(\frac{T_{total4}}{\phi_{GG4} \cdot 0.8 \cdot F \cdot \alpha \cdot \sigma_v \cdot C \cdot R_c} \le 1\ m, 1\ m, \frac{T_{total4}}{\phi_{GG4} \cdot 0.8 \cdot F \cdot \alpha \cdot \sigma_v \cdot C \cdot R_c}\right)$$

Top layer to be extended and wrapped within the liner support fill to length of 1.5 m
$$l_{add} := \text{if}(n = N, Z, 0)$$

Available length of embedment in resistant zone
$$L_{ea} := L_r - L_a + l_{add}$$

Pullout Check
$$\textbf{PulloutCheck4} := \text{if}\left(L_{ea} \ge L_{e4}, \text{"OK"}, \text{"Not OK"}\right)$$

$$\textbf{PulloutCheck4} = \text{"OK"}$$

3. Design Check - Internal Stability

The check is only carried above for a reinforcement at the depth of Z. Table attached below details the check for the all reinforement within MSE wall for Load Cases 1 to 4.

10.1. Load Case 1

TensileCheck1 = "OK"

PulloutCheck1 = "OK"

10.2. Load Case 2

TensileCheck2 = "OK"

PulloutCheck2 = "OK"

10.3. Load Case 3

TensileCheck3 = "OK"

PulloutCheck3 = "OK"

10.4. Load Case 4

TensileCheck4 = "OK"

PulloutCheck4 = "OK"

DESIGN OF MECHNICALLY STABILISED EARTH (MSE) WALLS - Load Case 1			
Chainage	**200**		
Wall Geometry			
Wall Height above ground, h (m)	11.83		
Embedment Depth, d (m)	3		
Top Width, W (m)	12		
Height of MSE Wall, H (m)	14.83		
Bottom Width of MSE Wall, L	14.8		
Top width of wall, w	11		
Slope of backfill behind wall (Deg), β	14		
Face inclination from horizontal (Deg), θ_I	108.4		
Ratio	1	0.75	
Reinforced Soil Block Parameters			
Eff. Frcition of reinforced soil (Deg), ϕ'_r	32		
Unit weight of reinfoced soil (kN/m^3), γ_r	20		
Load and Resistance factors			
Traffic surcharge, q_L	20		
Live load factor , γ_{LS}	1.75		
Maximum vertcial earth pressure factor γ_{EVmax}	1.35		
Load Case	1		
Geogrid paramaters			
	Top	**Middle**	**Bottom**
Ultimate strength of reinforcement, $_{Tult}$	GG120	GG200	GG200
Vertical spacing	0.6	0.6	0.3
Length	10.381	10.381	10.381
No of reinforcment layer	10	10	8
Partial factor - creep rupture - RF_{CR}	1.45	1.45	1.45
Partial factor - construction damage - RF_{ID}	1.1	1.1	1.1
Partial factor - environmental effects - RF_D	1.05	1.05	1.05
Resistance Factors for tensile and pullout resistance, ϕ_{GG}	0.9		
Pullout resistance factor, F*	0.42		
Scale correction factor, α	0.8		
Coverage ratio, R_c	1		
C	2		
	94.3		
Batter angle, θ (Deg)	94.3		
Angle of fric between retained backfill and rein. Soil (Deg), δ	14		
Rankine active earth pressure coefficient, K_a	0.28		
lateral earth pressure coefficient, K_r	0.28		
Surcharge equivalent height, h_{eq} (m)	1		
Inclination of failure surface with horizontal, ψ (Deg)	53.1	54.6	

Internal Stability with Respect to Tensile Failure of Reinforcement - Load Case 1									
Layer #	**Z (m)**	**σ_H (kPa)**	**S_v (m)**	**T_{max} (kN/m)**	**Rein. Type**	**T_{ult} (kN/m)**	**T_{al} (kN/m)**	**T_r (kN/m)**	**Check**
1	14.8	119.67	0.15	19.75	GG200	200	119.42	107.48	OK
2	14.5	117.41	0.3	38.74	GG200	200	119.42	107.48	OK
3	14.2	115.14	0.3	38.00	GG200	200	119.42	107.48	OK
4	13.9	112.87	0.3	37.25	GG200	200	119.42	107.48	OK
5	13.6	110.60	0.3	36.50	GG200	200	119.42	107.48	OK
6	13.3	108.33	0.3	35.75	GG200	200	119.42	107.48	OK
7	13.0	106.07	0.3	35.00	GG200	200	119.42	107.48	OK
8	12.7	103.80	0.45	51.38	GG200	200	119.42	107.48	OK
9	12.1	99.26	0.6	65.51	GG200	200	119.42	107.48	OK
10	11.5	94.73	0.6	62.52	GG200	200	119.42	107.48	OK
11	10.9	90.19	0.6	59.53	GG200	200	119.42	107.48	OK
12	10.3	85.65	0.6	56.53	GG200	200	119.42	107.48	OK
13	9.7	81.12	0.6	53.54	GG200	200	119.42	107.48	OK
14	9.1	76.58	0.6	50.54	GG200	200	119.42	107.48	OK
15	8.5	72.05	0.6	47.55	GG200	200	119.42	107.48	OK
16	7.9	67.51	0.6	44.56	GG200	200	119.42	107.48	OK
17	7.3	62.97	0.6	41.56	GG200	200	119.42	107.48	OK
18	6.7	58.44	0.6	38.57	GG200	200	119.42	107.48	OK
19	6.1	53.90	0.6	35.58	GG120	120	71.65	64.49	OK
20	5.5	49.37	0.6	32.58	GG120	120	71.65	64.49	OK
21	4.9	44.83	0.6	29.59	GG120	120	71.65	64.49	OK
22	4.3	40.29	0.6	26.59	GG120	120	71.65	64.49	OK
23	3.7	35.76	0.6	23.60	GG120	120	71.65	64.49	OK
24	3.1	31.22	0.6	20.61	GG120	120	71.65	64.49	OK
25	2.5	26.69	0.6	17.61	GG120	120	71.65	64.49	OK
26	1.9	22.15	0.6	14.62	GG120	120	71.65	64.49	OK
27	1.3	17.61	0.6	11.63	GG120	120	71.65	64.49	OK
28	0.7	13.08	1.03	14.82	GG120	120	71.65	64.49	OK

Internal Stability with Respect to Pullout Failure of Reinforcement - Load Case 1							
Layer #	**Z (m)**	**σ'_v (kPa)**	**T_{max} (kN/m)**	**L_e (m)**	**L_a (m)**	**L (m)**	**Check**
1	14.8	296.6	19.75	1.00	0	10.4	OK
2	14.5	290.6	38.74	1.00	0.2	10.4	OK
3	14.2	284.6	38.00	1.00	0.3	10.4	OK
4	13.9	278.6	37.25	1.00	0.4	10.4	OK
5	13.6	272.6	36.50	1.00	0.6	10.4	OK
6	13.3	266.6	35.75	1.00	0.7	10.4	OK
7	13.0	260.6	35.00	1.00	0.8	10.4	OK
8	12.7	254.6	51.38	1.00	0.9	10.4	OK
9	12.1	242.6	65.51	1.00	1.2	10.4	OK
10	11.5	230.6	62.52	1.00	1.4	10.4	OK
11	10.9	218.6	59.53	1.00	1.7	10.4	OK
12	10.3	206.6	56.53	1.00	1.9	10.4	OK
13	9.7	194.6	53.54	1.00	2.2	10.4	OK
14	9.1	182.6	50.54	1.00	2.4	10.4	OK
15	8.5	170.6	47.55	1.00	2.7	10.4	OK
16	7.9	158.6	44.56	1.00	2.9	10.4	OK
17	7.3	146.6	41.56	1.00	3.2	10.4	OK
18	6.7	134.6	38.57	1.00	3.4	10.4	OK
19	6.1	122.6	35.58	1.00	3.7	10.4	OK
20	5.5	110.6	32.58	1.00	3.9	10.4	OK
21	4.9	98.6	29.59	1.00	4.2	10.4	OK
22	4.3	86.6	26.59	1.00	4.4	10.4	OK
23	3.7	74.6	23.60	1.00	4.7	10.4	OK
24	3.1	62.6	20.61	1.00	4.9	10.4	OK
25	2.5	50.6	17.61	1.00	5.2	10.4	OK
26	1.9	38.6	14.62	1.00	5.4	10.4	OK
27	1.3	26.6	11.63	1.00	5.7	10.4	OK
28	0.7	14.6	14.82	1.68	5.9	10.4	OK

DESIGN OF MECHNICALLY STABILISED EARTH (MSE) WALLS - Load Case 2

Earthquake Parameters			
Peak ground acceleration coefficient, a	0.08		
Horizontal coefficient of acceleration, a_h	0.04		
Hor. Coeffi. of acceleration with amplification of motion, a_{mh}	0.056		
Weight of active zone, W_a	920		
Load and Resistance factors			
Traffic surcharge, q_L	20		
Live load factor , γ_{EQ}	1.00		
Maximum vertcial earth pressure factor γ_{EVmax}	1.35		
Load Case	2		
Resistance Factors for tensile and pullout resistance, ϕ_{GG}	1.2		
Surcharge equivalent height, h_{eq} (m)	0.74		

Internal Stability with Respect to Tensile Failure of Reinforcement - Load Case 2									
Layer #	Z (m)	σ_H (kPa)	S_v (m)	T_{max} (kN/m)	T_{md} (kN/m)	T_{ult} (kN/m)	S_{rs} (kN/m)	S_{rt} (kN/m)	Check
1	14.8	117.71	0.15	19.42	1.92	200	27.11	1.85	OK
2	14.5	115.44	0.3	38.10	1.92	200	53.17	1.85	OK
3	14.2	113.17	0.3	37.35	1.92	200	52.12	1.85	OK
4	13.9	110.91	0.3	36.60	1.92	200	51.08	1.85	OK
5	13.6	108.64	0.3	35.85	1.92	200	50.03	1.85	OK
6	13.3	106.37	0.3	35.10	1.92	200	48.99	1.85	OK
7	13.0	104.10	0.3	34.35	1.92	200	47.94	1.85	OK
8	12.7	101.83	0.45	50.41	1.92	200	70.35	1.85	OK
9	12.1	97.30	0.6	64.22	1.92	200	89.62	1.85	OK
10	11.5	92.76	0.6	61.22	1.92	200	85.44	1.85	OK
11	10.9	88.23	0.6	58.23	1.92	200	81.27	1.85	OK
12	10.3	83.69	0.6	55.23	1.92	200	77.09	1.85	OK
13	9.7	79.15	0.6	52.24	1.92	200	72.91	1.85	OK
14	9.1	74.62	0.6	49.25	1.92	200	68.73	1.85	OK
15	8.5	70.08	0.6	46.25	1.92	200	64.55	1.85	OK
16	7.9	65.55	0.6	43.26	1.92	200	60.37	1.85	OK
17	7.3	61.01	0.6	40.27	1.92	200	56.20	1.85	OK
18	6.7	56.47	0.6	37.27	1.92	200	52.02	1.85	OK
19	6.1	51.94	0.6	34.28	1.92	120	47.84	1.85	OK
20	5.5	47.40	0.6	31.28	1.92	120	43.66	1.85	OK
21	4.9	42.87	0.6	28.29	1.92	120	39.48	1.85	OK
22	4.3	38.33	0.6	25.30	1.92	120	35.31	1.85	OK
23	3.7	33.79	0.6	22.30	1.92	120	31.13	1.85	OK
24	3.1	29.26	0.6	19.31	1.92	120	26.95	1.85	OK
25	2.5	24.72	0.6	16.32	1.92	120	22.77	1.85	OK
26	1.9	20.19	0.6	13.32	1.92	120	18.59	1.85	OK
27	1.3	15.65	0.6	10.33	1.92	120	14.41	1.85	OK
28	0.7	11.11	1.03	12.59	1.92	120	17.57	1.85	OK

Internal Stability with Respect to Pullout Failure of Reinforcement - Load Case 2							
Layer #	**Z (m)**	**σ'_v (kPa)**	**T_{total} (kN/m)**	**Required L_e (m)**	**L (m)**	**Available L_e (m)**	**Check**
1	14.8	296.6	21.34	1.00	10.4	10.381	OK
2	14.5	290.6	40.02	1.00	10.4	10.281	OK
3	14.2	284.6	39.27	1.00	10.4	10.081	OK
4	13.9	278.6	38.52	1.00	10.4	9.981	OK
5	13.6	272.6	37.77	1.00	10.4	9.881	OK
6	13.3	266.6	37.02	1.00	10.4	9.781	OK
7	13.0	260.6	36.27	1.00	10.4	9.581	OK
8	12.7	254.6	52.33	1.00	10.4	9.481	OK
9	12.1	242.6	66.14	1.00	10.4	9.281	OK
10	11.5	230.6	63.14	1.00	10.4	8.981	OK
11	10.9	218.6	60.15	1.00	10.4	8.781	OK
12	10.3	206.6	57.16	1.00	10.4	8.481	OK
13	9.7	194.6	54.16	1.00	10.4	8.281	OK
14	9.1	182.6	51.17	1.00	10.4	7.981	OK
15	8.5	170.6	48.17	1.00	10.4	7.781	OK
16	7.9	158.6	45.18	1.00	10.4	7.481	OK
17	7.3	146.6	42.19	1.00	10.4	7.281	OK
18	6.7	134.6	39.19	1.00	10.4	6.981	OK
19	6.1	122.6	36.20	1.00	10.4	6.781	OK
20	5.5	110.6	33.21	1.00	10.4	6.481	OK
21	4.9	98.6	30.21	1.00	10.4	6.281	OK
22	4.3	86.6	27.22	1.00	10.4	5.981	OK
23	3.7	74.6	24.22	1.00	10.4	5.781	OK
24	3.1	62.6	21.23	1.00	10.4	5.481	OK
25	2.5	50.6	18.24	1.00	10.4	5.281	OK
26	1.9	38.6	15.24	1.00	10.4	4.981	OK
27	1.3	26.6	12.25	1.00	10.4	4.781	OK
28	0.7	14.6	14.51	1.54	10.4	4.481	OK

DESIGN OF MECHNICALLY STABILISED EARTH (MSE) WALLS - Load Case 3

Wall Geometry			
Wall Height (m), H	14.83		
Bottom width of wall, L	14.83		
Top width of wall, w	11		
Slope of backfill behind wall (Deg), β	14		
Face inclination from horizontal (Deg), θ_I	108.4		
Reinforced Soil Block Parameters			
Eff. Frcition of reinforced soil (Deg), ϕ'_r	32		
Unit weight of reinfoced soil (kN/m^3), γ_r	20		
Load and Resistance factors			
Traffic surcharge, q_L	10		
Live load factor , γ_{LS}	1.75		
Maximum vertcial earth pressure factor γ_{EVmax}	1.35		
Load Case	3		
Geogrid paramaters			
	Top	**Middle**	**Bottom**
Ultimate strength of reinforcement, $_{Tult}$	GG120	GG200	GG200
Vertical spacing	0.6	0.6	0.3
Length	10.381	10.381	10.381
No of reinforcment layer	10	10	8
Partial factor - creep rupture - RF_{CR}	1.00	1.00	1.00
Partial factor - construction damage - RF_{ID}	1.1	1.1	1.1
Partial factor - environmental effects - RF_D	1.05	1.05	1.05
Resistance Factors for tensile and pullout resistance, ϕ_{GG}	0.9		
Pullout resistance factor, F*	0.42		
Scale correction factor, α	0.8		
Coverage ratio, R_c	1		
C	2		
Batter angle, θ (Deg)	94.3		
Angle of fric between retained backfill and rein. Soil (Deg), δ	14		
Rankine active earth pressure coefficient, K_a	0.28		
lateral earth pressure coefficient, K_r	0.28		
Surcharge equivalent height, h_{eq} (m)	0.65		
Inclination of failure surface with horizontal, ψ (Deg)	53.1		
Note:			
1. For extensible reinforcement (geogrid), lateral stress ratio is equla to 1.0			

Internal Stability with Respect to Tensile Failure of Reinforcement - Load Case 3

Layer #	Z (m)	σ_H (kPa)	S_v (m)	T_{max} (kN/m)	Rein. Type	T_{ult} (kN/m)	T_{al} (kN/m)	T_r (kN/m)	Check
1	14.8	117.03	0.15	19.31	GG200	200	173.16	155.84	OK
2	14.5	114.76	0.3	37.87	GG200	200	173.16	155.84	OK
3	14.2	112.49	0.3	37.12	GG200	200	173.16	155.84	OK
4	13.9	110.22	0.3	36.37	GG200	200	173.16	155.84	OK
5	13.6	107.96	0.3	35.63	GG200	200	173.16	155.84	OK
6	13.3	105.69	0.3	34.88	GG200	200	173.16	155.84	OK
7	13.0	103.42	0.3	34.13	GG200	200	173.16	155.84	OK
8	12.7	101.15	0.45	50.07	GG200	200	173.16	155.84	OK
9	12.1	96.62	0.6	63.77	GG200	200	173.16	155.84	OK
10	11.5	92.08	0.6	60.77	GG200	200	173.16	155.84	OK
11	10.9	87.54	0.6	57.78	GG200	200	173.16	155.84	OK
12	10.3	83.01	0.6	54.79	GG200	200	173.16	155.84	OK
13	9.7	78.47	0.6	51.79	GG200	200	173.16	155.84	OK
14	9.1	73.94	0.6	48.80	GG200	200	173.16	155.84	OK
15	8.5	69.40	0.6	45.80	GG200	200	173.16	155.84	OK
16	7.9	64.86	0.6	42.81	GG200	200	173.16	155.84	OK
17	7.3	60.33	0.6	39.82	GG200	200	173.16	155.84	OK
18	6.7	55.79	0.6	36.82	GG200	200	173.16	155.84	OK
19	6.1	51.26	0.6	33.83	GG120	120	103.90	93.51	OK
20	5.5	46.72	0.6	30.84	GG120	120	103.90	93.51	OK
21	4.9	42.18	0.6	27.84	GG120	120	103.90	93.51	OK
22	4.3	37.65	0.6	24.85	GG120	120	103.90	93.51	OK
23	3.7	33.11	0.6	21.85	GG120	120	103.90	93.51	OK
24	3.1	28.58	0.6	18.86	GG120	120	103.90	93.51	OK
25	2.5	24.04	0.6	15.87	GG120	120	103.90	93.51	OK
26	1.9	19.50	0.6	12.87	GG120	120	103.90	93.51	OK
27	1.3	14.97	0.6	9.88	GG120	120	103.90	93.51	OK
28	0.7	10.43	1.03	11.82	GG120	120	103.90	93.51	OK

Internal Stability with Respect to Pullout Failure of Reinforcement - Load Case 3							
Layer #	**Z (m)**	**σ'_v (kPa)**	**T_{max} (kN/m)**	**L_e (m)**	**L_a (m)**	**L (m)**	**Check**
1	14.8	296.6	19.31	1.00	0	10.4	OK
2	14.5	290.6	37.87	1.00	0.2	10.4	OK
3	14.2	284.6	37.12	1.00	0.3	10.4	OK
4	13.9	278.6	36.37	1.00	0.4	10.4	OK
5	13.6	272.6	35.63	1.00	0.6	10.4	OK
6	13.3	266.6	34.88	1.00	0.7	10.4	OK
7	13.0	260.6	34.13	1.00	0.8	10.4	OK
8	12.7	254.6	50.07	1.00	0.9	10.4	OK
9	12.1	242.6	63.77	1.00	1.2	10.4	OK
10	11.5	230.6	60.77	1.00	1.4	10.4	OK
11	10.9	218.6	57.78	1.00	1.7	10.4	OK
12	10.3	206.6	54.79	1.00	1.9	10.4	OK
13	9.7	194.6	51.79	1.00	2.2	10.4	OK
14	9.1	182.6	48.80	1.00	2.4	10.4	OK
15	8.5	170.6	45.80	1.00	2.7	10.4	OK
16	7.9	158.6	42.81	1.00	2.9	10.4	OK
17	7.3	146.6	39.82	1.00	3.2	10.4	OK
18	6.7	134.6	36.82	1.00	3.4	10.4	OK
19	6.1	122.6	33.83	1.00	3.7	10.4	OK
20	5.5	110.6	30.84	1.00	3.9	10.4	OK
21	4.9	98.6	27.84	1.00	4.2	10.4	OK
22	4.3	86.6	24.85	1.00	4.4	10.4	OK
23	3.7	74.6	21.85	1.00	4.7	10.4	OK
24	3.1	62.6	18.86	1.00	4.9	10.4	OK
25	2.5	50.6	15.87	1.00	5.2	10.4	OK
26	1.9	38.6	12.87	1.00	5.4	10.4	OK
27	1.3	26.6	9.88	1.00	5.7	10.4	OK
28	0.7	14.6	11.82	1.34	5.9	10.4	OK

DESIGN OF MECHNICALLY STABILISED EARTH (MSE) WALLS - Load Case 4

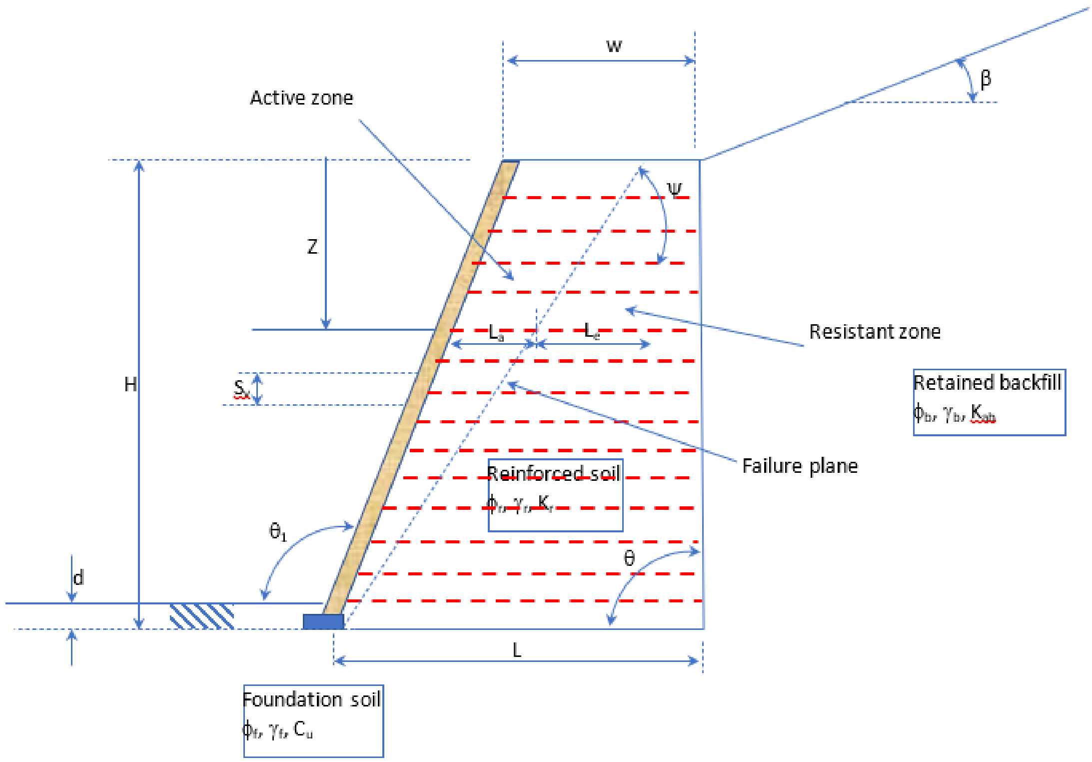
Load and Resistance factors			
Traffic surcharge, q_L	20		
Live load factor, , γ_{LS}	1.00		
Maximum vertcial earth pressure factor γ_{EVmax}	1.35		
Load Case	4		
Resistance Factors for tensile and pullout resistance, ϕ_{GG}	1		
Surcharge equivalent height, h_{eq} (m)	1		
Reinforcement Rupture			
Factored impact load on 1st layer (kN/m)	33.5		
Factored impact load on 2nd layer (kN/m)	8.8		
Reinforcement Pullout			
Factored impact load on 1st layer (kN/m)	19		
Factored impact load on 2nd layer (kN/m)	8.8		

Internal Stability with Respect to Tensile Failure of Reinforcement - Load Case 4									
Layer #	**Z (m)**	**σ_H (kPa)**	**S_v (m)**	**T_{max} (kN/m)**	**T_l(kN/m)**	**T_{ult} (kN/m)**	**S_{rs} (kN/m)**	**S_{rt} (kN/m)**	**Check**
1	14.8	119.67	0.15	19.75	0.00	200	33.07	0.00	OK
2	14.5	117.41	0.3	38.74	0.00	200	64.89	0.00	OK
3	14.2	115.14	0.3	38.00	0.00	200	63.63	0.00	OK
4	13.9	112.87	0.3	37.25	0.00	200	62.38	0.00	OK
5	13.6	110.60	0.3	36.50	0.00	200	61.13	0.00	OK
6	13.3	108.33	0.3	35.75	0.00	200	59.87	0.00	OK
7	13.0	106.07	0.3	35.00	0.00	200	58.62	0.00	OK
8	12.7	103.80	0.45	51.38	0.00	200	86.05	0.00	OK
9	12.1	99.26	0.6	65.51	0.00	200	109.72	0.00	OK
10	11.5	94.73	0.6	62.52	0.00	200	104.70	0.00	OK
11	10.9	90.19	0.6	59.53	0.00	200	99.69	0.00	OK
12	10.3	85.65	0.6	56.53	0.00	200	94.68	0.00	OK
13	9.7	81.12	0.6	53.54	0.00	200	89.66	0.00	OK
14	9.1	76.58	0.6	50.54	0.00	200	84.65	0.00	OK
15	8.5	72.05	0.6	47.55	0.00	200	79.64	0.00	OK
16	7.9	67.51	0.6	44.56	0.00	200	74.62	0.00	OK
17	7.3	62.97	0.6	41.56	0.00	200	69.61	0.00	OK
18	6.7	58.44	0.6	38.57	0.00	200	64.59	0.00	OK
19	6.1	53.90	0.6	35.58	0.00	120	59.58	0.00	OK
20	5.5	49.37	0.6	32.58	0.00	120	54.57	0.00	OK
21	4.9	44.83	0.6	29.59	0.00	120	49.55	0.00	OK
22	4.3	40.29	0.6	26.59	0.00	120	44.54	0.00	OK
23	3.7	35.76	0.6	23.60	0.00	120	39.53	0.00	OK
24	3.1	31.22	0.6	20.61	0.00	120	34.51	0.00	OK
25	2.5	26.69	0.6	17.61	0.00	120	29.50	0.00	OK
26	1.9	22.15	0.6	14.62	8.80	120	24.48	10.16	OK
27	1.3	17.61	0.6	11.63	33.50	120	19.47	38.69	OK
28	0.7	13.08	1.03	14.82	0.00	120	24.82	0.00	OK

Internal Stability with Respect to Pullout Failure of Reinforcement - Load Case 4								
Layer #	**Z (m)**	**σ'_v (kPa)**	**T_l(kN/m)**	**T_{total} (kN/m)**	**Required L_e (m)**	**L (m)**	**Available L_e (m)**	**Check**
1	14.8	296.6	0	19.75	1.00	10.4	10.38	OK
2	14.5	290.6	0	38.74	1.00	10.4	10.26	OK
3	14.2	284.6	0	38.00	1.00	10.4	10.13	OK
4	13.9	278.6	0	37.25	1.00	10.4	10.00	OK
5	13.6	272.6	0	36.50	1.00	10.4	9.88	OK
6	13.3	266.6	0	35.75	1.00	10.4	9.75	OK
7	13.0	260.6	0	35.00	1.00	10.4	9.63	OK
8	12.7	254.6	0	51.38	1.00	10.4	9.50	OK
9	12.1	242.6	0	65.51	1.00	10.4	9.25	OK
10	11.5	230.6	0	62.52	1.00	10.4	9.00	OK
11	10.9	218.6	0	59.53	1.00	10.4	8.75	OK
12	10.3	206.6	0	56.53	1.00	10.4	8.50	OK
13	9.7	194.6	0	53.54	1.00	10.4	8.25	OK
14	9.1	182.6	0	50.54	1.00	10.4	8.00	OK
15	8.5	170.6	0	47.55	1.00	10.4	7.75	OK
16	7.9	158.6	0	44.56	1.00	10.4	7.50	OK
17	7.3	146.6	0	41.56	1.00	10.4	7.24	OK
18	6.7	134.6	0	38.57	1.00	10.4	6.99	OK
19	6.1	122.6	0	35.58	1.00	10.4	6.74	OK
20	5.5	110.6	0	32.58	1.00	10.4	6.49	OK
21	4.9	98.6	0	29.59	1.00	10.4	6.24	OK
22	4.3	86.6	0	26.59	1.00	10.4	5.99	OK
23	3.7	74.6	0	23.60	1.00	10.4	5.74	OK
24	3.1	62.6	0	20.61	1.00	10.4	5.49	OK
25	2.5	50.6	0	17.61	1.00	10.4	5.24	OK
26	1.9	38.6	8.8	23.42	1.13	10.4	4.99	OK
27	1.3	26.6	19	30.63	2.14	10.4	4.74	OK
28	0.7	14.6	0	14.82	1.89	10.4	5.21	OK

Layer #	Z (m)	σ_H (kPa)	S (m)	T_{max} (kN/m)	σ'_v (kPa)	Required L_{loop} (m)	Provided L_{loop} (m)	Check
1	14.83	119.7	0.3	18.0	296.6	0.006	1.5	OK
2	14.53	117.4	0.3	17.6	290.6	0.006	1.5	OK
3	14.23	115.1	0.3	17.3	284.6	0.006	1.5	OK
4	13.93	112.9	0.3	16.9	278.6	0.006	1.5	OK
5	13.63	110.6	0.3	16.6	272.6	0.006	1.5	OK
6	13.33	108.3	0.3	16.3	266.6	0.006	1.5	OK
7	13.03	106.1	0.3	15.9	260.6	0.006	1.5	OK
8	12.73	103.8	0.3	15.6	254.6	0.006	1.5	OK
9	12.13	99.3	0.6	29.8	242.6	0.012	1.5	OK
10	11.53	94.7	0.6	28.4	230.6	0.012	1.5	OK
11	10.93	90.2	0.6	27.1	218.6	0.012	1.5	OK
12	10.33	0.8	0.6	0.2	206.6	0.000	1.5	OK
13	9.73	81.1	0.6	24.3	194.6	0.013	1.5	OK
14	9.13	76.6	0.6	23.0	182.6	0.013	1.5	OK
15	8.53	72.0	0.6	21.6	170.6	0.013	1.5	OK
16	7.93	67.5	0.6	20.3	158.6	0.013	1.5	OK
17	7.33	63.0	0.6	18.9	146.6	0.013	1.5	OK
18	6.73	58.4	0.6	17.5	134.6	0.013	1.5	OK
19	6.13	53.9	0.6	16.2	122.6	0.013	1.5	OK
20	5.53	49.4	0.6	14.8	110.6	0.013	1.5	OK
21	4.93	44.8	0.6	13.4	98.6	0.014	1.5	OK
22	4.33	40.3	0.6	12.1	86.6	0.014	1.5	OK
23	3.73	35.8	0.6	10.7	74.6	0.014	1.5	OK
24	3.13	31.2	0.6	9.4	62.6	0.015	1.5	OK
25	2.53	26.7	0.6	8.0	50.6	0.016	1.5	OK
26	1.93	22.2	0.6	6.6	38.6	0.017	1.5	OK
27	1.33	17.6	0.6	5.3	26.6	0.020	1.5	OK
28	0.73	13.1	0.6	3.9	14.6	0.027	1.5	OK

Internal Stability - Sample Calculations



Calculation below shows the sample calcuation of tensile and pull-out failure of reinforcement at a specific depth/layer.

Layer # $n := 20$ Refer to excel sheet below

Depth of reinforcement $Z := 14.0\ m$

Vertical reinforcement spacing/Contributory height $S_v := 0.3\ m$

Ultimate Tensile Strength $T_{ult} := 200\ \dfrac{kN}{m}$ For GG200

Length of reinforcement $L_r := 12.9\ m$

Total no of reinforcement layers within the wall $N := 43$ To be estimated based on geometry and spacing

Partial factor - creep rupture $RF_{CR} := 1.45$

Partial factor - construction damage $RF_{ID} := 1.1$

Partial factor - environmental effects $RF_D := 1.05$

Combined strength reduction factor $RF := RF_{CR} \cdot RF_{ID} \cdot RF_D$

Pullout resistance factor $F := 0.42$

Scale correction factor $\alpha := 0.8$

Coverage ratio $R_c := 1$ $C := 2$

Rankine active earth pressure coefficient

$$K_a := \frac{\sin(\theta + \phi_r)^2}{\sin(\theta)^3 \cdot \left(1 + \frac{\sin(\phi_r)}{\sin(\theta)}\right)^2}$$

lateral earth pressure coefficient $K_r := K_a$ Extensible reinforcement (geogrid)

$a_1 := \tan(\phi_r - \beta)$ $a_2 := \cot(\phi_r + \theta - 90.\ deg)$ $a_3 := \tan(\delta + 90.\ deg - \theta)$

Inclination of failure plane

$$\psi := \operatorname{atan}\left(\frac{-a_1 + \sqrt{a_1 \cdot (a_1 + a_2)\ (1 + a_3 \cdot a_2)}}{1 + a_3 \cdot (a_1 + a_2)}\right) + \phi_r$$

1. Tensile Failure of Reinforcement

1.1. Load Case 1

Surcharge equivalent height

$$h_{eq1} := \frac{q_L \cdot \gamma_{LS}}{\gamma_r \cdot \gamma_{EVmax}}$$

Horizontal stress at depth Z

$$\sigma_{H1} := K_r \cdot \gamma_r \cdot (Z + h_{eq1}) \cdot \gamma_{EVmax}$$

Maximum factored tension

$$T_{max1} := \sigma_{H1} \cdot S_v$$

Nominal long-term reinforcement strength

$$T_{al1} := \frac{T_{ult}}{RF_{CR} \cdot RF_{ID} \cdot RF_D}$$

Resistance Factors for tensile and pullout resistance

$$\phi_{GG1} := 0.9$$

Factored tensile resistance $T_{r1} := \phi_{GG1} \cdot T_{al1}$

Tensile Check $\mathbf{TensileCheck1} := \mathbf{if}\left(T_{r1} \geq T_{max1}, \text{"OK"}, \text{"Not OK"}\right)$

$\mathbf{TensileCheck1} = \text{"OK"}$

1.2. Load Case 2

Surcharge equivalent height $h_{eq2} := \dfrac{q_L \cdot \gamma_{EQ}}{\gamma_r \cdot \gamma_{EVmax}}$

Horizontal stress at depth Z $\sigma_{H2} := K_r \cdot \gamma_r \cdot \left(Z + h_{eq2}\right) \cdot \gamma_{EVmax}$

Maximum factored tension $T_{max2} := \sigma_{H2} \cdot S_v$

Soil weight of the active zone $W_a := \dfrac{1}{2} \cdot \gamma_r \cdot H^2 \cdot \left(\tan\left(90.\ deg - \psi\right) - \tan\left(\theta_1 - 90.\ deg\right)\right)$

Factored incremental dynamic inertia force $T_{md} := \dfrac{a_{mh} \cdot W_a}{N - 1}$

Resistance Factors for tensile and pullout resistance $\phi_{GG2} := 1.2$

Static component of resistance $S_{rs2} := \dfrac{T_{max2} \cdot RF}{\phi_{GG2} \cdot R_c}$

Dynamic component of resistance $S_{rt2} := \dfrac{T_{md} \cdot RF_{ID} \cdot RF_D}{\phi_{GG2} \cdot R_c}$

Tensile Check $\mathbf{TensileCheck2} := \mathbf{if}\left(T_{ult} \geq \left(S_{rs2} + S_{rt2}\right), \text{"OK"}, \text{"Not OK"}\right)$

$\mathbf{TensileCheck2} = \text{"OK"}$

1.3. Load Case 3

Surcharge equivalent height $h_{eq3} := \dfrac{q_c \cdot \gamma_{LS}}{\gamma_r \cdot \gamma_{EVmax}}$

Horizontal stress at depth Z $\sigma_{H3} := K_r \cdot \gamma_r \cdot \left(Z + h_{eq3}\right) \cdot \gamma_{EVmax}$

Maximum factored tension $T_{max3} := \sigma_{H3} \cdot S_v$

Nominal long-term reinforcement strength $T_{al3} := \dfrac{T_{ult}}{RF_{ID} \cdot RF_D}$ No creep considered during construction

Resistance Factors for tensile and pullout resistance $\phi_{GG3} := 0.9$

Factored tensile resistance $T_{r3} := \phi_{GG3} \cdot T_{al3}$

Tensile Check $\mathbf{TensileCheck3} := \mathbf{if}\left(T_{r3} \geq T_{max3}, \text{"OK"}, \text{"Not OK"}\right)$

$\mathbf{TensileCheck3} = \text{"OK"}$

1.4. Load Case 4

Surcharge equivalent height $h_{eq4} := \dfrac{q_L \cdot 1.0}{\gamma_r}$ Assumed 1 m

Horizontal stress at depth Z $\sigma_{H4} := K_r \cdot \gamma_r \cdot \left(Z + h_{eq4}\right) \cdot \gamma_{EVmax}$

Maximum factored tension $T_{max4} := \sigma_{H4} \cdot S_v$

Note

1. Traffic railing impact events tend to affect only the internal stability of MSE walls

2. The recommended static impact force is assumed 45 kN applied on a barrier with a minimum height of 810 mm above the road surface.

3. As per FHWA NHI-10-024, the static impact force, adds an additional horizontal force to the upper 2 layers of soil reinforcement.

4. The upper layer of soil reinforcement be designed for a rupture impact load equivalent to a static load of 33.5 kN/m of wall.

5. The second layer be designed with a rupture impact load equivalent to a static load of 8.8 kN/m.

Factored impact load $T_I := \mathbf{if}\left(n = N - 1, 33.5\ \dfrac{kN}{m}, \mathbf{if}\left(n = N - 2, 8.8\ \dfrac{kN}{m}, 0\ \dfrac{kN}{m}\right)\right)$

Resistance Factors for tensile and pullout resistance $\phi_{GG4} := 1.0$

Static component of resistance $$S_{rs4} := \frac{T_{max4} \cdot RF}{\phi_{GG4} \cdot R_c}$$

Dynamic component of resistance $$S_{rt4} := \frac{T_I \cdot RF_{ID} \cdot RF_D}{\phi_{GG2} \cdot R_c}$$

Tensile Check $$\mathbf{TensileCheck4} := \mathbf{if}\left(T_{ult} \ge \left(S_{rs4} + S_{rt4}\right), \text{“OK”}, \text{“Not OK”}\right)$$

$$\mathbf{TensileCheck4} = \text{“OK”}$$

2. Pullout Failure of Reinforcement

2.1. Load Case 1

Nominal vertical stress at depth Z $$\sigma_v := \gamma_r \cdot Z$$

min. length of embedment in resistant zone $$L_{e1} := \mathbf{if}\left(\frac{T_{max1}}{\phi_{GG1} \cdot F \cdot \alpha \cdot \sigma_v \cdot C \cdot R_c} \le 1\ m, 1\ m, \frac{T_{max1}}{\phi_{GG1} \cdot F \cdot \alpha \cdot \sigma_v \cdot C \cdot R_c}\right)$$

min. length of embedment in active zone $$L_a := (H - Z) \cdot \left(\tan\left(90.\ deg - \psi\right) - \tan\left(\theta_1 - 90.\ deg\right)\right)$$

Pullout Check $$\mathbf{PulloutCheck1} := \mathbf{if}\left(L_r \ge L_{e1} + L_a, \text{“OK”}, \text{“Not OK”}\right)$$

$$\mathbf{PulloutCheck1} = \text{“OK”}$$

2.2. Load Case 2

Total factored load (static = dynamic) $$T_{total2} := T_{max2} + T_{md}$$

min. length of embedment in resistant zone

$$L_{e2} := \mathbf{if}\left(\frac{T_{total2}}{\phi_{GG2} \cdot 0.8 \cdot F \cdot \alpha \cdot \sigma_v \cdot C \cdot R_c} \le 1\ m, 1\ m, \frac{T_{total2}}{\phi_{GG2} \cdot 0.8 \cdot F \cdot \alpha \cdot \sigma_v \cdot C \cdot R_c}\right)$$

Available length of embedment in resistant zone $$L_{ea} := L_r - L_a$$

Pullout Check $$\mathbf{PulloutCheck2} := \mathbf{if}\left(L_{ea} \ge L_{e2}, \text{“OK”}, \text{“Not OK”}\right)$$

2.3. Load Case 3

min. length of embedment in resistant zone

$$L_{e3} := \text{if}\left(\frac{T_{max3}}{\phi_{GG3} \cdot F \cdot \alpha \cdot \sigma_v \cdot C \cdot R_c} \leq 1\ m, 1\ m, \frac{T_{max3}}{\phi_{GG3} \cdot F \cdot \alpha \cdot \sigma_v \cdot C \cdot R_c}\right)$$

min. length of embedment in active zone

$$L_a := (H - Z) \cdot (\tan(90.\ deg - \psi) - \tan(\theta_1 - 90.\ deg))$$

Pullout Check

$$\textbf{PulloutCheck3} := \text{if}\left(L_r \geq L_{e3} + L_a, \text{“OK”}, \text{“Not OK”}\right)$$

$$\textbf{PulloutCheck3} = \text{“OK”}$$

2.4. Load Case 4

Note.

1. Soil reinforcement be designed for a pullout impact load equivalent to a static load of 19.0 kN/m.

2. The second layer be designed with a pullout impact load equivalent to a static load of 8.8 kN/m.

Factored impact load for pullout

$$T_{IP} := \text{if}\left(n = N - 1, 19.0\ \frac{kN}{m}, \text{if}\left(n = N - 2, 8.8\ \frac{kN}{m}, 0\ \frac{kN}{m}\right)\right)$$

Total factored load (static = dynamic)

$$T_{total4} := T_{max4} + T_{IP}$$

min. length of embedment in resistant zone

$$L_{e4} := \text{if}\left(\frac{T_{total4}}{\phi_{GG4} \cdot 0.8 \cdot F \cdot \alpha \cdot \sigma_v \cdot C \cdot R_c} \leq 1\ m, 1\ m, \frac{T_{total4}}{\phi_{GG4} \cdot 0.8 \cdot F \cdot \alpha \cdot \sigma_v \cdot C \cdot R_c}\right)$$

Top layer to be extended and wrapped within the liner support fill to length of 1.5 m

$$l_{add} := \text{if}(n = N, Z, 0)$$

Available length of embedment in resistant zone

$$L_{ea} := L_r - L_a + l_{add}$$

Pullout Check

$$\textbf{PulloutCheck4} := \text{if}\left(L_{ea} \geq L_{e4}, \text{“OK”}, \text{“Not OK”}\right)$$

$$\textbf{PulloutCheck4} = \text{“OK”}$$

3. Design Check - Internal Stability

The check is only carried above for a reinforcement at the depth of Z. Table attached below details the check for the all reinforement within MSE wall for Load Cases 1 to 4.

10.1. Load Case 1

TensileCheck1 = “OK”

PulloutCheck1 = “OK”

10.2. Load Case 2

TensileCheck2 = “OK”

PulloutCheck2 = “OK”

10.3. Load Case 3

TensileCheck3 = “OK”

PulloutCheck3 = “OK”

10.4. Load Case 4

TensileCheck4 = “OK”

PulloutCheck4 = “OK”

DESIGN OF MECHNICALLY STABILISED EARTH (MSE) WALLS - Load Case 1

Chainage	600		
Wall Geometry			
Wall Height above ground, h (m)	17.59		
Embedment Depth, d (m)	2.1		
Top Width, W (m)	13.5		
Height of MSE Wall, H (m)	19.69		
Bottom Width of MSE Wall, L	18.4		
Top width of wall, w	12.5		
Slope of backfill behind wall (Deg), β	14		
Face inclination from horizontal (Deg), θ_I	108.4		
Ratio	1		
Reinforced Soil Block Parameters			
Eff. Frcition of reinforced soil (Deg), ϕ'_r	32		
Unit weight of reinfoced soil (kN/m^3), γ_r	20		
Load and Resistance factors			
Traffic surcharge, q_L	20		
Live load factor , γ_{LS}	1.75		
Maximum vertcial earth pressure factor γ_{EVmax}	1.35		
Load Case	1		
Geogrid paramaters			
	Top	**Middle**	**Bottom**
Ultimate strength of reinforcement, $_{Tult}$	GG120	GG200	GG200
Vertical spacing	0.6	0.6	0.3
Length	12.5	12.5	12.5
No of reinforcment layer	11	10	22
Partial factor - creep rupture - RF_{CR}	1.45	1.45	1.45
Partial factor - construction damage - RF_{ID}	1.1	1.1	1.1
Partial factor - environmental effects - RF_D	1.05	1.05	1.05
Resistance Factors for tensile and pullout resistance, ϕ_{GG}	0.9		
Pullout resistance factor, F*	0.42		
Scale correction factor, α	0.8		
Coverage ratio, R_c	1		
C	2		
	92.0		
Batter angle, θ (Deg)	92.0		
Angle of fric between retained backfill and rein. Soil (Deg), δ	14		
Rankine active earth pressure coefficient, K_a	0.294		
lateral earth pressure coefficient, K_r	0.294		
Surcharge equivalent height, h_{eq} (m)	1		
Inclination of failure surface with horizontal, ψ (Deg)	53.8	54.6	

Internal Stability with Respect to Tensile Failure of Reinforcement - Load Case 1									
Layer #	**Z (m)**	**σ_H (kPa)**	**S_v (m)**	**T_{max} (kN/m)**	**Rein. Type**	**T_{ult} (kN/m)**	**T_{al} (kN/m)**	**T_r (kN/m)**	**Check**
1	19.7	164.24	0.15	27.10	GG200	200	119.42	107.48	OK
2	19.4	161.86	0.3	53.41	GG200	200	119.42	107.48	OK
3	19.1	159.47	0.3	52.63	GG200	200	119.42	107.48	OK
4	18.8	157.09	0.3	51.84	GG200	200	119.42	107.48	OK
5	18.5	154.71	0.3	51.05	GG200	200	119.42	107.48	OK
6	18.2	152.33	0.3	50.27	GG200	200	119.42	107.48	OK
7	17.9	149.95	0.3	49.48	GG200	200	119.42	107.48	OK
8	17.6	147.57	0.3	48.70	GG200	200	119.42	107.48	OK
9	17.3	145.19	0.3	47.91	GG200	200	119.42	107.48	OK
10	17.0	142.80	0.3	47.13	GG200	200	119.42	107.48	OK
11	16.7	140.42	0.3	46.34	GG200	200	119.42	107.48	OK
12	16.4	138.04	0.3	45.55	GG200	200	119.42	107.48	OK
13	16.1	135.66	0.3	44.77	GG200	200	119.42	107.48	OK
14	15.8	133.28	0.3	43.98	GG200	200	119.42	107.48	OK
15	15.5	130.90	0.3	43.20	GG200	200	119.42	107.48	OK
16	15.2	128.52	0.3	42.41	GG200	200	119.42	107.48	OK
17	14.9	126.13	0.3	41.62	GG200	200	119.42	107.48	OK
18	14.6	123.75	0.3	40.84	GG200	200	119.42	107.48	OK
19	14.3	121.37	0.3	40.05	GG200	200	119.42	107.48	OK
20	14.0	118.99	0.3	39.27	GG200	200	119.42	107.48	OK
21	13.7	116.61	0.3	38.48	GG200	200	119.42	107.48	OK
22	13.4	114.23	0.45	56.54	GG200	200	119.42	107.48	OK
23	12.8	109.47	0.6	72.25	GG200	200	119.42	107.48	OK
24	12.2	104.70	0.6	69.10	GG200	200	119.42	107.48	OK
25	11.6	99.94	0.6	65.96	GG200	200	119.42	107.48	OK
26	11.0	95.18	0.6	62.82	GG200	200	119.42	107.48	OK
27	10.4	90.41	0.6	59.67	GG200	200	119.42	107.48	OK
28	9.8	85.65	0.6	56.53	GG200	200	119.42	107.48	OK
29	9.2	80.89	0.6	53.39	GG200	200	119.42	107.48	OK
30	8.6	76.13	0.6	50.24	GG200	200	119.42	107.48	OK
31	8.0	71.36	0.6	47.10	GG200	200	119.42	107.48	OK
32	7.4	66.60	0.6	43.96	GG200	200	119.42	107.48	OK
33	6.8	61.84	0.6	40.81	GG120	120	71.65	64.49	OK
34	6.2	57.07	0.6	37.67	GG120	120	71.65	64.49	OK
35	5.6	52.31	0.6	34.53	GG120	120	71.65	64.49	OK
36	5.0	47.55	0.6	31.38	GG120	120	71.65	64.49	OK
37	4.4	42.79	0.6	28.24	GG120	120	71.65	64.49	OK
38	3.8	38.02	0.6	25.10	GG120	120	71.65	64.49	OK
39	3.2	33.26	0.6	21.95	GG120	120	71.65	64.49	OK
40	2.6	28.50	0.6	18.81	GG120	120	71.65	64.49	OK
41	2.0	23.73	0.6	15.66	GG120	120	71.65	64.49	OK
42	1.4	18.97	0.6	12.52	GG120	120	71.65	64.49	OK
43	0.8	14.21	1.09	17.04	GG120	120	71.65	64.49	OK

Internal Stability with Respect to Pullout Failure of Reinforcement - Load Case 1							
Layer #	**Z (m)**	**σ'_v (kPa)**	**T_{max} (kN/m)**	**L_e (m)**	**L_a (m)**	**L (m)**	**Check**
1	19.7	393.8	27.10	1.00	0	12.5	OK
2	19.4	387.8	53.41	1.00	0.2	12.5	OK
3	19.1	381.8	52.63	1.00	0.3	12.5	OK
4	18.8	375.8	51.84	1.00	0.4	12.5	OK
5	18.5	369.8	51.05	1.00	0.5	12.5	OK
6	18.2	363.8	50.27	1.00	0.6	12.5	OK
7	17.9	357.8	49.48	1.00	0.8	12.5	OK
8	17.6	351.8	48.70	1.00	0.9	12.5	OK
9	17.3	345.8	47.91	1.00	1	12.5	OK
10	17.0	339.8	47.13	1.00	1.1	12.5	OK
11	16.7	333.8	46.34	1.00	1.2	12.5	OK
12	16.4	327.8	45.55	1.00	1.4	12.5	OK
13	16.1	321.8	44.77	1.00	1.5	12.5	OK
14	15.8	315.8	43.98	1.00	1.6	12.5	OK
15	15.5	309.8	43.20	1.00	1.7	12.5	OK
16	15.2	303.8	42.41	1.00	1.8	12.5	OK
17	14.9	297.8	41.62	1.00	2	12.5	OK
18	14.6	291.8	40.84	1.00	2.1	12.5	OK
19	14.3	285.8	40.05	1.00	2.2	12.5	OK
20	14.0	279.8	39.27	1.00	2.3	12.5	OK
21	13.7	273.8	38.48	1.00	2.4	12.5	OK
22	13.4	267.8	56.54	1.00	2.6	12.5	OK
23	12.8	255.8	72.25	1.00	2.8	12.5	OK
24	12.2	243.8	69.10	1.00	3	12.5	OK
25	11.6	231.8	65.96	1.00	3.3	12.5	OK
26	11.0	219.8	62.82	1.00	3.5	12.5	OK
27	10.4	207.8	59.67	1.00	3.8	12.5	OK
28	9.8	195.8	56.53	1.00	4	12.5	OK
29	9.2	183.8	53.39	1.00	4.2	12.5	OK
30	8.6	171.8	50.24	1.00	4.5	12.5	OK
31	8.0	159.8	47.10	1.00	4.7	12.5	OK
32	7.4	147.8	43.96	1.00	5	12.5	OK
33	6.8	135.8	40.81	1.00	5.2	12.5	OK
34	6.2	123.8	37.67	1.00	5.4	12.5	OK
35	5.6	111.8	34.53	1.00	5.7	12.5	OK
36	5.0	99.8	31.38	1.00	5.9	12.5	OK
37	4.4	87.8	28.24	1.00	6.2	12.5	OK
38	3.8	75.8	25.10	1.00	6.4	12.5	OK
39	3.2	63.8	21.95	1.00	6.6	12.5	OK
40	2.6	51.8	18.81	1.00	6.9	12.5	OK
41	2.0	39.8	15.66	1.00	7.1	12.5	OK
42	1.4	27.8	12.52	1.00	7.4	12.5	OK
43	0.8	15.8	17.04	1.78	7.6	12.5	OK

DESIGN OF MECHNICALLY STABILISED EARTH (MSE) WALLS - Load Case 2

Earthquake Parameters			
Peak ground acceleration coefficient, a	0.08		
Horizontal coefficient of acceleration, a_h	0.04		
Hor. Coeffi. of acceleration with amplification of motion, a_{mh}	0.056		
Weight of active zone, W_a	1548		
Load and Resistance factors			
Traffic surcharge, q_L	20		
Live load factor , γ_{EQ}	1.00		
Maximum vertcial earth pressure factor γ_{EVmax}	1.35		
Load Case	2		
Resistance Factors for tensile and pullout resistance, ϕ_{GG}	1.2		
Surcharge equivalent height, h_{eq} (m)	0.74		

Internal Stability with Respect to Tensile Failure of Reinforcement - Load Case 2									
Layer #	Z (m)	σ_H (kPa)	S_v (m)	T_{max} (kN/m)	T_{md} (kN/m)	T_{ult} (kN/m)	S_{rs} (kN/m)	S_{rt} (kN/m)	Check
1	19.7	162.17	0.15	26.76	2.08	200	37.34	2.00	OK
2	19.4	159.79	0.3	52.73	2.08	200	73.59	2.00	OK
3	19.1	157.41	0.3	51.95	2.08	200	72.50	2.00	OK
4	18.8	155.03	0.3	51.16	2.08	200	71.40	2.00	OK
5	18.5	152.65	0.3	50.37	2.08	200	70.30	2.00	OK
6	18.2	150.27	0.3	49.59	2.08	200	69.21	2.00	OK
7	17.9	147.88	0.3	48.80	2.08	200	68.11	2.00	OK
8	17.6	145.50	0.3	48.02	2.08	200	67.01	2.00	OK
9	17.3	143.12	0.3	47.23	2.08	200	65.92	2.00	OK
10	17.0	140.74	0.3	46.44	2.08	200	64.82	2.00	OK
11	16.7	138.36	0.3	45.66	2.08	200	63.72	2.00	OK
12	16.4	135.98	0.3	44.87	2.08	200	62.63	2.00	OK
13	16.1	133.60	0.3	44.09	2.08	200	61.53	2.00	OK
14	15.8	131.22	0.3	43.30	2.08	200	60.43	2.00	OK
15	15.5	128.83	0.3	42.52	2.08	200	59.34	2.00	OK
16	15.2	126.45	0.3	41.73	2.08	200	58.24	2.00	OK
17	14.9	124.07	0.3	40.94	2.08	200	57.14	2.00	OK
18	14.6	121.69	0.3	40.16	2.08	200	56.04	2.00	OK
19	14.3	119.31	0.3	39.37	2.08	200	54.95	2.00	OK
20	14.0	116.93	0.3	38.59	2.08	200	53.85	2.00	OK
21	13.7	114.55	0.3	37.80	2.08	200	52.75	2.00	OK
22	13.4	112.16	0.45	55.52	2.08	200	77.49	2.00	OK
23	12.8	107.40	0.6	70.88	2.08	200	98.93	2.00	OK
24	12.2	102.64	0.6	67.74	2.08	200	94.54	2.00	OK
25	11.6	97.88	0.6	64.60	2.08	200	90.15	2.00	OK
26	11.0	93.11	0.6	61.45	2.08	200	85.77	2.00	OK
27	10.4	88.35	0.6	58.31	2.08	200	81.38	2.00	OK
28	9.8	83.59	0.6	55.17	2.08	200	76.99	2.00	OK
29	9.2	78.82	0.6	52.02	2.08	200	72.61	2.00	OK
30	8.6	74.06	0.6	48.88	2.08	200	68.22	2.00	OK
31	8.0	69.30	0.6	45.74	2.08	200	63.83	2.00	OK
32	7.4	64.54	0.6	42.59	2.08	200	59.44	2.00	OK
33	6.8	59.77	0.6	39.45	2.08	120	55.06	2.00	OK
34	6.2	55.01	0.6	36.31	2.08	120	50.67	2.00	OK
35	5.6	50.25	0.6	33.16	2.08	120	46.28	2.00	OK
36	5.0	45.48	0.6	30.02	2.08	120	41.90	2.00	OK
37	4.4	40.72	0.6	26.88	2.08	120	37.51	2.00	OK
38	3.8	35.96	0.6	23.73	2.08	120	33.12	2.00	OK
39	3.2	31.20	0.6	20.59	2.08	120	28.74	2.00	OK
40	2.6	26.43	0.6	17.45	2.08	120	24.35	2.00	OK
41	2.0	21.67	0.6	14.30	2.08	120	19.96	2.00	OK
42	1.4	16.91	0.6	11.16	2.08	120	15.57	2.00	OK
43	0.8	12.15	1.09	14.56	2.08	120	20.32	2.00	OK

Internal Stability with Respect to Pullout Failure of Reinforcement - Load Case 2

Layer #	Z (m)	σ'_v (kPa)	T_{total} (kN/m)	Required L_e (m)	L (m)	Available L_e (m)	Check
1	19.7	393.8	28.84	1.00	12.5	12.5	OK
2	19.4	387.8	54.81	1.00	12.5	12.4	OK
3	19.1	381.8	54.02	1.00	12.5	12.3	OK
4	18.8	375.8	53.24	1.00	12.5	12.1	OK
5	18.5	369.8	52.45	1.00	12.5	12	OK
6	18.2	363.8	51.67	1.00	12.5	11.9	OK
7	17.9	357.8	50.88	1.00	12.5	11.8	OK
8	17.6	351.8	50.09	1.00	12.5	11.7	OK
9	17.3	345.8	49.31	1.00	12.5	11.5	OK
10	17.0	339.8	48.52	1.00	12.5	11.4	OK
11	16.7	333.8	47.74	1.00	12.5	11.3	OK
12	16.4	327.8	46.95	1.00	12.5	11.2	OK
13	16.1	321.8	46.17	1.00	12.5	11.1	OK
14	15.8	315.8	45.38	1.00	12.5	10.9	OK
15	15.5	309.8	44.59	1.00	12.5	10.8	OK
16	15.2	303.8	43.81	1.00	12.5	10.7	OK
17	14.9	297.8	43.02	1.00	12.5	10.6	OK
18	14.6	291.8	42.24	1.00	12.5	10.5	OK
19	14.3	285.8	41.45	1.00	12.5	10.3	OK
20	14.0	279.8	40.66	1.00	12.5	10.2	OK
21	13.7	273.8	39.88	1.00	12.5	10.1	OK
22	13.4	267.8	57.60	1.00	12.5	10	OK
23	12.8	255.8	72.96	1.00	12.5	9.7	OK
24	12.2	243.8	69.82	1.00	12.5	9.5	OK
25	11.6	231.8	66.68	1.00	12.5	9.3	OK
26	11.0	219.8	63.53	1.00	12.5	9	OK
27	10.4	207.8	60.39	1.00	12.5	8.8	OK
28	9.8	195.8	57.25	1.00	12.5	8.5	OK
29	9.2	183.8	54.10	1.00	12.5	8.3	OK
30	8.6	171.8	50.96	1.00	12.5	8.1	OK
31	8.0	159.8	47.82	1.00	12.5	7.8	OK
32	7.4	147.8	44.67	1.00	12.5	7.6	OK
33	6.8	135.8	41.53	1.00	12.5	7.3	OK
34	6.2	123.8	38.39	1.00	12.5	7.1	OK
35	5.6	111.8	35.24	1.00	12.5	6.9	OK
36	5.0	99.8	32.10	1.00	12.5	6.6	OK
37	4.4	87.8	29.0	1.0	12.5	6.4	OK
38	3.8	75.8	25.8	1.0	12.5	6.2	OK
39	3.2	63.8	22.7	1.0	12.5	5.9	OK
40	2.6	51.8	19.5	1.0	12.5	5.7	OK
41	2.0	39.8	16.4	1.0	12.5	5.4	OK
42	1.4	27.8	13.2	1.0	12.5	5.2	OK
43	0.8	15.8	16.6	1.6	12.5	5.0	OK

DESIGN OF MECHNICALLY STABILISED EARTH (MSE) WALLS - Load Case 3

Wall Geometry			
Wall Height (m), H	19.69		
Bottom width of wall, L	18.36333333		
Top width of wall, w	12.5		
Slope of backfill behind wall (Deg), β	14		
Face inclination from horizontal (Deg), θ_I	108.4		
Reinforced Soil Block Parameters			
Eff. Frcition of reinforced soil (Deg), ϕ'_r	32		
Unit weight of reinfoced soil (kN/m^3), γ_r	20		
Load and Resistance factors			
Traffic surcharge, q_L	10		
Live load factor , γ_{LS}	1.75		
Maximum vertcial earth pressure factor γ_{EVmax}	1.35		
Load Case	3		
Geogrid paramaters			
	Top	**Middle**	**Bottom**
Ultimate strength of reinforcement, $_{Tult}$	GG120	GG200	GG200
Vertical spacing	0.6	0.6	0.3
Length	12.5	12.5	12.5
No of reinforcment layer	11	10	22
Partial factor - creep rupture - RF_{CR}	1.00	1.00	1.00
Partial factor - construction damage - RF_{ID}	1.1	1.1	1.1
Partial factor - environmental effects - RF_D	1.05	1.05	1.05
Resistance Factors for tensile and pullout resistance, ϕ_{GG}	0.9		
Pullout resistance factor, F*	0.42		
Scale correction factor, α	0.8		
Coverage ratio, R_c	1		
C	2		
Batter angle, θ (Deg)	92.0		
Angle of fric between retained backfill and rein. Soil (Deg), δ	14		
Rankine active earth pressure coefficient, K_a	0.294		
lateral earth pressure coefficient, K_r	0.294		
Surcharge equivalent height, h_{eq} (m)	0.65		
Inclination of failure surface with horizontal, ψ (Deg)	53.8		
Note:			
1. For extensible reinforcement (geogrid), lateral stress ratio is equla to 1.0			

Internal Stability with Respect to Tensile Failure of Reinforcement - Load Case 3									
Layer #	Z (m)	σ_H (kPa)	S_v (m)	T_{max} (kN/m)	Rein. Type	T_{ult} (kN/m)	T_{al} (kN/m)	T_r (kN/m)	Check
1	19.7	161.46	0.15	26.64	GG200	200	173.16	155.84	OK
2	19.4	159.08	0.3	52.50	GG200	200	173.16	155.84	OK
3	19.1	156.70	0.3	51.71	GG200	200	173.16	155.84	OK
4	18.8	154.31	0.3	50.92	GG200	200	173.16	155.84	OK
5	18.5	151.93	0.3	50.14	GG200	200	173.16	155.84	OK
6	18.2	149.55	0.3	49.35	GG200	200	173.16	155.84	OK
7	17.9	147.17	0.3	48.57	GG200	200	173.16	155.84	OK
8	17.6	144.79	0.3	47.78	GG200	200	173.16	155.84	OK
9	17.3	142.41	0.3	46.99	GG200	200	173.16	155.84	OK
10	17.0	140.03	0.3	46.21	GG200	200	173.16	155.84	OK
11	16.7	137.64	0.3	45.42	GG200	200	173.16	155.84	OK
12	16.4	135.26	0.3	44.64	GG200	200	173.16	155.84	OK
13	16.1	132.88	0.3	43.85	GG200	200	173.16	155.84	OK
14	15.8	130.50	0.3	43.07	GG200	200	173.16	155.84	OK
15	15.5	128.12	0.3	42.28	GG200	200	173.16	155.84	OK
16	15.2	125.74	0.3	41.49	GG200	200	173.16	155.84	OK
17	14.9	123.36	0.3	40.71	GG200	200	173.16	155.84	OK
18	14.6	120.98	0.3	39.92	GG200	200	173.16	155.84	OK
19	14.3	118.59	0.3	39.14	GG200	200	173.16	155.84	OK
20	14.0	116.21	0.3	38.35	GG200	200	173.16	155.84	OK
21	13.7	113.83	0.3	37.56	GG200	200	173.16	155.84	OK
22	13.4	111.45	0.45	55.17	GG200	200	173.16	155.84	OK
23	12.8	106.69	0.6	70.41	GG200	200	173.16	155.84	OK
24	12.2	101.92	0.6	67.27	GG200	200	173.16	155.84	OK
25	11.6	97.16	0.6	64.13	GG200	200	173.16	155.84	OK
26	11.0	92.40	0.6	60.98	GG200	200	173.16	155.84	OK
27	10.4	87.64	0.6	57.84	GG200	200	173.16	155.84	OK
28	9.8	82.87	0.6	54.70	GG200	200	173.16	155.84	OK
29	9.2	78.11	0.6	51.55	GG200	200	173.16	155.84	OK
30	8.6	73.35	0.6	48.41	GG200	200	173.16	155.84	OK
31	8.0	68.58	0.6	45.27	GG200	200	173.16	155.84	OK
32	7.4	63.82	0.6	42.12	GG200	200	173.16	155.84	OK
33	6.8	59.06	0.6	38.98	GG120	120	103.90	93.51	OK
34	6.2	54.30	0.6	35.84	GG120	120	103.90	93.51	OK
35	5.6	49.53	0.6	32.69	GG120	120	103.90	93.51	OK
36	5.0	44.77	0.6	29.55	GG120	120	103.90	93.51	OK
37	4.4	40.01	0.6	26.40	GG120	120	103.90	93.51	OK
38	3.8	35.24	0.6	23.26	GG120	120	103.90	93.51	OK
39	3.2	30.48	0.6	20.12	GG120	120	103.90	93.51	OK
40	2.6	25.72	0.6	16.97	GG120	120	103.90	93.51	OK
41	2.0	20.96	0.6	13.83	GG120	120	103.90	93.51	OK
42	1.4	16.19	0.6	10.69	GG120	120	103.90	93.51	OK
43	0.8	11.43	1.09	13.71	GG120	120	103.90	93.51	OK

Internal Stability with Respect to Pullout Failure of Reinforcement - Load Case 3							
Layer #	**Z (m)**	**σ′$_v$ (kPa)**	**T_{max} (kN/m)**	**L_e (m)**	**L_a (m)**	**L (m)**	**Check**
1	19.7	393.8	26.64	1.00	0	12.5	OK
2	19.4	387.8	52.50	1.00	0.2	12.5	OK
3	19.1	381.8	51.71	1.00	0.3	12.5	OK
4	18.8	375.8	50.92	1.00	0.4	12.5	OK
5	18.5	369.8	50.14	1.00	0.5	12.5	OK
6	18.2	363.8	49.35	1.00	0.6	12.5	OK
7	17.9	357.8	48.57	1.00	0.8	12.5	OK
8	17.6	351.8	47.78	1.00	0.9	12.5	OK
9	17.3	345.8	46.99	1.00	1	12.5	OK
10	17.0	339.8	46.21	1.00	1.1	12.5	OK
11	16.7	333.8	45.42	1.00	1.2	12.5	OK
12	16.4	327.8	44.64	1.00	1.4	12.5	OK
13	16.1	321.8	43.85	1.00	1.5	12.5	OK
14	15.8	315.8	43.07	1.00	1.6	12.5	OK
15	15.5	309.8	42.28	1.00	1.7	12.5	OK
16	15.2	303.8	41.49	1.00	1.8	12.5	OK
17	14.9	297.8	40.71	1.00	2	12.5	OK
18	14.6	291.8	39.92	1.00	2.1	12.5	OK
19	14.3	285.8	39.14	1.00	2.2	12.5	OK
20	14.0	279.8	38.35	1.00	2.3	12.5	OK
21	13.7	273.8	37.56	1.00	2.4	12.5	OK
22	13.4	267.8	55.17	1.00	2.6	12.5	OK
23	12.8	255.8	70.41	1.00	2.8	12.5	OK
24	12.2	243.8	67.27	1.00	3	12.5	OK
25	11.6	231.8	64.13	1.00	3.3	12.5	OK
26	11.0	219.8	60.98	1.00	3.5	12.5	OK
27	10.4	207.8	57.84	1.00	3.8	12.5	OK
28	9.8	195.8	54.70	1.00	4	12.5	OK
29	9.2	183.8	51.55	1.00	4.2	12.5	OK
30	8.6	171.8	48.41	1.00	4.5	12.5	OK
31	8.0	159.8	45.27	1.00	4.7	12.5	OK
32	7.4	147.8	42.12	1.00	5	12.5	OK
33	6.8	135.8	38.98	1.00	5.2	12.5	OK
34	6.2	123.8	35.84	1.00	5.4	12.5	OK
35	5.6	111.8	32.69	1.00	5.7	12.5	OK
36	5.0	99.8	29.55	1.00	5.9	12.5	OK
37	4.4	87.8	26.40	1.00	6.2	12.5	OK
38	3.8	75.8	23.26	1.00	6.4	12.5	OK
39	3.2	63.8	20.12	1.00	6.6	12.5	OK
40	2.6	51.8	16.97	1.00	6.9	12.5	OK
41	2.0	39.8	13.83	1.00	7.1	12.5	OK
42	1.4	27.8	10.69	1.00	7.4	12.5	OK
43	0.8	15.8	13.71	1.43	7.6	12.5	OK

DESIGN OF MECHNICALLY STABILISED EARTH (MSE) WALLS - Load Case 4

Load and Resistance factors			
Traffic surcharge, q_L	20		
Live load factor, , γ_{LS}	1.00		
Maximum vertcial earth pressure factor γ_{EVmax}	1.35		
Load Case	4		
Resistance Factors for tensile and pullout resistance, ϕ_{GG}	1		
Surcharge equivalent height, h_{eq} (m)	1		
Reinforcement Rupture			
Factored impact load on 1st layer (kN/m)	33.5		
Factored impact load on 2nd layer (kN/m)	8.8		
Reinforcement Pullout			
Factored impact load on 1st layer (kN/m)	19		
Factored impact load on 2nd layer (kN/m)	8.8		

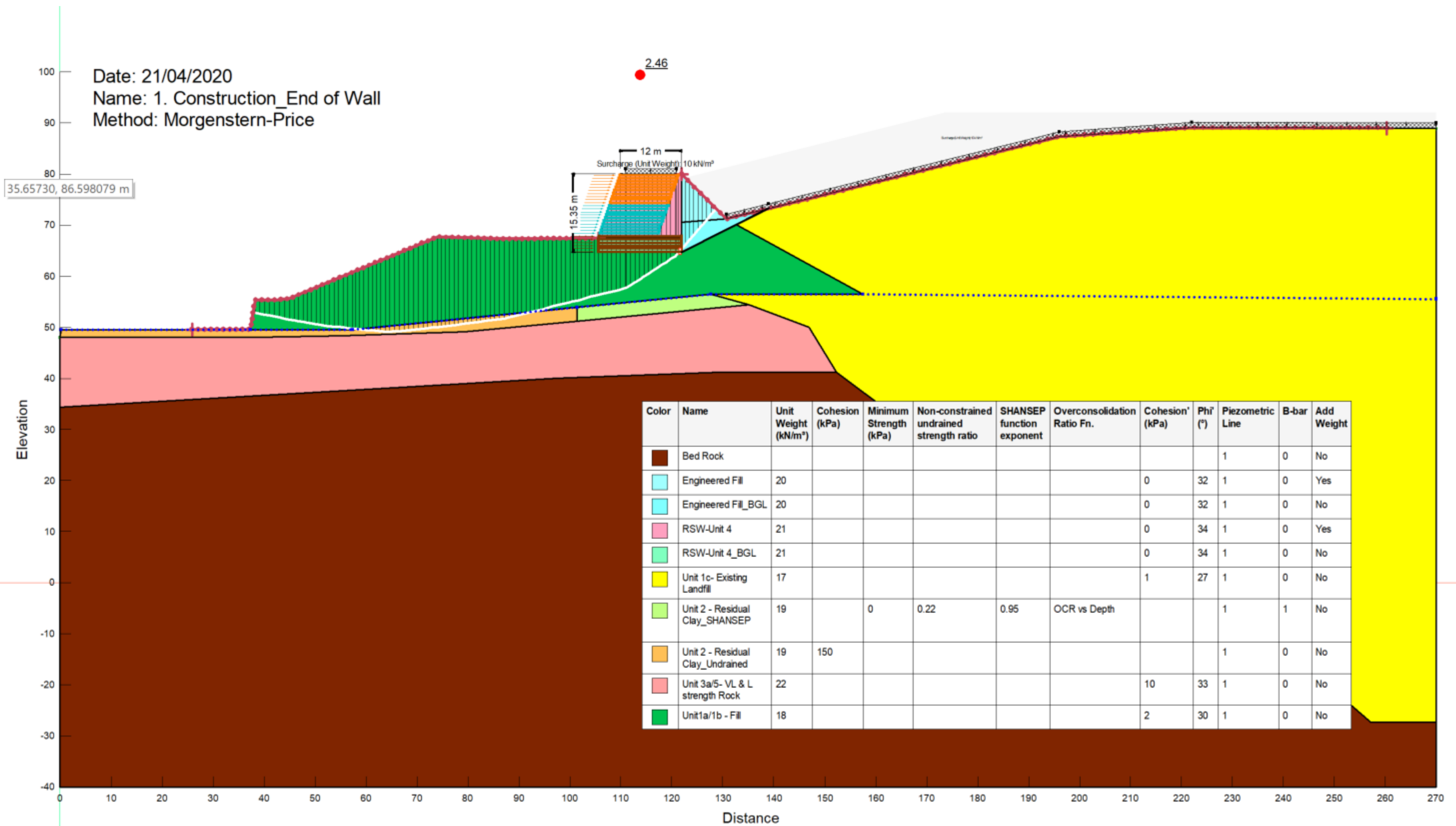
Internal Stability with Respect to Tensile Failure of Reinforcement - Load Case 4									
Layer #	**Z (m)**	**σ_H (kPa)**	**S_v (m)**	**T_{max} (kN/m)**	**T_l(kN/m)**	**T_{ult} (kN/m)**	**S_{rs} (kN/m)**	**S_{rt} (kN/m)**	**Check**
1	19.7	164.24	0.15	27.10	0.00	200	45.38	0.00	OK
2	19.4	161.86	0.3	53.41	0.00	200	89.45	0.00	OK
3	19.1	159.47	0.3	52.63	0.00	200	88.14	0.00	OK
4	18.8	157.09	0.3	51.84	0.00	200	86.82	0.00	OK
5	18.5	154.71	0.3	51.05	0.00	200	85.50	0.00	OK
6	18.2	152.33	0.3	50.27	0.00	200	84.19	0.00	OK
7	17.9	149.95	0.3	49.48	0.00	200	82.87	0.00	OK
8	17.6	147.57	0.3	48.70	0.00	200	81.56	0.00	OK
9	17.3	145.19	0.3	47.91	0.00	200	80.24	0.00	OK
10	17.0	142.80	0.3	47.13	0.00	200	78.92	0.00	OK
11	16.7	140.42	0.3	46.34	0.00	200	77.61	0.00	OK
12	16.4	138.04	0.3	45.55	0.00	200	76.29	0.00	OK
13	16.1	135.66	0.3	44.77	0.00	200	74.98	0.00	OK
14	15.8	133.28	0.3	43.98	0.00	200	73.66	0.00	OK
15	15.5	130.90	0.3	43.20	0.00	200	72.34	0.00	OK
16	15.2	128.52	0.3	42.41	0.00	200	71.03	0.00	OK
17	14.9	126.13	0.3	41.62	0.00	200	69.71	0.00	OK
18	14.6	123.75	0.3	40.84	0.00	200	68.39	0.00	OK
19	14.3	121.37	0.3	40.05	0.00	200	67.08	0.00	OK
20	14.0	118.99	0.3	39.27	0.00	200	65.76	0.00	OK
21	13.7	116.61	0.3	38.48	0.00	200	64.45	0.00	OK
22	13.4	114.23	0.45	56.54	0.00	200	94.70	0.00	OK
23	12.8	109.47	0.6	72.25	0.00	200	121.00	0.00	OK
24	12.2	104.70	0.6	69.10	0.00	200	115.73	0.00	OK
25	11.6	99.94	0.6	65.96	0.00	200	110.47	0.00	OK
26	11.0	95.18	0.6	62.82	0.00	200	105.20	0.00	OK
27	10.4	90.41	0.6	59.67	0.00	200	99.94	0.00	OK
28	9.8	85.65	0.6	56.53	0.00	200	94.67	0.00	OK
29	9.2	80.89	0.6	53.39	0.00	200	89.41	0.00	OK
30	8.6	76.13	0.6	50.24	0.00	200	84.14	0.00	OK
31	8.0	71.36	0.6	47.10	0.00	200	78.88	0.00	OK
32	7.4	66.60	0.6	43.96	0.00	200	73.62	0.00	OK
33	6.8	61.84	0.6	40.81	0.00	120	68.35	0.00	OK
34	6.2	57.07	0.6	37.67	0.00	120	63.09	0.00	OK
35	5.6	52.31	0.6	34.53	0.00	120	57.82	0.00	OK
36	5.0	47.55	0.6	31.38	0.00	120	52.56	0.00	OK
37	4.4	42.79	0.6	28.24	0.00	120	47.29	0.00	OK
38	3.8	38.02	0.6	25.10	0.00	120	42.03	0.00	OK
39	3.2	33.26	0.6	21.95	0.00	120	36.76	0.00	OK
40	2.6	28.50	0.6	18.81	0.00	120	31.50	0.00	OK
41	2.0	23.73	0.6	15.66	8.80	120	26.23	10.16	OK
42	1.4	18.97	0.6	12.52	33.50	120	20.97	38.69	OK
43	0.8	14.21	1.09	17.04	0.00	120	28.53	0.00	OK

Internal Stability with Respect to Pullout Failure of Reinforcement - Load Case 4								
Layer #	Z (m)	σ′$_v$ (kPa)	T_l(kN/m)	T_{total} (kN/m)	Required L_e (m)	L (m)	Available L_e (m)	Check
1	19.7	393.8	0	27.10	1.00	12.5	12.50	OK
2	19.4	387.8	0	53.41	1.00	12.5	12.38	OK
3	19.1	381.8	0	52.63	1.00	12.5	12.26	OK
4	18.8	375.8	0	51.84	1.00	12.5	12.14	OK
5	18.5	369.8	0	51.05	1.00	12.5	12.02	OK
6	18.2	363.8	0	50.27	1.00	12.5	11.90	OK
7	17.9	357.8	0	49.48	1.00	12.5	11.78	OK
8	17.6	351.8	0	48.70	1.00	12.5	11.66	OK
9	17.3	345.8	0	47.91	1.00	12.5	11.54	OK
10	17.0	339.8	0	47.13	1.00	12.5	11.42	OK
11	16.7	333.8	0	46.34	1.00	12.5	11.30	OK
12	16.4	327.8	0	45.55	1.00	12.5	11.18	OK
13	16.1	321.8	0	44.77	1.00	12.5	11.06	OK
14	15.8	315.8	0	43.98	1.00	12.5	10.94	OK
15	15.5	309.8	0	43.20	1.00	12.5	10.82	OK
16	15.2	303.8	0	42.41	1.00	12.5	10.70	OK
17	14.9	297.8	0	41.62	1.00	12.5	10.58	OK
18	14.6	291.8	0	40.84	1.00	12.5	10.46	OK
19	14.3	285.8	0	40.05	1.00	12.5	10.34	OK
20	14.0	279.8	0	39.27	1.00	12.5	10.22	OK
21	13.7	273.8	0	38.48	1.00	12.5	10.10	OK
22	13.4	267.8	0	56.54	1.00	12.5	9.98	OK
23	12.8	255.8	0	72.25	1.00	12.5	9.75	OK
24	12.2	243.8	0	69.10	1.00	12.5	9.51	OK
25	11.6	231.8	0	65.96	1.00	12.5	9.27	OK
26	11.0	219.8	0	62.82	1.00	12.5	9.03	OK
27	10.4	207.8	0	59.67	1.00	12.5	8.79	OK
28	9.8	195.8	0	56.53	1.00	12.5	8.55	OK
29	9.2	183.8	0	53.39	1.00	12.5	8.31	OK
30	8.6	171.8	0	50.24	1.00	12.5	8.07	OK
31	8.0	159.8	0	47.10	1.00	12.5	7.83	OK
32	7.4	147.8	0	43.96	1.00	12.5	7.59	OK
33	6.8	135.8	0	40.81	1.00	12.5	7.35	OK
34	6.2	123.8	0	37.67	1.00	12.5	7.11	OK
35	5.6	111.8	0	34.53	1.00	12.5	6.87	OK
36	5.0	99.8	0	31.38	1.00	12.5	6.63	OK
37	4.4	87.8	0	28.24	1.00	12.5	6.39	OK
38	3.8	75.8	0	25.10	1.00	12.5	6.15	OK
39	3.2	63.8	0	21.95	1.00	12.5	5.91	OK
40	2.6	51.8	0	18.81	1.00	12.5	5.67	OK
41	2.0	39.8	8.8	24.46	1.14	12.5	5.43	OK
42	1.4	27.8	19	31.52	2.11	12.5	5.19	OK
43	0.8	15.8	0	17.04	2.01	12.5	5.74	OK

Layer #	Z (m)	σ_H (kPa)	S (m)	T_{max} (kN/m)	σ'_v (kPa)	Required L_{loop} (m)	Provided L_{loop} (m)	Check
1	19.69	164.2	0.3	24.6	393.8	0.005	1.5	OK
2	19.39	161.9	0.3	24.3	387.8	0.005	1.5	OK
3	19.09	159.5	0.3	23.9	381.8	0.005	1.5	OK
4	18.79	157.1	0.3	23.6	375.8	0.005	1.5	OK
5	18.49	154.7	0.3	23.2	369.8	0.005	1.5	OK
6	18.19	152.3	0.3	22.8	363.8	0.005	1.5	OK
7	17.89	149.9	0.3	22.5	357.8	0.005	1.5	OK
8	17.59	147.6	0.3	22.1	351.8	0.005	1.5	OK
9	17.29	145.2	0.3	21.8	345.8	0.005	1.5	OK
10	16.99	142.8	0.3	21.4	339.8	0.005	1.5	OK
11	16.69	140.4	0.3	21.1	333.8	0.005	1.5	OK
12	16.39	138.0	0.3	20.7	327.8	0.005	1.5	OK
13	16.09	135.7	0.3	20.3	321.8	0.005	1.5	OK
14	15.79	133.3	0.3	20.0	315.8	0.005	1.5	OK
15	15.49	130.9	0.3	19.6	309.8	0.005	1.5	OK
16	15.19	128.5	0.3	19.3	303.8	0.005	1.5	OK
17	14.89	126.1	0.3	18.9	297.8	0.005	1.5	OK
18	14.59	123.8	0.3	18.6	291.8	0.005	1.5	OK
19	14.29	121.4	0.3	18.2	285.8	0.005	1.5	OK
20	13.99	119.0	0.3	17.8	279.8	0.005	1.5	OK
21	13.69	116.6	0.3	17.5	273.8	0.005	1.5	OK
22	13.39	114.2	0.3	17.1	267.8	0.005	1.5	OK
23	12.79	109.5	0.6	32.8	255.8	0.010	1.5	OK
24	12.19	104.7	0.6	31.4	243.8	0.010	1.5	OK
25	11.59	99.9	0.6	30.0	231.8	0.010	1.5	OK
26	10.99	95.2	0.6	28.6	219.8	0.010	1.5	OK
27	10.39	90.4	0.6	27.1	207.8	0.010	1.5	OK
28	9.79	85.7	0.6	25.7	195.8	0.010	1.5	OK
29	9.19	80.9	0.6	24.3	183.8	0.010	1.5	OK
30	8.59	76.1	0.6	22.8	171.8	0.010	1.5	OK
31	7.99	71.4	0.6	21.4	159.8	0.010	1.5	OK
32	7.39	66.6	0.6	20.0	147.8	0.010	1.5	OK
33	6.79	61.8	0.6	18.6	135.8	0.010	1.5	OK
34	6.19	57.1	0.6	17.1	123.8	0.010	1.5	OK
35	5.59	52.3	0.6	15.7	111.8	0.011	1.5	OK
36	4.99	47.5	0.6	14.3	99.8	0.011	1.5	OK
37	4.39	42.8	0.6	12.8	87.8	0.011	1.5	OK
38	3.79	38.0	0.6	11.4	75.8	0.011	1.5	OK
39	3.19	33.3	0.6	10.0	63.8	0.012	1.5	OK
40	2.59	28.5	0.6	8.5	51.8	0.012	1.5	OK
41	1.99	23.7	0.6	7.1	39.8	0.014	1.5	OK
42	1.39	19.0	0.6	5.7	27.8	0.015	1.5	OK
43	0.79	14.2	0.6	4.3	15.8	0.020	1.5	OK

APPENDIX E

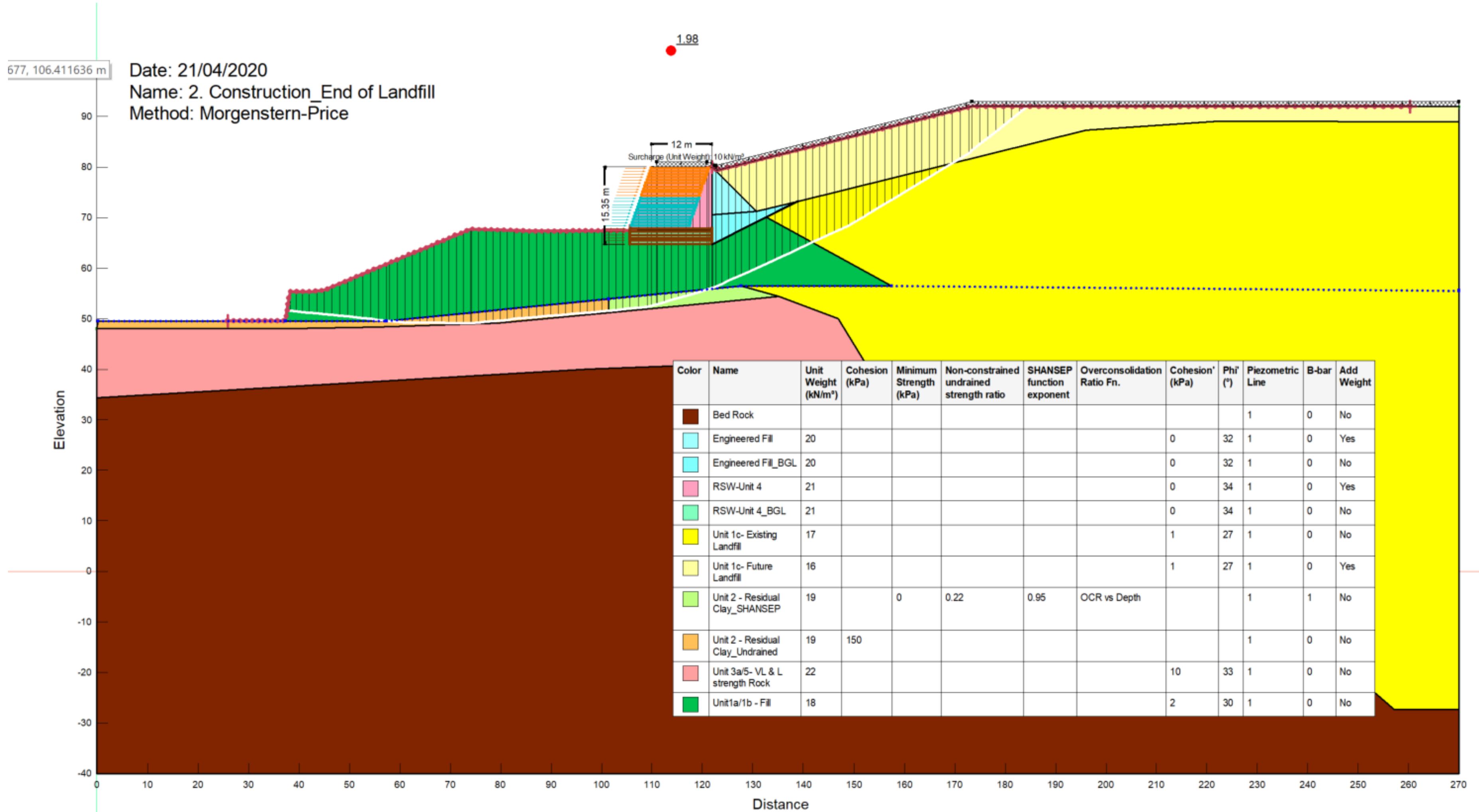
Global Stability



Color	Name	Unit Weight (kN/m³)	Cohesion (kPa)	Minimum Strength (kPa)	Non-constrained undrained strength ratio	SHANSEP function exponent	Overconsolidation Ratio Fn.	Cohesion' (kPa)	Phi' (°)	Piezometric Line	B-bar	Add Weight
	Bed Rock									1	0	No
	Engineered Fill	20						0	32	1	0	Yes
	Engineered Fill_BGL	20						0	32	1	0	No
	RSW-Unit 4	21						0	34	1	0	Yes
	RSW-Unit 4_BGL	21						0	34	1	0	No
	Unit 1c- Existing Landfill	17						1	27	1	0	No
	Unit 2 - Residual Clay_SHANSEP	19		0	0.22	0.95	OCR vs Depth			1	1	No
	Unit 2 - Residual Clay_Undrained	19	150							1	0	No
	Unit 3a/5- VL & L strength Rock	22						10	33	1	0	No
	Unit1a/1b - Fill	18						2	30	1	0	No

GOLDER

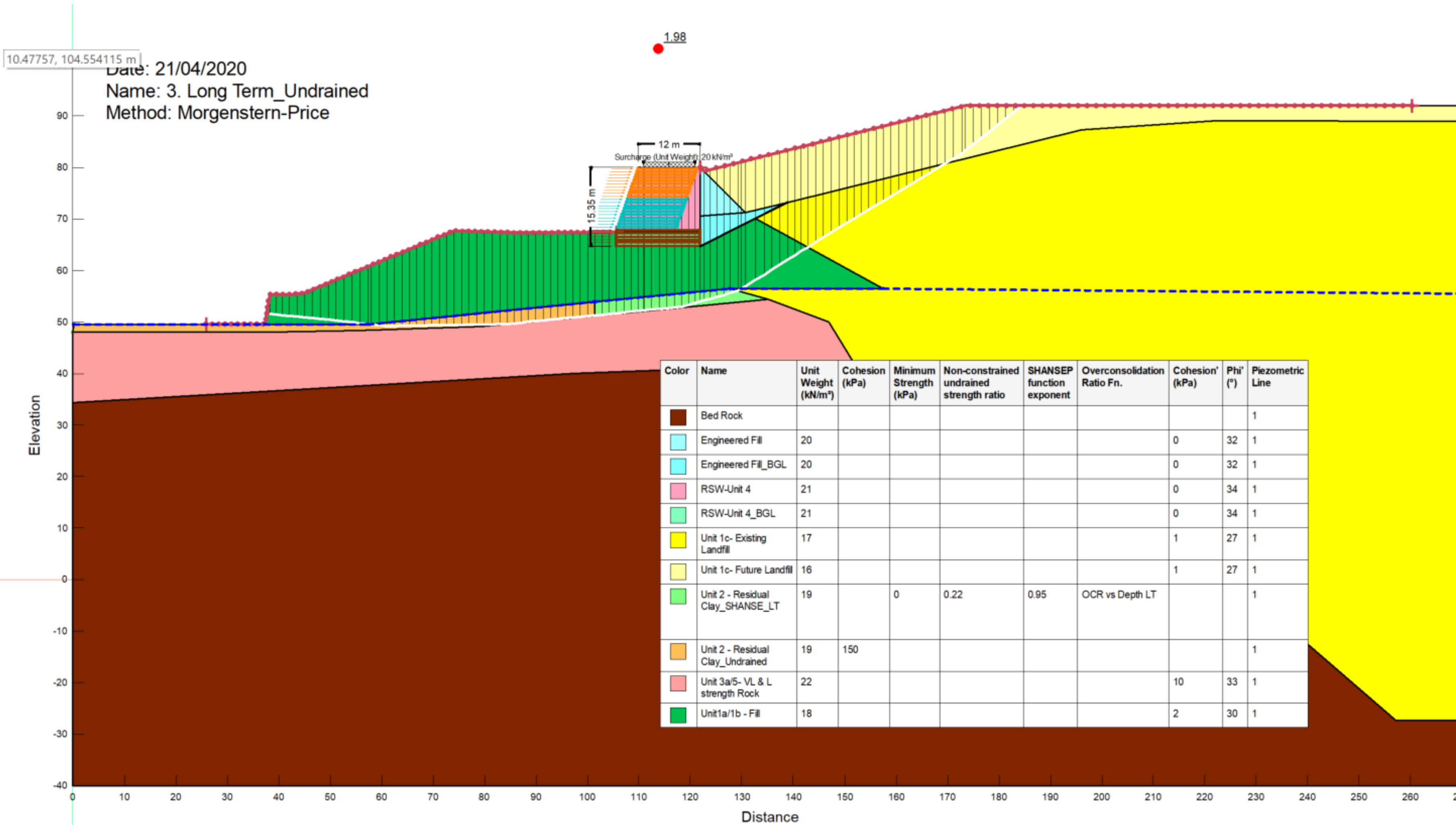
CLIENT Enviroguard Pty Ltd		PROJECT Erskine Park Landfill	A4
DRAWN KT	DATE 22/04/2020	TITLE CS4-Construction_End of Wall	
CHECKED LP	DATE 22/04/2020		
SCALE N.T.S		PROJECT No 19135652	CONTROL 19135652-006-R-RevA	FIFURE E.1



Color	Name	Unit Weight (kN/m³)	Cohesion (kPa)	Minimum Strength (kPa)	Non-constrained undrained strength ratio	SHANSEP function exponent	Overconsolidation Ratio Fn.	Cohesion' (kPa)	Phi' (°)	Piezometric Line	B-bar	Add Weight
	Bed Rock									1	0	No
	Engineered Fill	20						0	32	1	0	Yes
	Engineered Fill_BGL	20						0	32	1	0	No
	RSW-Unit 4	21						0	34	1	0	Yes
	RSW-Unit 4_BGL	21						0	34	1	0	No
	Unit 1c- Existing Landfill	17						1	27	1	0	No
	Unit 1c- Future Landfill	16						1	27	1	0	Yes
	Unit 2 - Residual Clay_SHANSEP	19		0	0.22	0.95	OCR vs Depth			1	1	No
	Unit 2 - Residual Clay_Undrained	19	150							1	0	No
	Unit 3a/5- VL & L strength Rock	22						10	33	1	0	No
	Unit1a/1b - Fill	18						2	30	1	0	No

GOLDER

CLIENT Enviroguard Pty Ltd		PROJECT Erskine Park Landfill		A4
DRAWN KT	DATE 22/04/2020	TITLE CS4-Construction_End of Landfill		
CHECKED LP	DATE 22/04/2020			
SCALE N.T.S		PROJECT No 19135652	CONTROL 19135652-006-R-RevA	FIFURE E.2

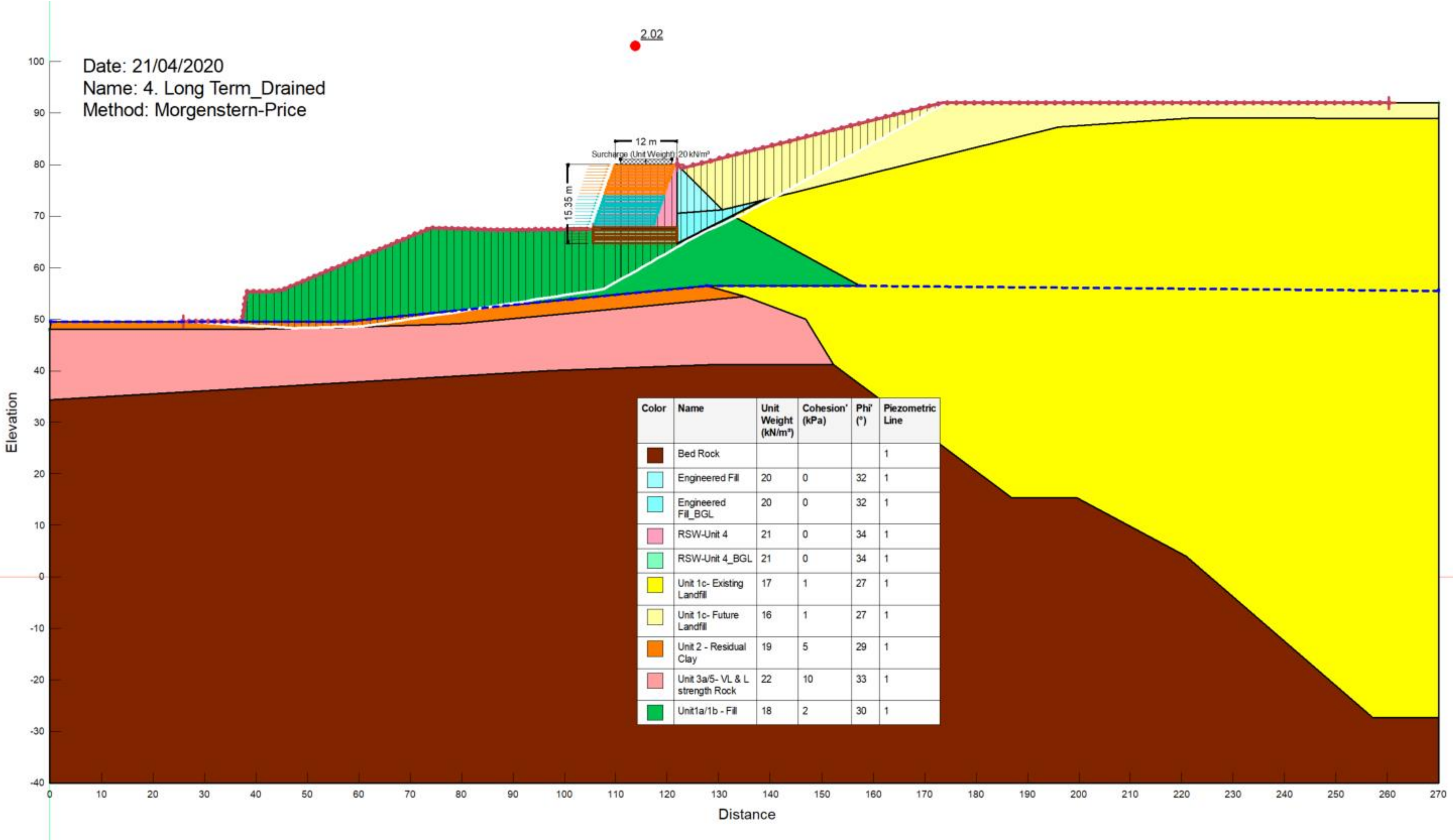


Color	Name	Unit Weight (kN/m³)	Cohesion (kPa)	Minimum Strength (kPa)	Non-constrained undrained strength ratio	SHANSEP function exponent	Overconsolidation Ratio Fn.	Cohesion' (kPa)	Phi' (°)	Piezometric Line
	Bed Rock									1
	Engineered Fill	20						0	32	1
	Engineered Fill_BGL	20						0	32	1
	RSW-Unit 4	21						0	34	1
	RSW-Unit 4_BGL	21						0	34	1
	Unit 1c- Existing Landfill	17						1	27	1
	Unit 1c- Future Landfill	16						1	27	1
	Unit 2 - Residual Clay_SHANSE_LT	19		0	0.22	0.95	OCR vs Depth LT			1
	Unit 2 - Residual Clay_Undrained	19	150							1
	Unit 3a/5- VL & L strength Rock	22						10	33	1
	Unit1a/1b - Fill	18						2	30	1

GOLDER

CLIENT Enviroguard Pty Ltd		PROJECT Erskine Park Landfill	A4
DRAWN KT	DATE 22/04/2020	TITLE CS4-Long Term_Undrained	
CHECKED LP	DATE 22/04/2020		
SCALE N.T.S		PROJECT No 19135652 · CONTROL 19135652-006-R-RevA	FIGURE E.3

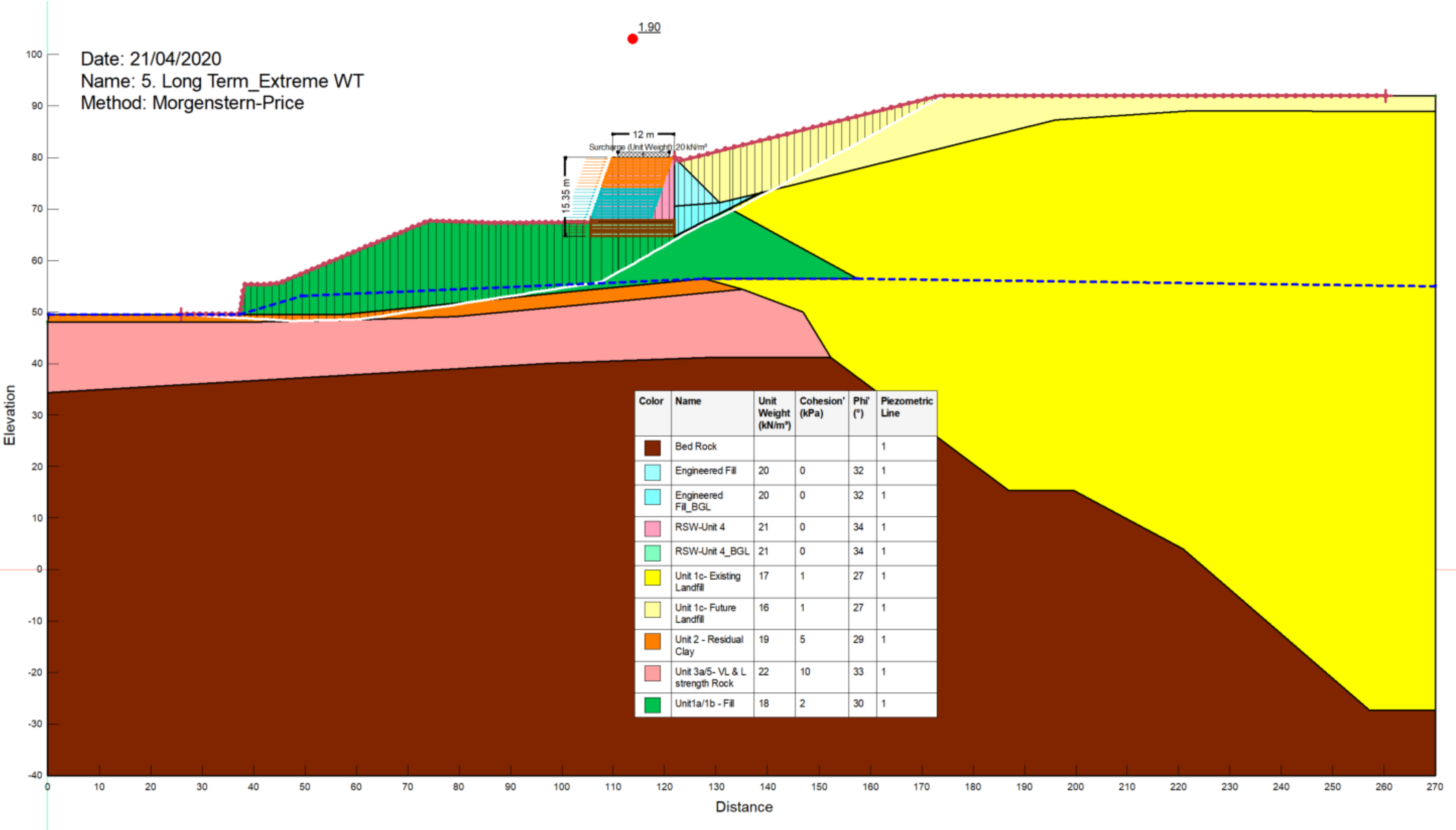
Document Set ID: 9209113
Version: 1, Version Date: 09/07/2020



Color	Name	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Piezometric Line
	Bed Rock				1
	Engineered Fill	20	0	32	1
	Engineered Fill_BGL	20	0	32	1
	RSW-Unit 4	21	0	34	1
	RSW-Unit 4_BGL	21	0	34	1
	Unit 1c- Existing Landfill	17	1	27	1
	Unit 1c- Future Landfill	16	1	27	1
	Unit 2 - Residual Clay	19	5	29	1
	Unit 3a/5- VL & L strength Rock	22	10	33	1
	Unit1a/1b - Fill	18	2	30	1

GOLDER

CLIENT Enviroguard Pty Ltd		PROJECT Erskine Park Landfill			A4
DRAWN KT	DATE 22/04/2020	TITLE CS4-Long Term_Drained			
CHECKED LP	DATE 22/04/2020				
SCALE N.T.S		PROJECT No 19135652	CONTROL 19135652-006-R-RevA	FIGURE E.4	



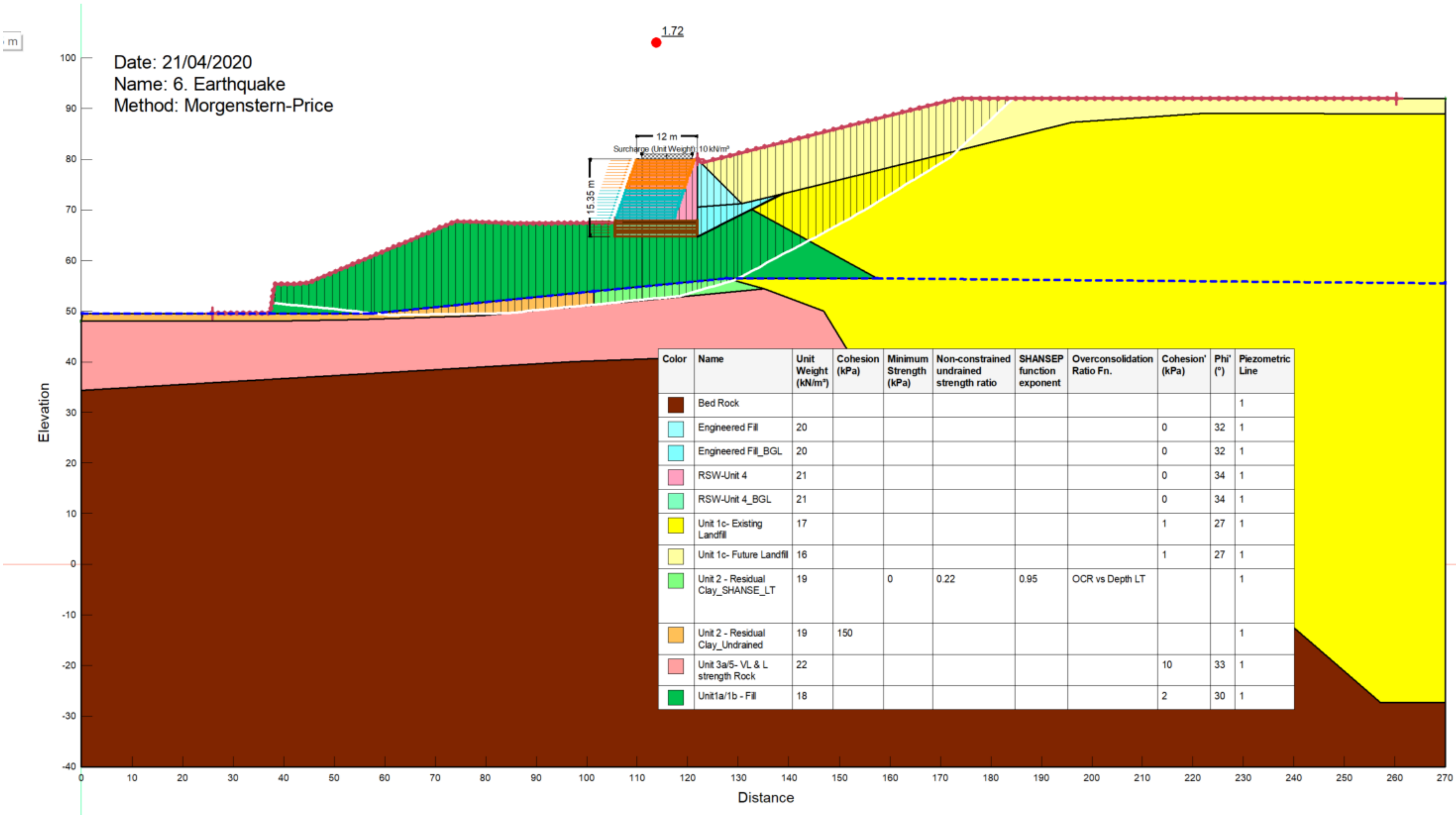
Color	Name	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Piezometric Line
	Bed Rock				1
	Engineered Fill	20	0	32	1
	Engineered Fill_BGL	20	0	32	1
	RSW-Unit 4	21	0	34	1
	RSW-Unit 4_BGL	21	0	34	1
	Unit 1c- Existing Landfill	17	1	27	1
	Unit 1c- Future Landfill	16	1	27	1
	Unit 2 - Residual Clay	19	5	29	1
	Unit 3a/5- VL & L strength Rock	22	10	33	1
	Unit1a/1b - Fill	18	2	30	1

GOLDER

CLIENT	Enviroguard Pty Ltd
PROJECT	Erskine Park Landfill
	A4
DRAWN	KT
DATE	22/04/2020
CHECKED	LP
DATE	22/04/2020
TITLE	CS4-Long Term_Extreme Water Table
SCALE	N.T.S
PROJECT No	19135652
CONTROL	19135652-006-R-RevA
FIFURE	E.5

Document Set ID: 9209113
Version: 1, Version Date: 09/07/2020

Date: 21/04/2020
Name: 6. Earthquake
Method: Morgenstern-Price

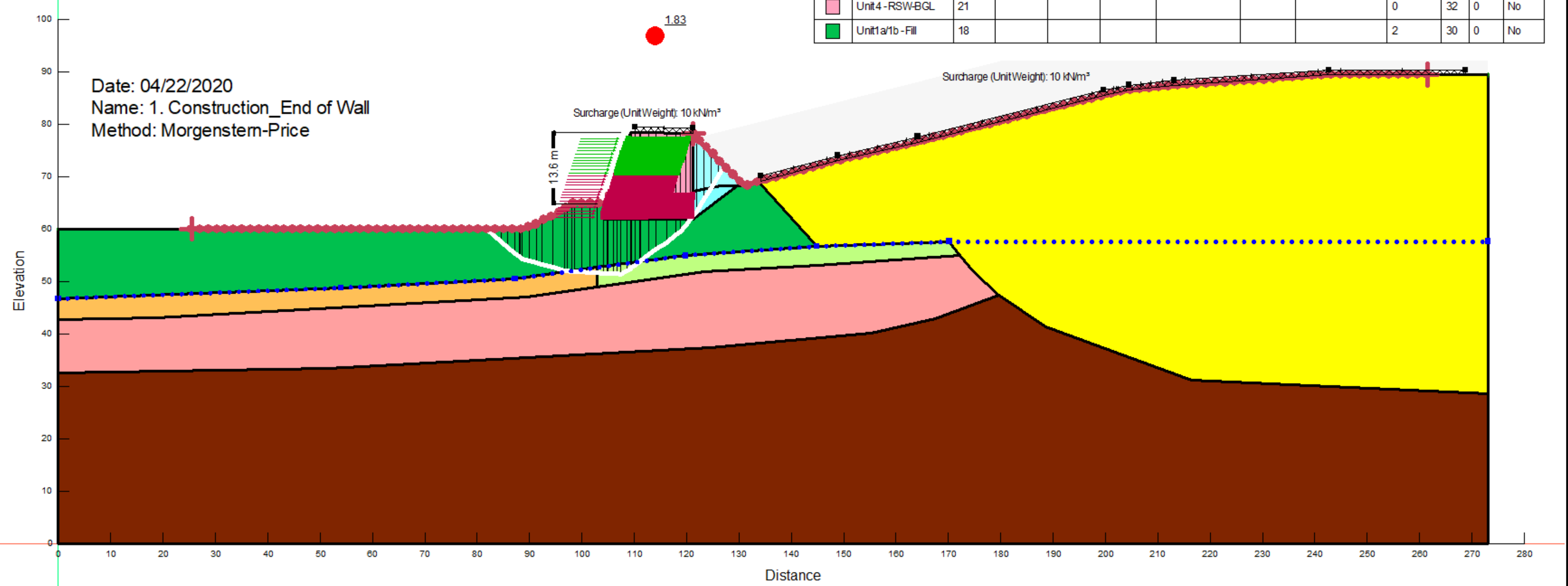


Color	Name	Unit Weight (kN/m³)	Cohesion (kPa)	Minimum Strength (kPa)	Non-constrained undrained strength ratio	SHANSEP function exponent	Overconsolidation Ratio Fn.	Cohesion' (kPa)	Phi' (°)	Piezometric Line
	Bed Rock									1
	Engineered Fill	20						0	32	1
	Engineered Fill_BGL	20						0	32	1
	RSW-Unit 4	21						0	34	1
	RSW-Unit 4_BGL	21						0	34	1
	Unit 1c- Existing Landfill	17						1	27	1
	Unit 1c- Future Landfill	16						1	27	1
	Unit 2 - Residual Clay_SHANSE_LT	19		0	0.22	0.95	OCR vs Depth LT			1
	Unit 2 - Residual Clay_Undrained	19	150							1
	Unit 3a/5- VL & L strength Rock	22						10	33	1
	Unit1a/1b - Fill	18						2	30	1

GOLDER

CLIENT Enviroguard Pty Ltd		PROJECT Erskine Park Landfill	A4
DRAWN KT	DATE 22/04/2020	TITLE CS4-Earthquake	
CHECKED LP	DATE 22/04/2020		
SCALE N.T.S		PROJECT No 19135652 / CONTROL 19135652-006-R-RevA	FIGURE E.6

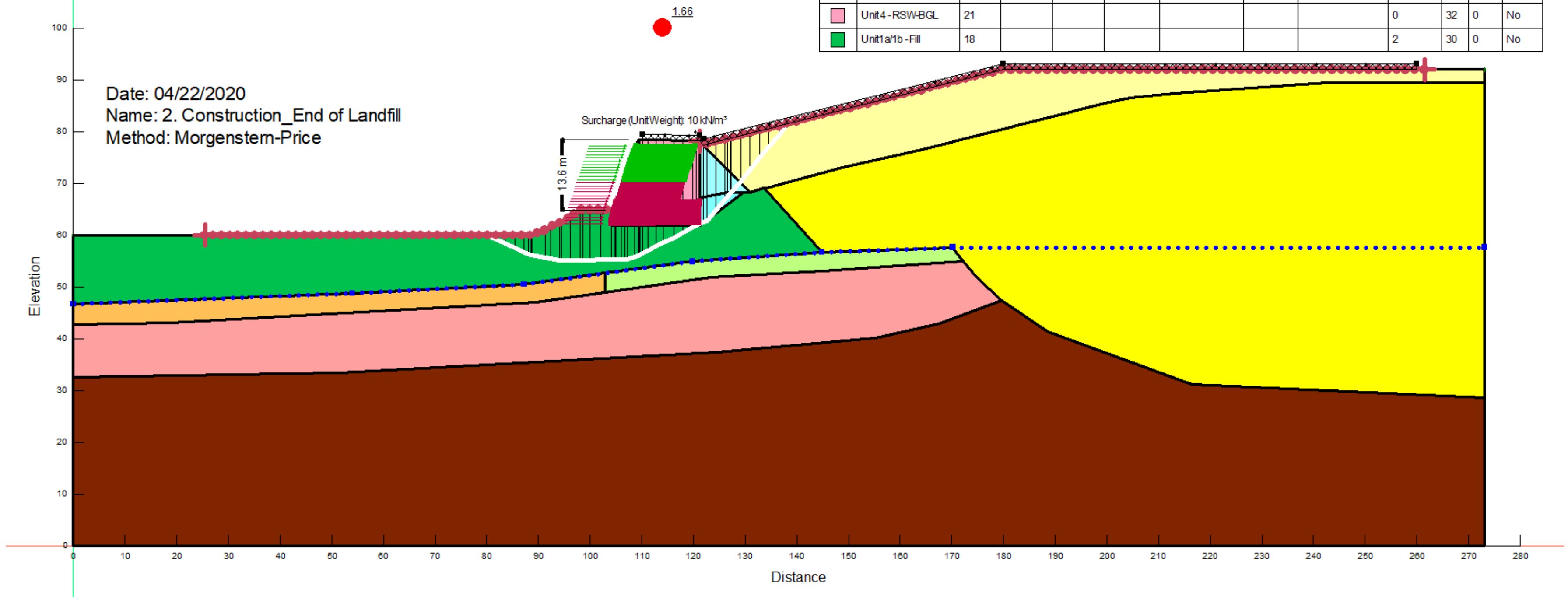
Color	Name	Unit Weight (kN/m³)	Cohesion (kPa)	Minimum Strength (kPa)	Maximum Strength (kPa)	Non-constrained undrained strength ratio	SHANSEP function exponent	Overconsolidation Ratio Fn.	Cohesion' (kPa)	Phi' (°)	B-bar	Add Weight
	BedRock										0	No
	Engineered Fill-BGL	20							0	32	0	No
	Engineered Fill	20							0	32	0	Yes
	Unit 1c-Existing Landfill	17							1	27	0	No
	Unit 2 - Residual Clay_SHANCEP	19		149	151	0.22	0.95	OCR vs Depth			1	No
	Unit 2 - Residual Clay_Undrained	19	150								0	No
	Unit 3a/5- VL & L strength Rock	22							10	33	0	No
	Unit 4 - RSW	21							0	32	0	Yes
	Unit 4 - RSW-BGL	21							0	32	0	No
	Unit1a/1b - Fill	18							2	30	0	No



CLIENT Enviroguard Pty Ltd		PROJECT Erskine Park Landfill	A4
DRAWN KT	DATE 22/04/2020	TITLE CS5-Construction_End of Wall	
CHECKED LP	DATE 22/04/2020		
SCALE N.T.S		PROJECT No 19135652 / CONTROL 19135652-006-R-RevA	FIFURE E.7

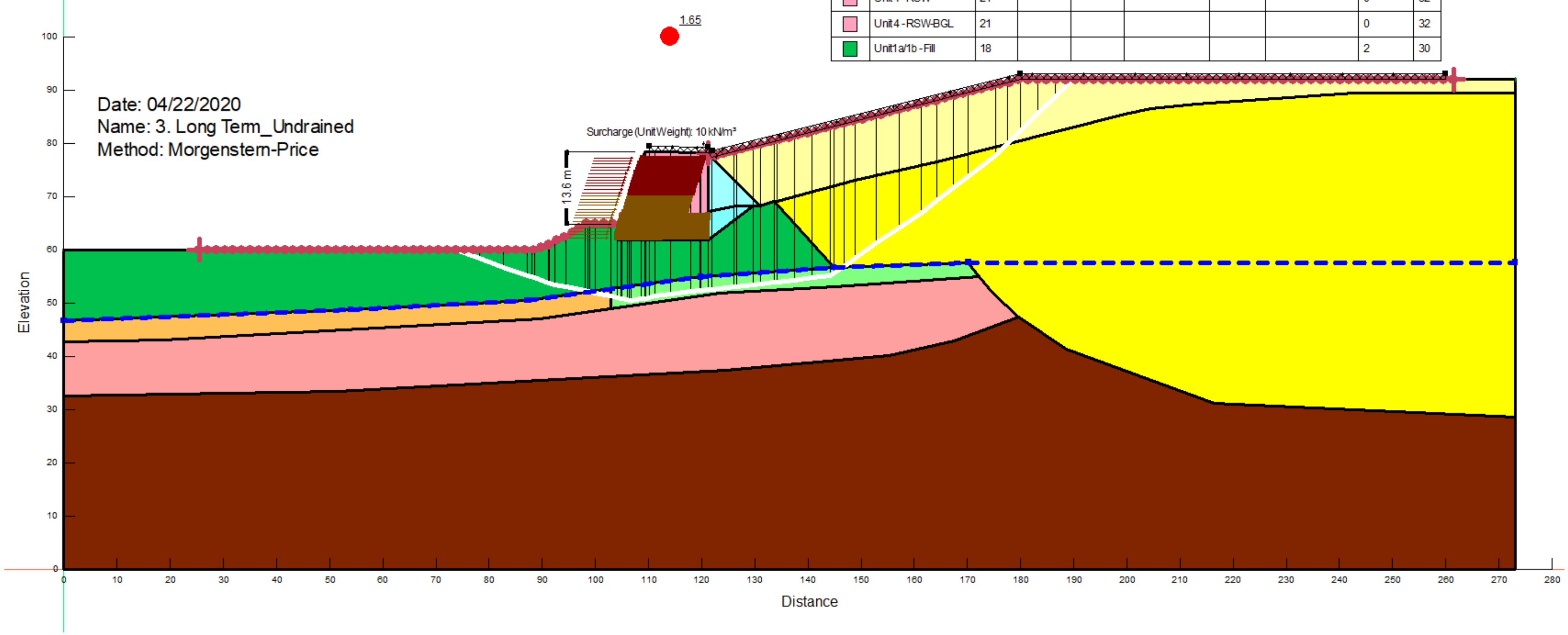
GOLDER

Color	Name	Unit Weight (kN/m³)	Cohesion (kPa)	Minimum Strength (kPa)	Maximum Strength (kPa)	Non-constrained undrained strength ratio	SHANSEP function exponent	Overconsolidation Ratio Fn.	Cohesion' (kPa)	Phi' (°)	B-bar	Add Weight
	Bed Rock										0	No
	Engineered Fill-BGL	20							0	32	0	No
	Engineered Fill	20							0	32	0	Yes
	Unit 1c-Existing Landfill	17							1	27	0	No
	Unit 1c-Future Landfill	16							1	27	0	Yes
	Unit2 - Residual Clay_SHANCEP	19		149	151	0.22	0.95	OCR vs Depth			1	No
	Unit2 - Residual Clay_Undrained	19	150								0	No
	Unit3a/5- VL & L strength Rock	22							10	33	0	No
	Unit4 - RSW	21							0	32	0	Yes
	Unit4 - RSW-BGL	21							0	32	0	No
	Unit1a/1b - Fill	18							2	30	0	No



CLIENT Enviroguard Pty Ltd		PROJECT Erskine Park Landfill	A4
DRAWN KT	DATE 22/04/2020	TITLE CS5-Construction_End of Landfill	
CHECKED LP	DATE 22/04/2020		
SCALE N.T.S		PROJECT No 19135652	CONTROL 19135652-006-R-RevA	FIFURE E.8

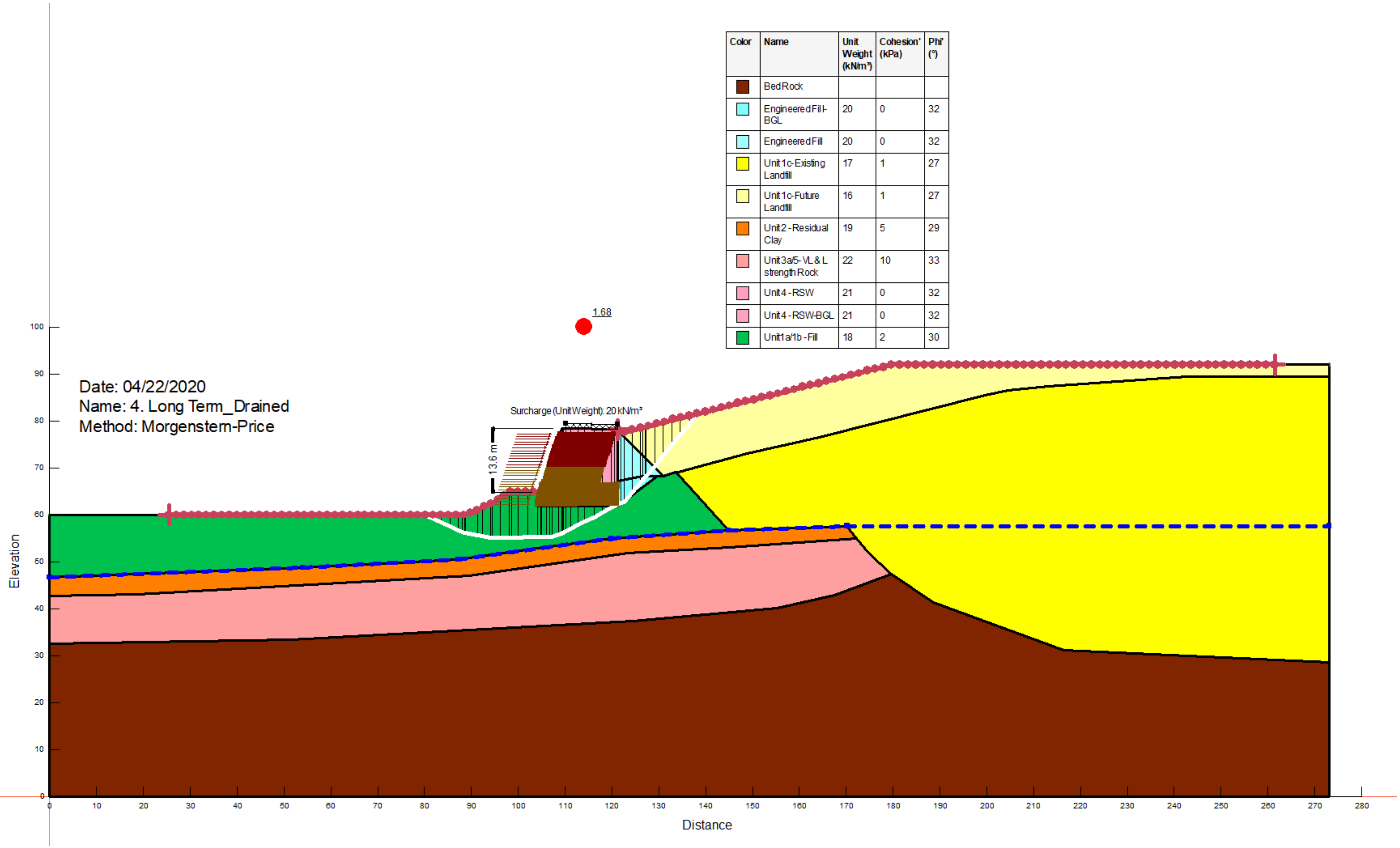
GOLDER



Color	Name	Unit Weight (kN/m³)	Cohesion (kPa)	Minimum Strength (kPa)	Non-constrained undrained strength ratio	SHANSEP function exponent	Overconsolidation Ratio Fn.	Cohesion' (kPa)	Phi' (°)
	Bed Rock								
	Engineered Fill-BGL	20						0	32
	Engineered Fill	20						0	32
	Unit 1c-Existing Landfill	17						1	27
	Unit 1c-Future Landfill	16						1	27
	Unit 2 - Residual Clay_SHANCEP_LT	19		0	0.22	0.95	OCR vs Depth_LT		
	Unit 2 - Residual Clay_Undrained	19	150						
	Unit 3a/5- VL & L strength Rock	22						10	33
	Unit 4 - RSW	21						0	32
	Unit 4 - RSW-BGL	21						0	32
	Unit 1a/1b - Fill	18						2	30

GOLDER

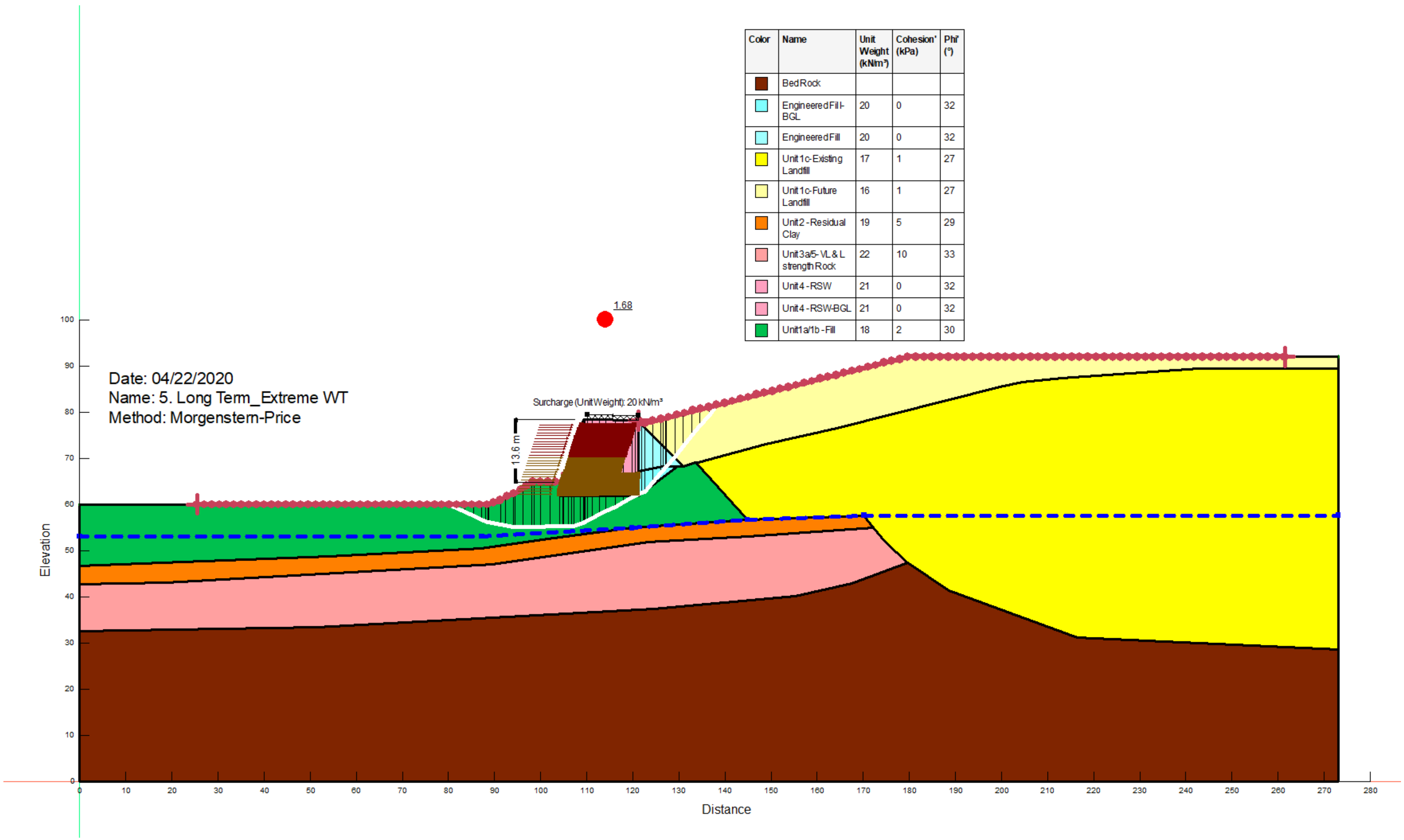
CLIENT Enviroguard Pty Ltd		PROJECT Erskine Park Landfill	A4
DRAWN KT	DATE 22/04/2020	TITLE CS5-Long Term_Undrained	
CHECKED LP	DATE 22/04/2020		
SCALE N.T.S		PROJECT No 19135652 / CONTROL 19135652-006-R-RevA	FIFURE E.9



Color	Name	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)
	Bed Rock			
	Engineered Fill - BGL	20	0	32
	Engineered Fill	20	0	32
	Unit 1c-Existing Landfill	17	1	27
	Unit 1c-Future Landfill	16	1	27
	Unit 2 - Residual Clay	19	5	29
	Unit 3a/5- VL & L strength Rock	22	10	33
	Unit 4 - RSW	21	0	32
	Unit 4 - RSW-BGL	21	0	32
	Unit1a/1b - Fill	18	2	30

GOLDER

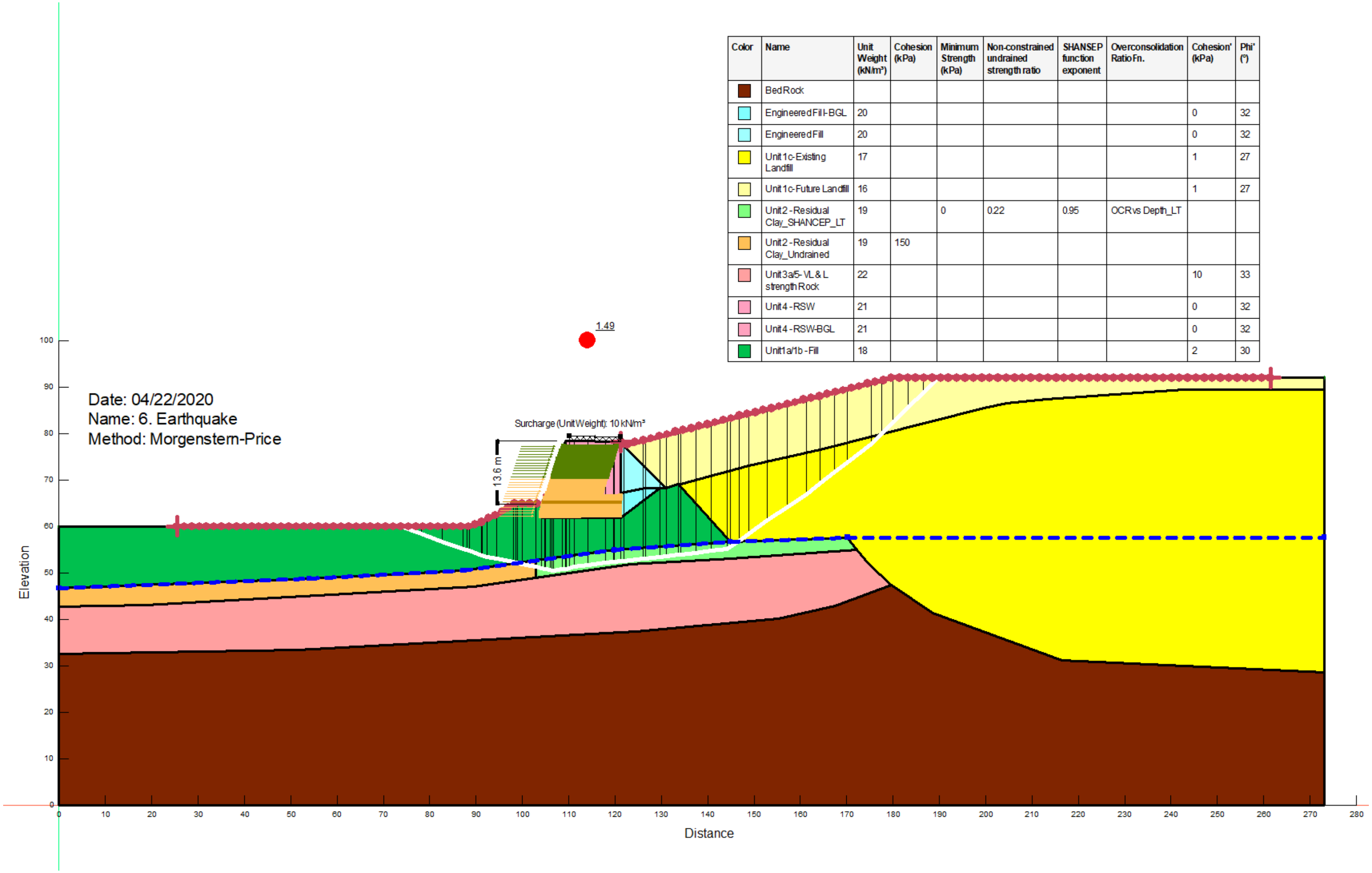
CLIENT Enviroguard Pty Ltd		PROJECT Erskine Park Landfill	A4
DRAWN KT	DATE 22/04/2020	TITLE CS5-Long Term_Drained	
CHECKED LP	DATE 22/04/2020		
SCALE N.T.S		PROJECT No 19135652 / CONTROL 19135652-006-R-RevA	FIFURE E.10



Color	Name	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)
	Bed Rock			
	Engineered Fill - BGL	20	0	32
	Engineered Fill	20	0	32
	Unit 1c-Existing Landfill	17	1	27
	Unit 1c-Future Landfill	16	1	27
	Unit 2 - Residual Clay	19	5	29
	Unit 3a/5 - VL & L strength Rock	22	10	33
	Unit 4 - RSW	21	0	32
	Unit 4 - RSW-BGL	21	0	32
	Unit 1a/1b - Fill	18	2	30

GOLDER

CLIENT Enviroguard Pty Ltd		PROJECT Erskine Park Landfill	A4
DRAWN KT	DATE 22/04/2020	TITLE CS5-Long Term_Extreme Water Table	
CHECKED LP	DATE 22/04/2020		
SCALE N.T.S		PROJECT No 19135652 / CONTROL 19135652-006-R-RevA	FIFURE E.11

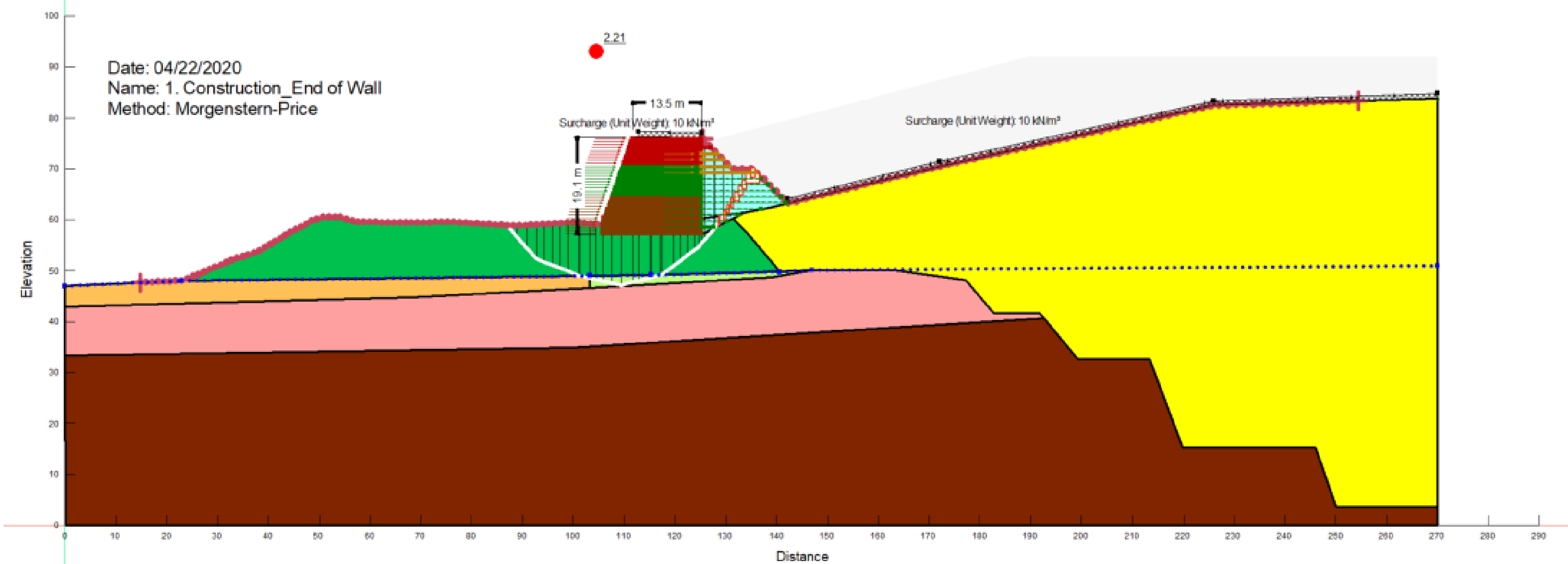


Color	Name	Unit Weight (kN/m³)	Cohesion (kPa)	Minimum Strength (kPa)	Non-constrained undrained strength ratio	SHANSEP function exponent	Overconsolidation Ratio Fn.	Cohesion' (kPa)	Phi' (°)
	BedRock								
	Engineered Fill-BGL	20						0	32
	Engineered Fill	20						0	32
	Unit 1c-Existing Landfill	17						1	27
	Unit 1c-Future Landfill	16						1	27
	Unit 2 - Residual Clay_SHANCEP_LT	19		0	0.22	0.95	OCR vs Depth_LT		
	Unit 2 - Residual Clay_Undrained	19	150						
	Unit 3a/5 - VL & L strength Rock	22						10	33
	Unit 4 - RSW	21						0	32
	Unit 4 - RSW-BGL	21						0	32
	Unit 1a/1b - Fill	18						2	30

GOLDER

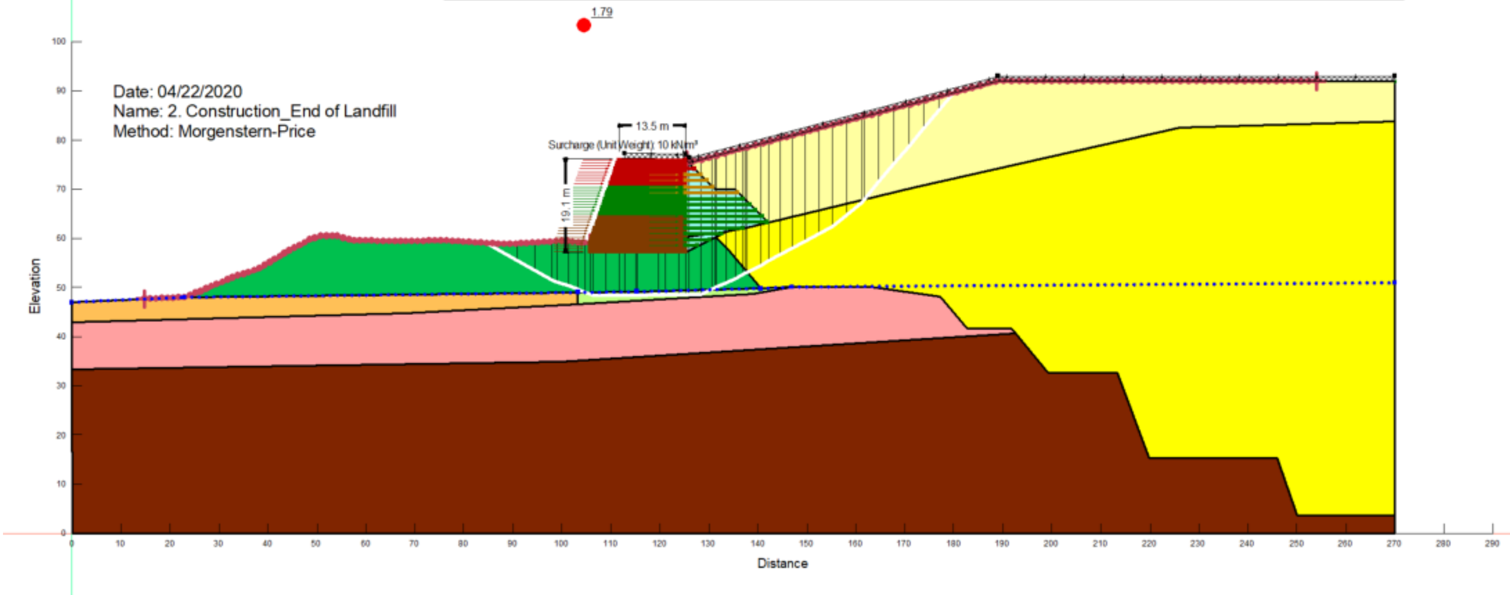
CLIENT Enviroguard Pty Ltd		PROJECT Erskine Park Landfill	A4
DRAWN KT	DATE 22/04/2020	TITLE CS5-Earthquake	
CHECKED LP	DATE 22/04/2020		
SCALE N.T.S		PROJECT No 19135652 / CONTROL 19135652-006-R-RevA	FIFURE E.12

Color	Name	Unit Weight (kN/m³)	Cohesion (kPa)	Minimum Strength (kPa)	Maximum Strength (kPa)	Non-constrained undrained strength ratio	SHANSEP function exponent	Overconsolidation Ratio Fn.	Cohesion' (kPa)	Phi' (°)	B-bar	Add Weight
	Bed Rock										0	No
	Engineered Fill	20							0	32	0	Yes
	Engineered Fill - BGL	20							0	32	0	No
	RSW-Unit 4	21							0	32	0	Yes
	RSW-Unit 4 - BGL	21							0	32	0	No
	Unit 1c- Existing Landfill	17							1	27	0	No
	Unit 2 - Residual Clay _SHANCEP	18		149	150	0.22	0.95	Depth vs OCR			1	No
	Unit 2 - Residual Clay _Undrained	19	150								0	No
	Unit 3a/5- VL & L Strength Rock	22							10	33	0	No
	Unit1a/1b - Fill	18							2	30	0	No



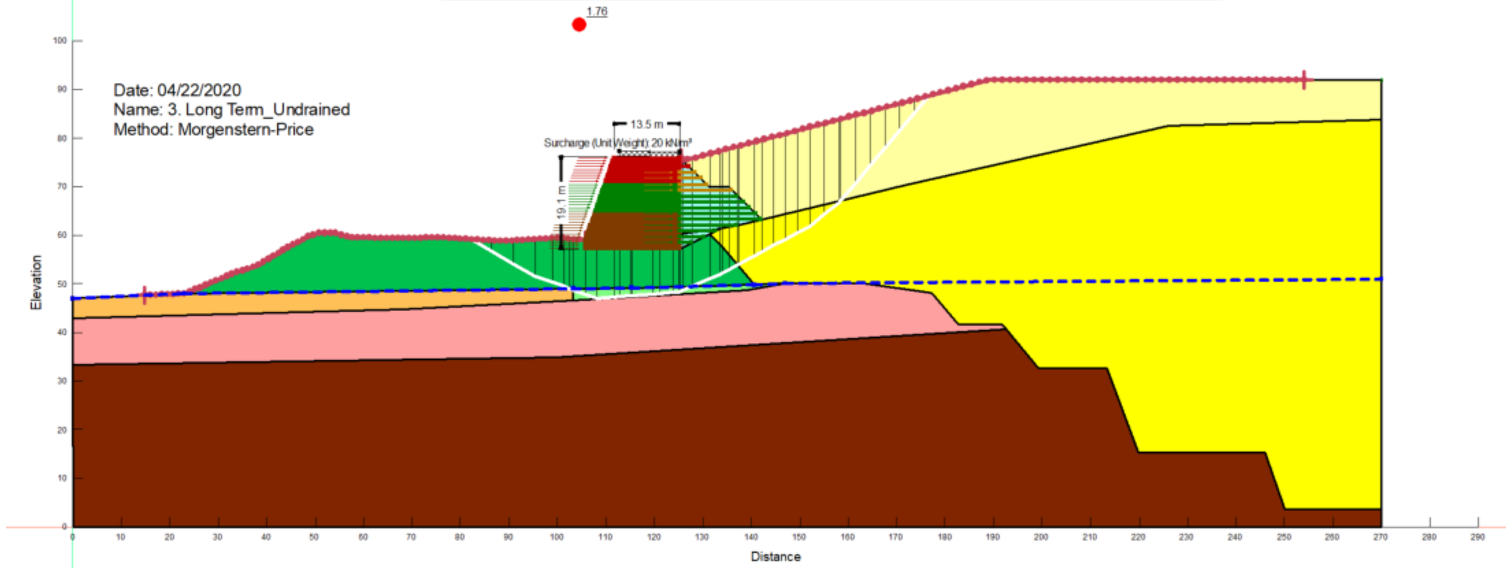
GOLDER	CLIENT Enviroguard Pty Ltd	PROJECT Erskine Park Landfill	A4
	DRAWN KT / DATE 22/04/2020	TITLE CS6-Construction_End of Wall	
	CHECKED LP / DATE 22/04/2020		
	SCALE N.T.S	PROJECT No 19135652 / CONTROL 19135652-006-R-RevA	FIFURE E.13

Color	Name	Unit Weight (kN/m³)	Cohesion (kPa)	Minimum Strength (kPa)	Maximum Strength (kPa)	Non-constrained undrained strength ratio	SHANSEP function exponent	Overconsolidation Ratio Fn.	Cohesion' (kPa)	Phi' (°)	B-bar	Add Weight
	Bed Rock										0	No
	Engineered Fill	20							0	32	0	Yes
	Engineered Fill - BGL	20							0	32	0	No
	RSW-Unit 4	21							0	32	0	Yes
	RSW-Unit 4 - BGL	21							0	32	0	No
	Unit 1c- Existing Landfill	17							1	27	0	No
	Unit 1c- Future Landfill	16							1	27	0	Yes
	Unit 2 - Residual Clay _SHANCEP	18		149	150	0.22	0.95	Depth vs OCR			1	No
	Unit 2 - Residual Clay _Undrained	19	150								0	No
	Unit 3a/5- VL & L Strength Rock	22							10	33	0	No
	Unit1a/1b - Fill	18							2	30	0	No



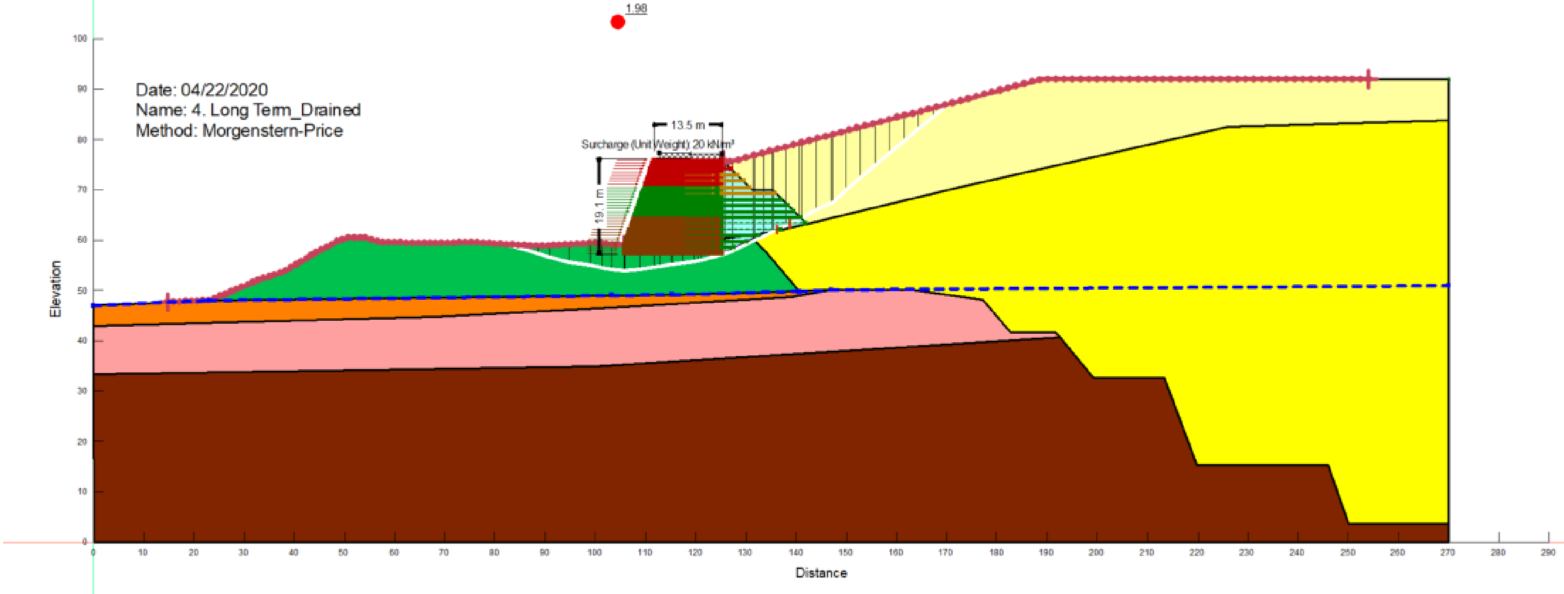
GOLDER	CLIENT Enviroguard Pty Ltd		PROJECT Erskine Park Landfill	A4
	DRAWN KT	DATE 22/04/2020	TITLE CS6-Construction_End of Landfill	
	CHECKED LP	DATE 22/04/2020		
	SCALE N.T.S		PROJECT No 19135652 / CONTROL 19135652-006-R-RevA	FIFURE E.14

Color	Name	Unit Weight (kN/m³)	Cohesion (kPa)	Minimum Strength (kPa)	Non-constrained undrained strength ratio	SHANSEP function exponent	Overconsolidation Ratio Fn.	Cohesion' (kPa)	Phi' (°)
	Bed Rock								
	Engineered Fill	20						0	32
	Engineered Fill - BGL	20						0	32
	RSW-Unit 4	21						0	32
	RSW-Unit 4 - BGL	21						0	32
	Unit 1c- Existing Landfill	17						1	27
	Unit 1c- Future Landfill	16						1	27
	Unit 2 - Residual Clay _SHANCEP_LT	18		0	0.22	0.95	Depth vs OCR- LT		
	Unit 2 - Residual Clay _Undrained	19	150						
	Unit 3a/5- VL & L Strength Rock	22						10	33
	Unit1a/1b - Fill	18						2	30



GOLDER	CLIENT Enviroguard Pty Ltd	PROJECT Erskine Park Landfill	A4
	DRAWN KT / DATE 22/04/2020	TITLE CS6-Long Term_Undrained	
	CHECKED LP / DATE 22/04/2020		
	SCALE N.T.S	PROJECT No 19135652 / CONTROL 19135652-006-R-RevA	FIGURE E.15

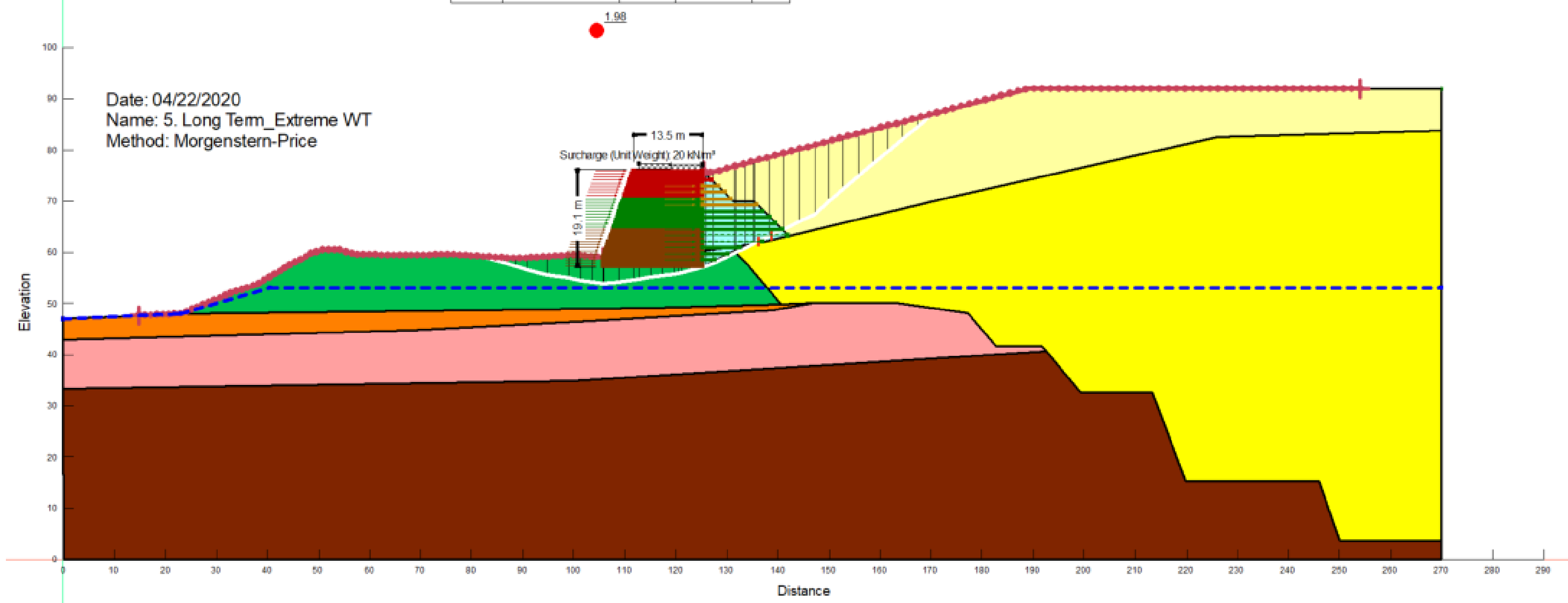
Color	Name	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)
	Bed Rock			
	Engineered Fill	20	0	32
	Engineered Fill - BGL	20	0	32
	RSW-Unit 4	21	0	32
	RSW-Unit 4 - BGL	21	0	32
	Unit 1c- Existing Landfill	17	1	27
	Unit 1c- Future Landfill	16	1	27
	Unit 2 - Residual Clay	19	5	29
	Unit 3a/5- VL & L Strength Rock	22	10	33
	Unit1a/1b - Fill	18	2	30



CLIENT Enviroguard Pty Ltd		PROJECT Erskine Park Landfill	A4
DRAWN KT	DATE 22/04/2020	TITLE CS6-Long Term_Drained	
CHECKED LP	DATE 22/04/2020		
SCALE N.T.S		PROJECT No 19135652 / CONTROL 19135652-006-R-RevA	FIFURE E.16

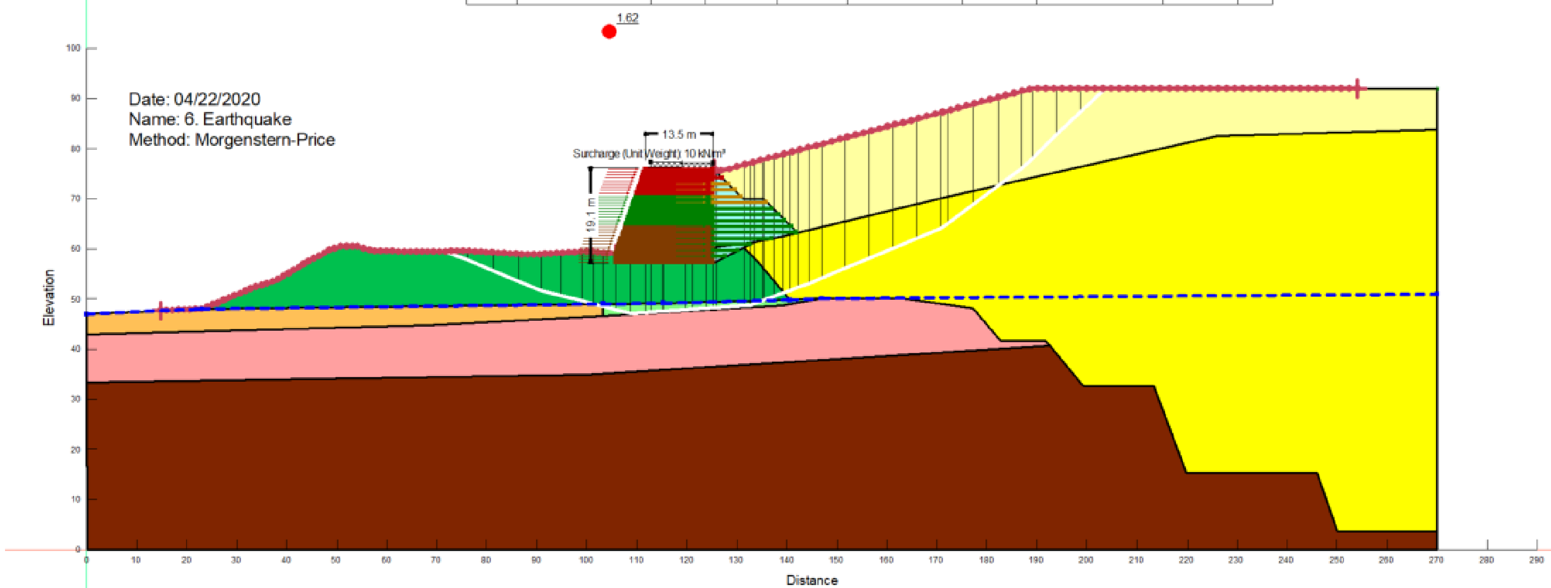
GOLDER

Color	Name	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)
	Bed Rock			
	Engineered Fill	20	0	32
	Engineered Fill - BGL	20	0	32
	RSW-Unit 4	21	0	32
	RSW-Unit 4 - BGL	21	0	32
	Unit 1c- Existing Landfill	17	1	27
	Unit 1c- Future Landfill	16	1	27
	Unit 2 - Residual Clay	19	5	29
	Unit 3a/5- VL & L Strength Rock	22	10	33
	Unit1a/1b - Fill	18	2	30



GOLDER	CLIENT Enviroguard Pty Ltd		PROJECT Erskine Park Landfill	A4
	DRAWN KT	DATE 22/04/2020	TITLE CS6-Long Term_Extreme Water Table	
	CHECKED LP	DATE 22/04/2020		
	SCALE N.T.S		PROJECT No 19135652 / CONTROL 19135652-006-R-RevA	FIFURE E.17

Color	Name	Unit Weight (kN/m³)	Cohesion (kPa)	Minimum Strength (kPa)	Non-constrained undrained strength ratio	SHANSEP function exponent	Overconsolidation Ratio Fn.	Cohesion' (kPa)	Phi' (°)
	Bed Rock								
	Engineered Fill	20						0	32
	Engineered Fill - BGL	20						0	32
	RSW-Unit 4	21						0	32
	RSW-Unit 4 - BGL	21						0	32
	Unit 1c- Existing Landfill	17						1	27
	Unit 1c- Future Landfill	16						1	27
	Unit 2 - Residual Clay _SHANCEP_LT	18		0	0.22	0.95	Depth vs OCR- LT		
	Unit 2 - Residual Clay _Undrained	19	150						
	Unit 3a/5- VL & L Strength Rock	22						10	33
	Unit1a/1b - Fill	18						2	30

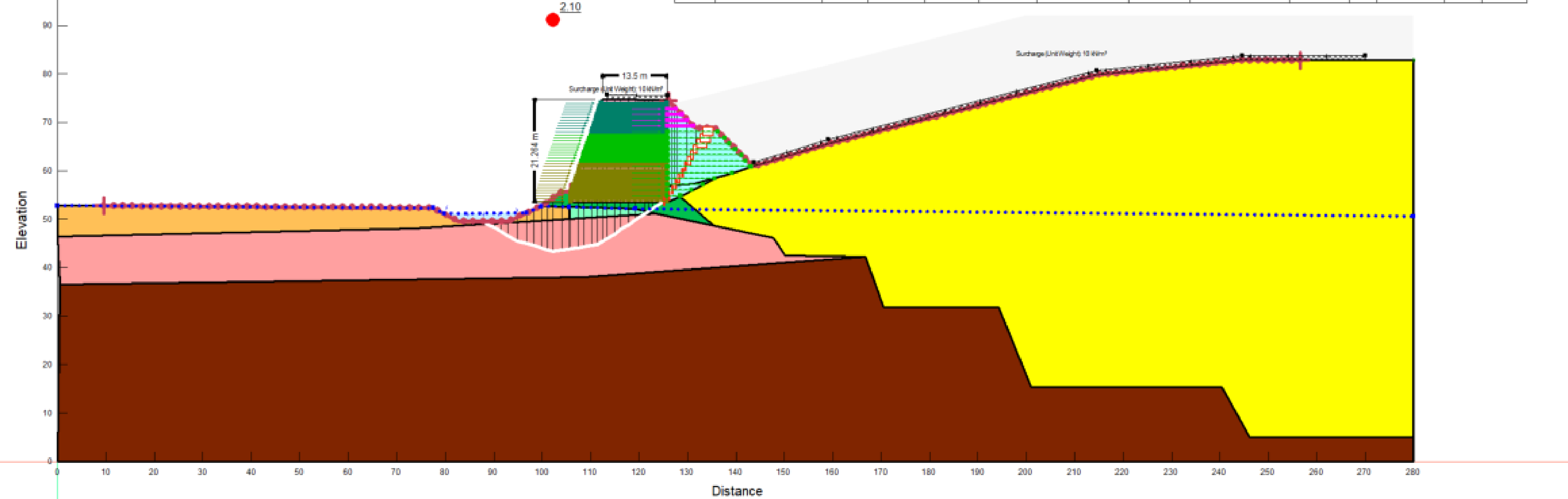


GOLDER

CLIENT Enviroguard Pty Ltd		PROJECT Erskine Park Landfill		A4
DRAWN KT	DATE 22/04/2020	TITLE CS6-Earthquake		
CHECKED LP	DATE 22/04/2020			
SCALE N.T.S		PROJECT No 19135652	CONTROL 19135652-006-R-RevA	FIFURE E.18

Date: 21/04/2020
Name: 1. Construction_End of Wall
Method: Morgenstern-Price

Color	Name	Unit Weight (kN/m³)	Cohesion (kPa)	Minimum Strength (kPa)	Maximum Strength (kPa)	Non-constrained undrained strength ratio	SHANSEP function exponent	Overconsolidation Ratio Fn.	Cohesion' (kPa)	Phi' (°)	Piezometric Line	B-bar	Add Weight
	Bed Rock										1	0	No
	Engineered Fill	20							0	32	1	0	Yes
	Engineered Fill - BGL	20							0	32	1	0	No
	RSW-Unit 4	21							0	32	1	0	Yes
	RSW-Unit 4 - BGL	21							0	32	1	0	No
	Unit 1c- Existing Landfill	17							1	27	1	0	No
	Unit 2 - Residual Clay_SHANSEP	19		0	150	0.22	0.95	OCR vs Depth			1	1	No
	Unit 2 - Residual Clay_Undrained	19	150								1	0	No
	Unit 3a/5- VL & L strength Rock	22							40	35	1	0	No
	Unit1a/1b - Fill	18							2	30	1	0	No

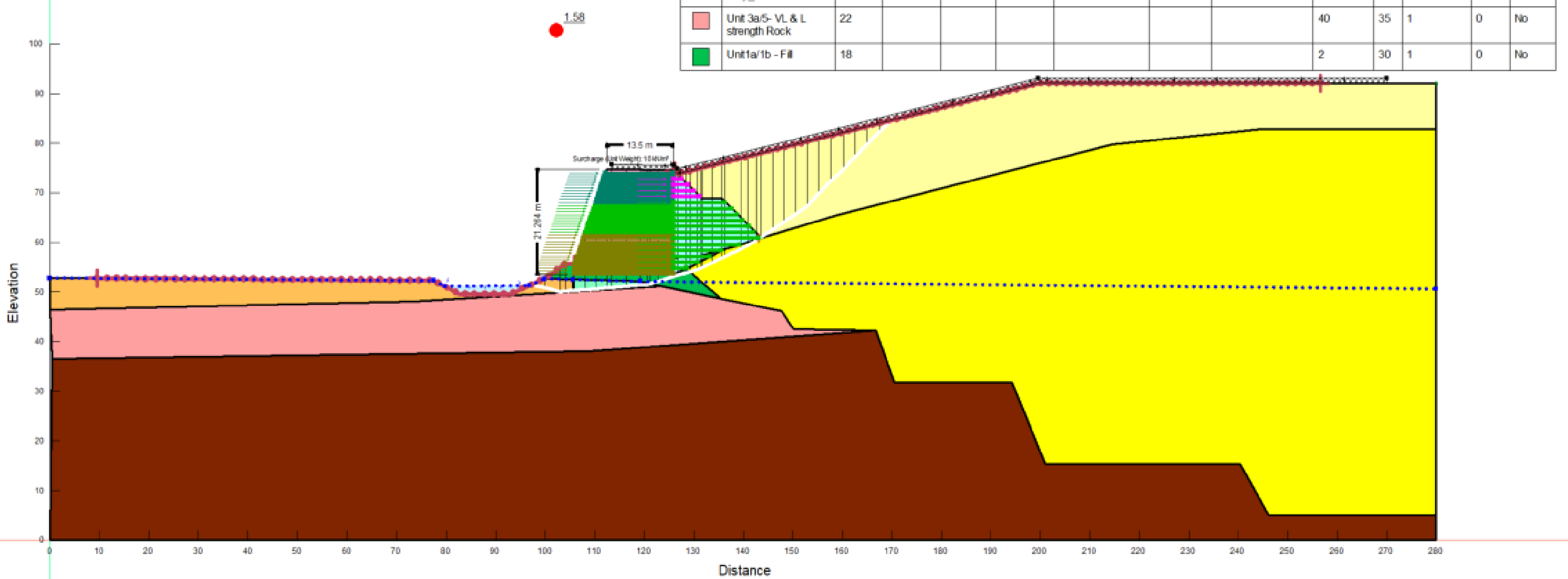


GOLDER	CLIENT Enviroguard Pty Ltd		PROJECT Erskine Park Landfill	A4
	DRAWN KT	DATE 22/04/2020	TITLE CS7-Construction_End of Wall	
	CHECKED LP	DATE 22/04/2020		
	SCALE N.T.S		PROJECT No 19135652 / CONTROL 19135652-006-R-RevA	FIGURE E.19

Document Set ID: 9209113
Version: 1, Version Date: 09/07/2020

Date: 21/04/2020
Name: 2. Construction_End of Landfill
Method: Morgenstern-Price

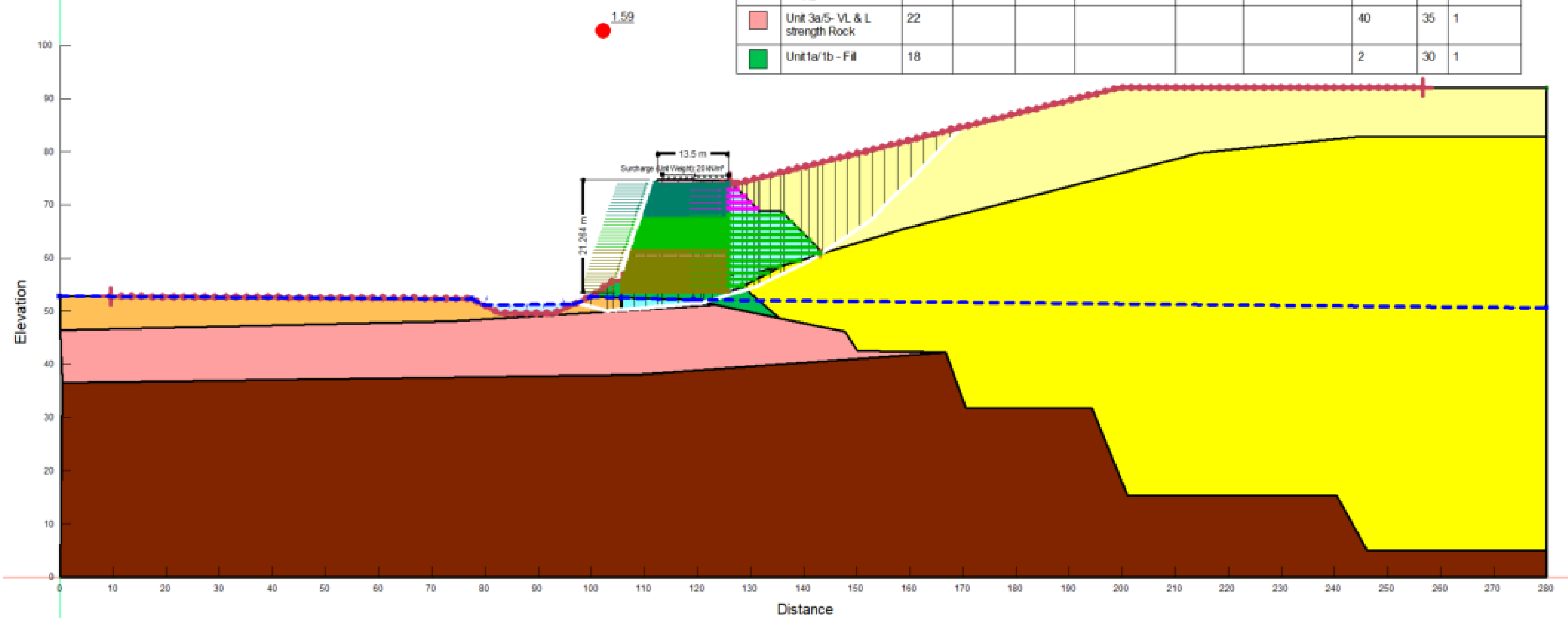
Color	Name	Unit Weight (kN/m³)	Cohesion (kPa)	Minimum Strength (kPa)	Maximum Strength (kPa)	Non-constrained undrained strength ratio	SHANSEP function exponent	Overconsolidation Ratio Fn.	Cohesion' (kPa)	Phi' (°)	Piezometric Line	B-bar	Add Weight
	Bed Rock										1	0	No
	Engineered Fill	20							0	32	1	0	Yes
	Engineered Fill - BGL	20							0	32	1	0	No
	RSW-Unit 4	21							0	32	1	0	Yes
	RSW-Unit 4 - BGL	21							0	32	1	0	No
	Unit 1c- Existing Landfill	17							1	27	1	0	No
	Unit 1c- Future Landfill	16							1	27	1	0	Yes
	Unit 2 - Residual Clay_SHANSEP	19		0	150	0.22	0.95	OCR vs Depth			1	1	No
	Unit 2 - Residual Clay_Undrained	19	150								1	0	No
	Unit 3a/5- VL & L strength Rock	22							40	35	1	0	No
	Unit1a/1b - Fill	18							2	30	1	0	No



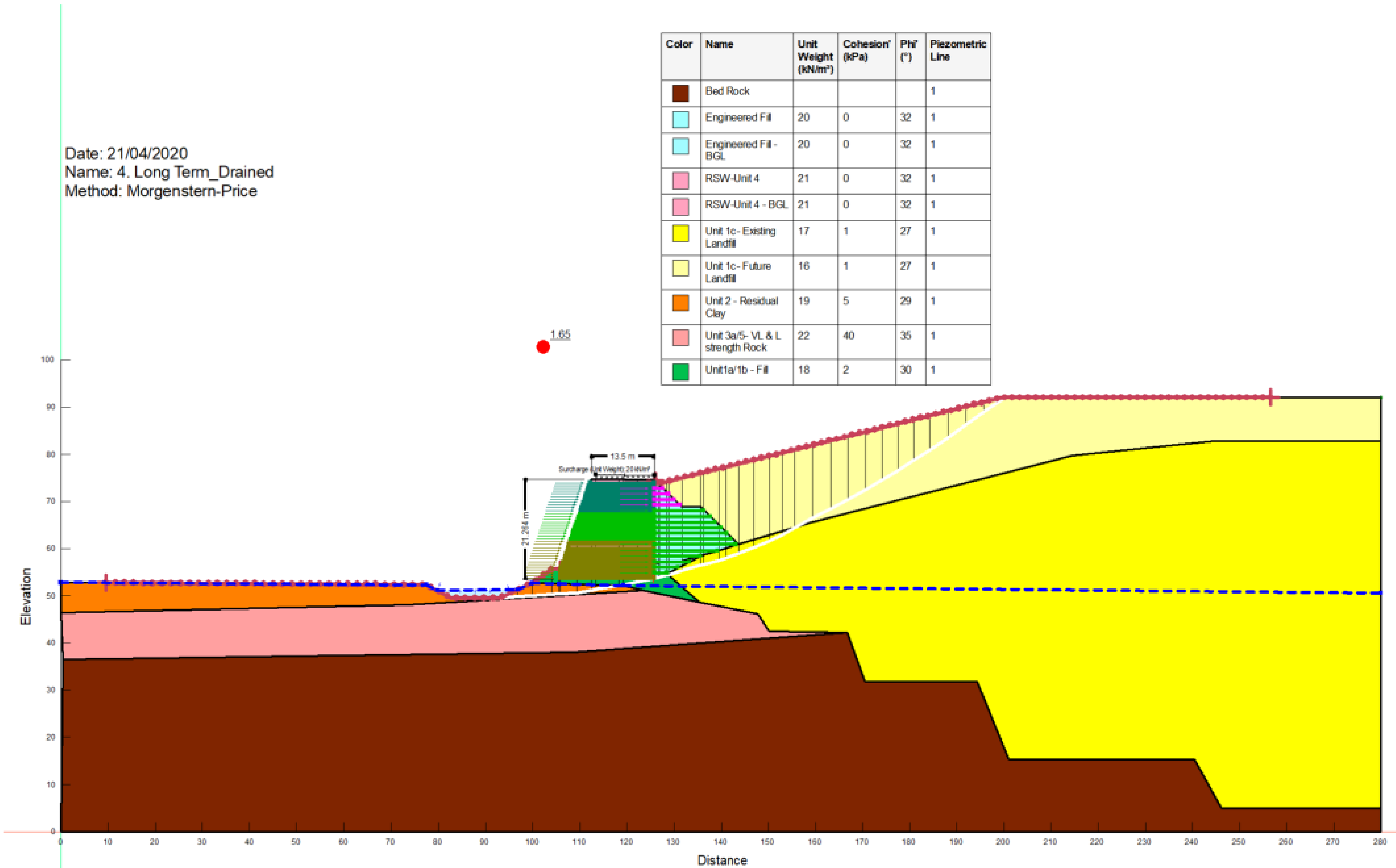
GOLDER	CLIENT Enviroguard Pty Ltd		PROJECT Erskine Park Landfill	A4
	DRAWN KT	DATE 22/04/2020	TITLE CS7-Construction_End of Landfill	
	CHECKED LP	DATE 22/04/2020		
	SCALE N.T.S		PROJECT No 19135652 / CONTROL 19135652-006-R-RevA	FIFURE E.20

Date: 21/04/2020
Name: 3. Long Term_Undrained
Method: Morgenstern-Price

Color	Name	Unit Weight (kN/m³)	Cohesion (kPa)	Minimum Strength (kPa)	Non-constrained undrained strength ratio	SHANSEP function exponent	Overconsolidation Ratio Fn.	Cohesion' (kPa)	Phi' (°)	Piezometric Line
	Bed Rock									1
	Engineered Fill	20						0	32	1
	Engineered Fill - BGL	20						0	32	1
	RSW-Unit 4	21						0	32	1
	RSW-Unit 4 - BGL	21						0	32	1
	Unit 1c - Existing Landfill	17						1	27	1
	Unit 1c - Future Landfill	16						1	27	1
	Unit 2 - Residual Clay_SHANSEP - LT	19		100	0.22	0.95	OCR vs Depth - LT			1
	Unit 2 - Residual Clay_Undrained	19	150							1
	Unit 3a/5- VL & L strength Rock	22						40	35	1
	Unit1a/1b - Fill	18						2	30	1

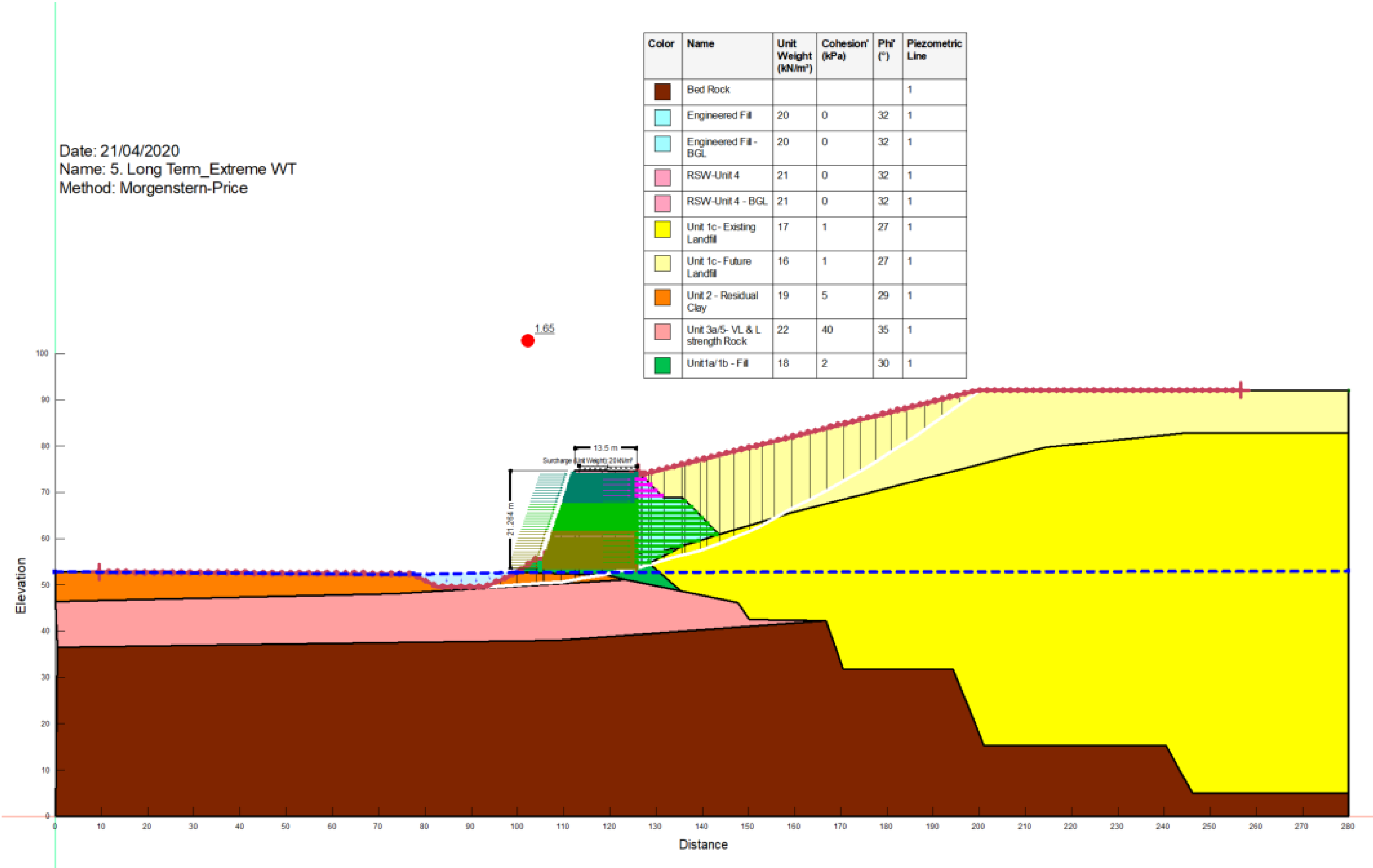


GOLDER	CLIENT Enviroguard Pty Ltd	PROJECT Erskine Park Landfill	A4
	DRAWN KT / DATE 22/04/2020	TITLE CS7-Long Term_Undrained	
	CHECKED LP / DATE 22/04/2020		
	SCALE N.T.S	PROJECT No 19135652 / CONTROL 19135652-006-R-RevA	FIGURE E.21



Color	Name	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Piezometric Line
	Bed Rock				1
	Engineered Fill	20	0	32	1
	Engineered Fill - BGL	20	0	32	1
	RSW-Unit 4	21	0	32	1
	RSW-Unit 4 - BGL	21	0	32	1
	Unit 1c - Existing Landfill	17	1	27	1
	Unit 1c - Future Landfill	16	1	27	1
	Unit 2 - Residual Clay	19	5	29	1
	Unit 3a/5 - VL & L strength Rock	22	40	35	1
	Unit1a/1b - Fill	18	2	30	1

GOLDER	CLIENT Enviroguard Pty Ltd		PROJECT Erskine Park Landfill	A4
	DRAWN KT	DATE 22/04/2020	TITLE CS7-Long Term_Drained
	CHECKED LP	DATE 22/04/2020	
	SCALE N.T.S		PROJECT No 19135652	CONTROL 19135652-006-R-RevA	FIFURE E.22



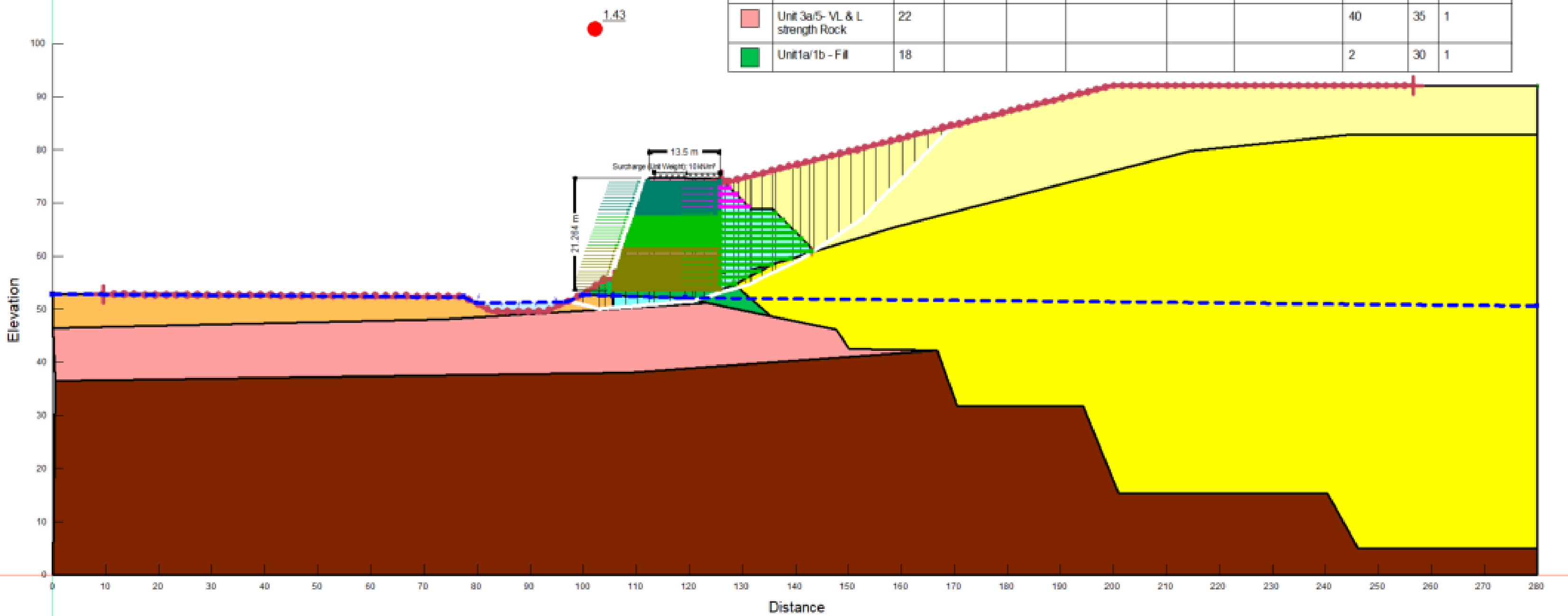
Color	Name	Unit Weight (kN/m^3)	Cohesion' (kPa)	Phi' (°)	Piezometric Line
	Bed Rock				1
	Engineered Fill	20	0	32	1
	Engineered Fill - BGL	20	0	32	1
	RSW-Unit 4	21	0	32	1
	RSW-Unit 4 - BGL	21	0	32	1
	Unit 1c- Existing Landfill	17	1	27	1
	Unit 1c- Future Landfill	16	1	27	1
	Unit 2 - Residual Clay	19	5	29	1
	Unit 3a/5- VL & L strength Rock	22	40	35	1
	Unit1a/1b - Fill	18	2	30	1

GOLDER

CLIENT Enviroguard Pty Ltd		PROJECT Erskine Park Landfill	A4
DRAWN KT	DATE 22/04/2020	TITLE CS7-Long Term_Extreme Water Table	
CHECKED LP	DATE 22/04/2020		
SCALE N.T.S		PROJECT No 19135652 / CONTROL 19135652-006-R-RevA	FIFURE E.23

Date: 21/04/2020
Name: 6. Earthquake
Method: Morgenstern-Price

Color	Name	Unit Weight (kN/m³)	Cohesion (kPa)	Minimum Strength (kPa)	Non-constrained undrained strength ratio	SHANSEP function exponent	Overconsolidation Ratio Fn.	Cohesion' (kPa)	Phi' (°)	Piezometric Line
	Bed Rock									1
	Engineered Fill	20						0	32	1
	Engineered Fill - BGL	20						0	32	1
	RSW-Unit 4	21						0	32	1
	RSW-Unit 4 - BGL	21						0	32	1
	Unit 1c- Existing Landfill	17						1	27	1
	Unit 1c- Future Landfill	16						1	27	1
	Unit 2 - Residual Clay_SHANSEP - LT	19		100	0.22	0.95	OCR vs Depth - LT			1
	Unit 2 - Residual Clay_Undrained	19	150							1
	Unit 3a/5- VL & L strength Rock	22						40	35	1
	Unit1a/1b - Fill	18						2	30	1



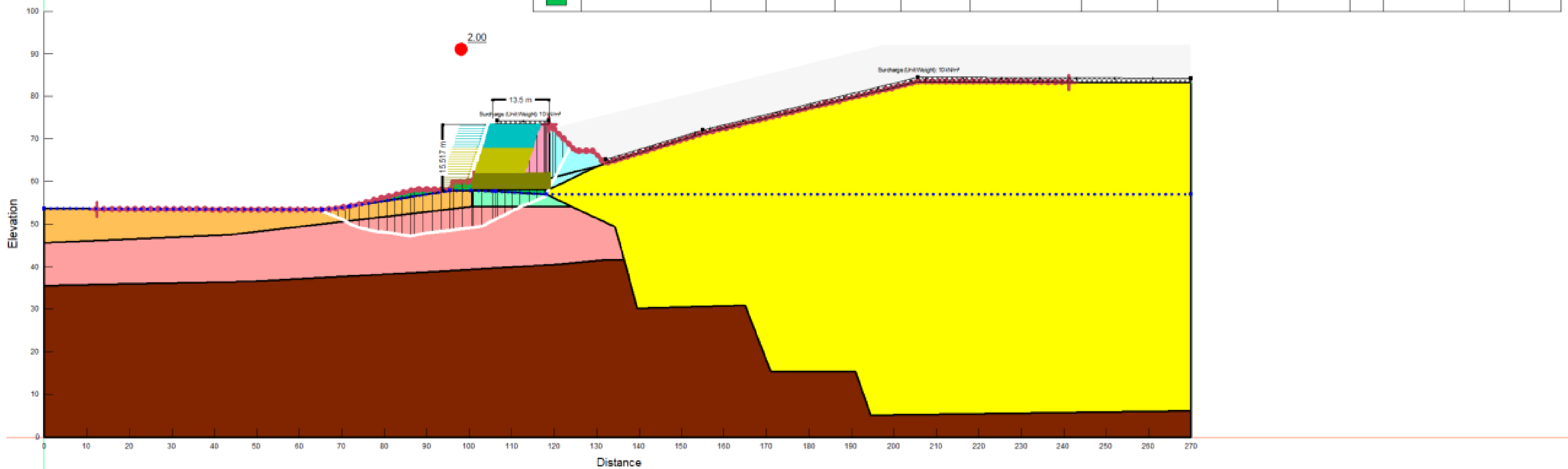
GOLDER

CLIENT Enviroguard Pty Ltd		PROJECT Erskine Park Landfill		A4
DRAWN KT	DATE 22/04/2020	TITLE CS7-Earthquake		
CHECKED LP	DATE 22/04/2020			
SCALE N.T.S		PROJECT No 19135652	CONTROL 19135652-006-R-RevA	FIFURE E.24

Document Set ID: 9209113
Version: 1, Version Date: 09/07/2020

Color	Name	Unit Weight (kN/m³)	Cohesion (kPa)	Minimum Strength (kPa)	Maximum Strength (kPa)	Non-constrained undrained strength ratio	SHANSEP function exponent	Overconsolidation Ratio Fn.	Cohesion' (kPa)	Phi' (°)	Piezometric Line	B-bar	Add Weight
	Bed Rock										1	0	No
	Engineered Fill	20							0	32	1	0	Yes
	Engineered Fill - BGL	20							0	32	1	0	No
	RSW-Unit 4	21							0	32	1	0	Yes
	RSW-Unit 4 - BGL	21							0	32	1	0	No
	Unit 1c- Existing Landfill	17							1	27	1	0	No
	Unit 2 - Residual Clay_SHANSEP	19		0	150	0.22	0.95	OCR vs Depth			1	1	No
	Unit 2 - Residual Clay_Undrained	19	150								1	0	No
	Unit 3a/5- VL & L strength Rock	22							10	33	1	0	No
	Unit1a/1b - Fill	18							2	30	1	0	No

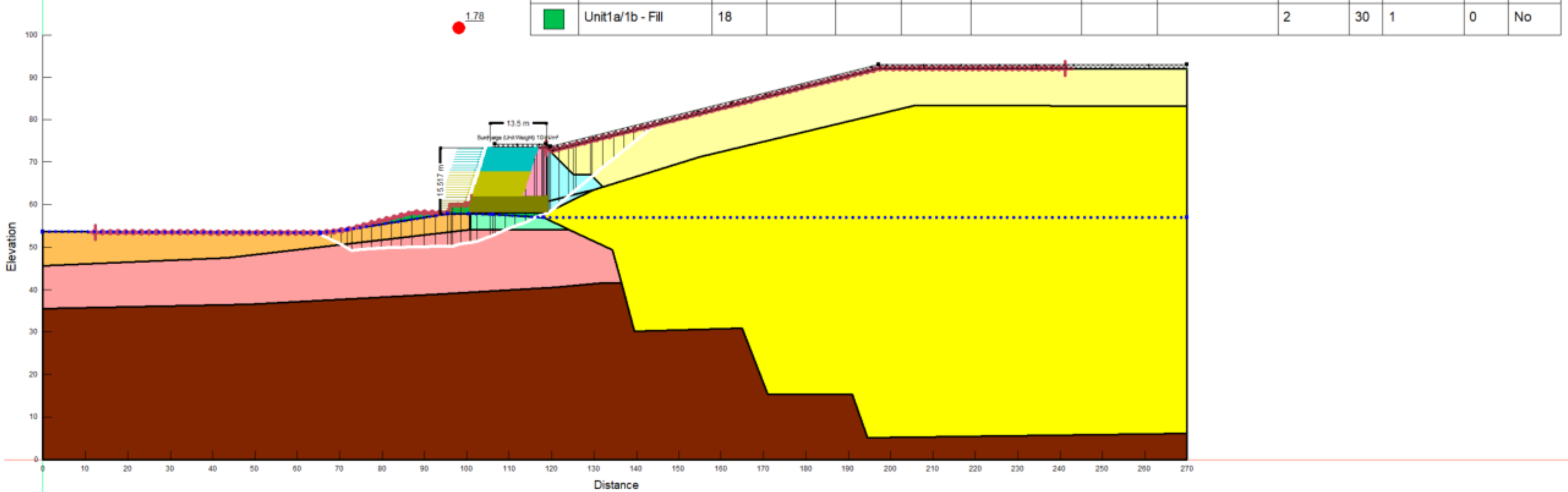
Date: 21/04/2020
Name: 1. Construction_End of Wall
Method: Morgenstern-Price



GOLDER	CLIENT Enviroguard Pty Ltd		PROJECT Erskine Park Landfill	A4
	DRAWN KT	DATE 22/04/2020	TITLE CS8-Construction_End of Wall	
	CHECKED LP	DATE 22/04/2020		
	SCALE N.T.S		PROJECT No 19135652 / CONTROL 19135652-006-R-RevA	FIFURE E.25

Date: 21/04/2020
Name: 2. Construction_End of Landfill
Method: Morgenstern-Price

Color	Name	Unit Weight (kN/m³)	Cohesion (kPa)	Minimum Strength (kPa)	Maximum Strength (kPa)	Non-constrained undrained strength ratio	SHANSEP function exponent	Overconsolidation Ratio Fn.	Cohesion' (kPa)	Phi' (°)	Piezometric Line	B-bar	Add Weight
	Bed Rock										1	0	No
	Engineered Fill	20							0	32	1	0	Yes
	Engineered Fill - BGL	20							0	32	1	0	No
	RSW-Unit 4	21							0	32	1	0	Yes
	RSW-Unit 4 - BGL	21							0	32	1	0	No
	Unit 1c- Existing Landfill	17							1	27	1	0	No
	Unit 1c- Future Landfill	16							1	27	1	0	Yes
	Unit 2 - Residual Clay_SHANSEP	19		0	150	0.22	0.95	OCR vs Depth			1	1	No
	Unit 2 - Residual Clay_Undrained	19	150								1	0	No
	Unit 3a/5- VL & L strength Rock	22							10	33	1	0	No
	Unit1a/1b - Fill	18							2	30	1	0	No

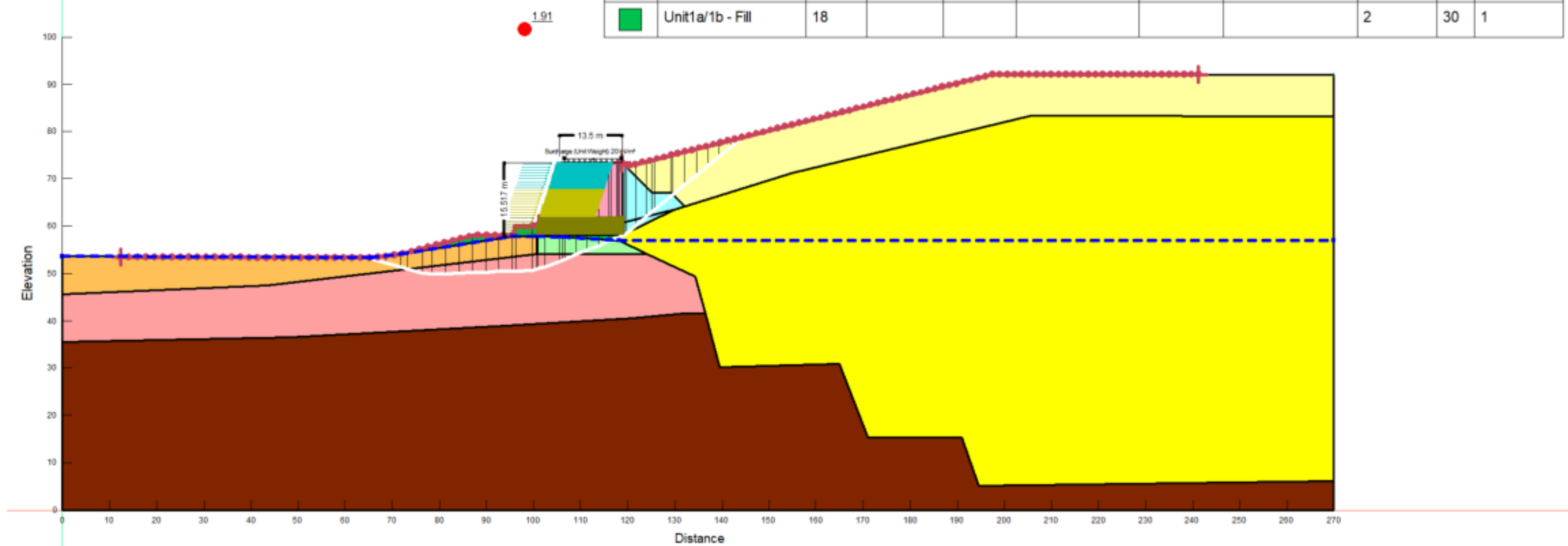


GOLDER

CLIENT Enviroguard Pty Ltd		PROJECT Erskine Park Landfill	A4
DRAWN KT	DATE 22/04/2020	TITLE CS8-Construction_End of Landfill	
CHECKED LP	DATE 22/04/2020		
SCALE N.T.S		PROJECT No 19135652	CONTROL 19135652-006-R-RevA	FIFURE E.26

Color	Name	Unit Weight (kN/m³)	Cohesion (kPa)	Minimum Strength (kPa)	Non-constrained undrained strength ratio	SHANSEP function exponent	Overconsolidation Ratio Fn.	Cohesion' (kPa)	Phi' (°)	Piezometric Line
	Bed Rock									1
	Engineered Fill	20						0	32	1
	Engineered Fill - BGL	20						0	32	1
	RSW-Unit 4	21						0	32	1
	RSW-Unit 4 - BGL	21						0	32	1
	Unit 1c- Existing Landfill	17						1	27	1
	Unit 1c- Future Landfill	16						1	27	1
	Unit 2 - Residual Clay_SHANSEP - LT	19		100	0.22	0.95	OCR vs Depth - LT			1
	Unit 2 - Residual Clay_Undrained	19	150							1
	Unit 3a/5- VL & L strength Rock	22						10	33	1
	Unit1a/1b - Fill	18						2	30	1

Date: 21/04/2020
Name: 3. Long Term_Undrained
Method: Morgenstern-Price

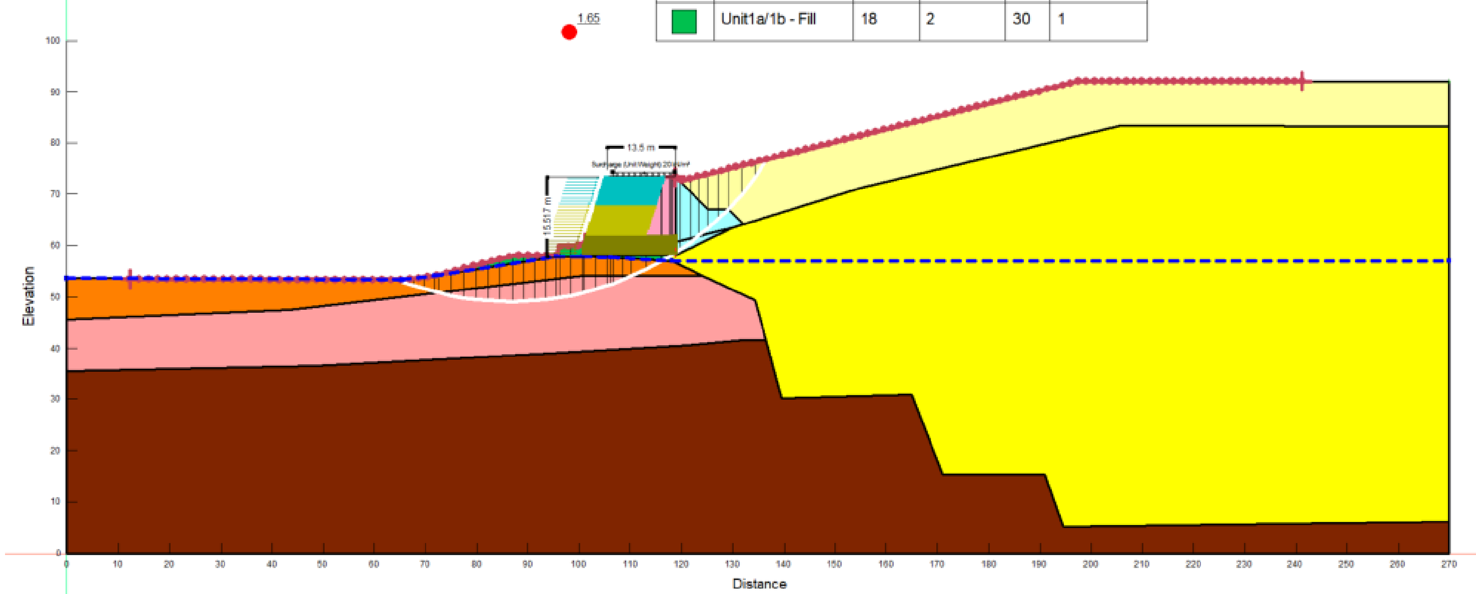


GOLDER

CLIENT Enviroguard Pty Ltd		PROJECT Erskine Park Landfill	A4
DRAWN KT	DATE 22/04/2020	TITLE CS8-Long Term_Undrained	
CHECKED LP	DATE 22/04/2020		
SCALE N.T.S		PROJECT No 19135652	CONTROL 19135652-006-R-RevA	FIFURE E.27

Date: 21/04/2020
Name: 4. Long Term_Drained
Method: Morgenstern-Price

Color	Name	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Piezometric Line
	Bed Rock				1
	Engineered Fill	20	0	32	1
	Engineered Fill - BGL	20	0	32	1
	RSW-Unit 4	21	0	32	1
	RSW-Unit 4 - BGL	21	0	32	1
	Unit 1c- Existing Landfill	17	1	27	1
	Unit 1c- Future Landfill	16	1	27	1
	Unit 2 - Residual Clay	19	5	29	1
	Unit 3a/5- VL & L strength Rock	22	10	33	1
	Unit1a/1b - Fill	18	2	30	1



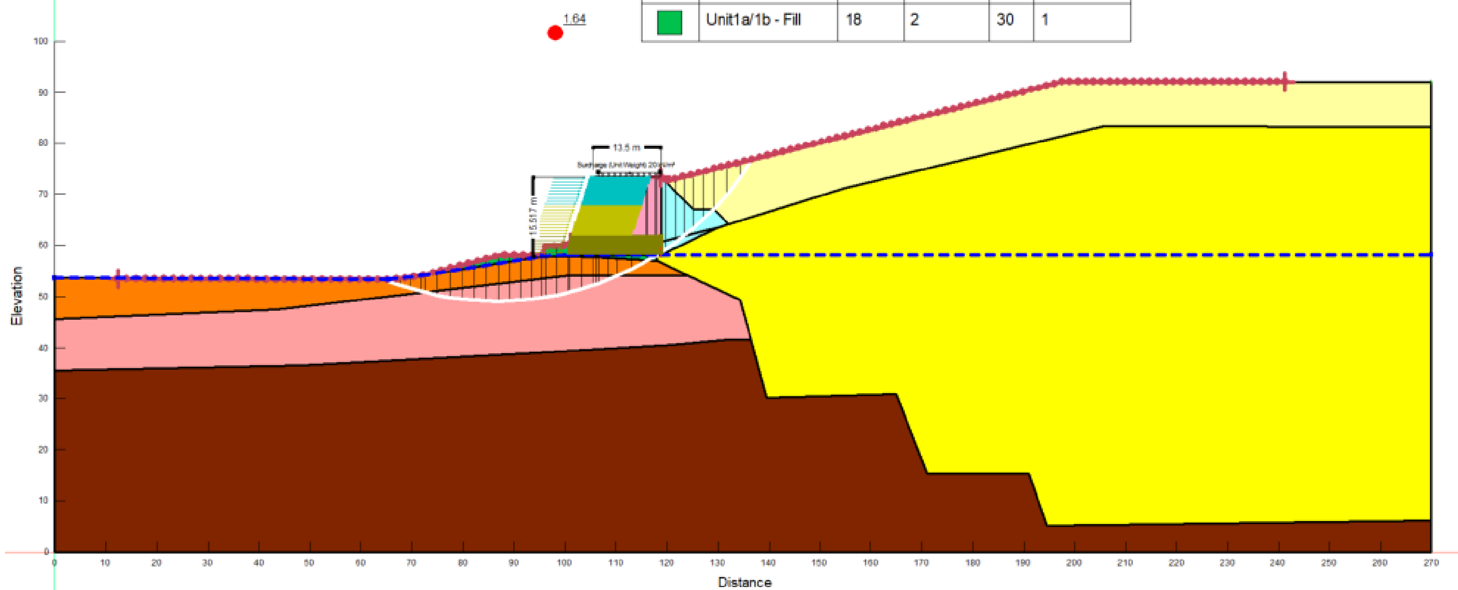
GOLDER

CLIENT Enviroguard Pty Ltd		PROJECT Erskine Park Landfill		A4
DRAWN KT	DATE 22/04/2020	TITLE CS8-Long Term_Drained		
CHECKED LP	DATE 22/04/2020			
SCALE N.T.S		PROJECT No 19135652	CONTROL 19135652-006-R-RevA	FIGURE E.28

Document Set ID: 9209113
Version: 1, Version Date: 09/07/2020

Color	Name	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Piezometric Line
	Bed Rock				1
	Engineered Fill	20	0	32	1
	Engineered Fill - BGL	20	0	32	1
	RSW-Unit 4	21	0	32	1
	RSW-Unit 4 - BGL	21	0	32	1
	Unit 1c- Existing Landfill	17	1	27	1
	Unit 1c- Future Landfill	16	1	27	1
	Unit 2 - Residual Clay	19	5	29	1
	Unit 3a/5- VL & L strength Rock	22	10	33	1
	Unit1a/1b - Fill	18	2	30	1

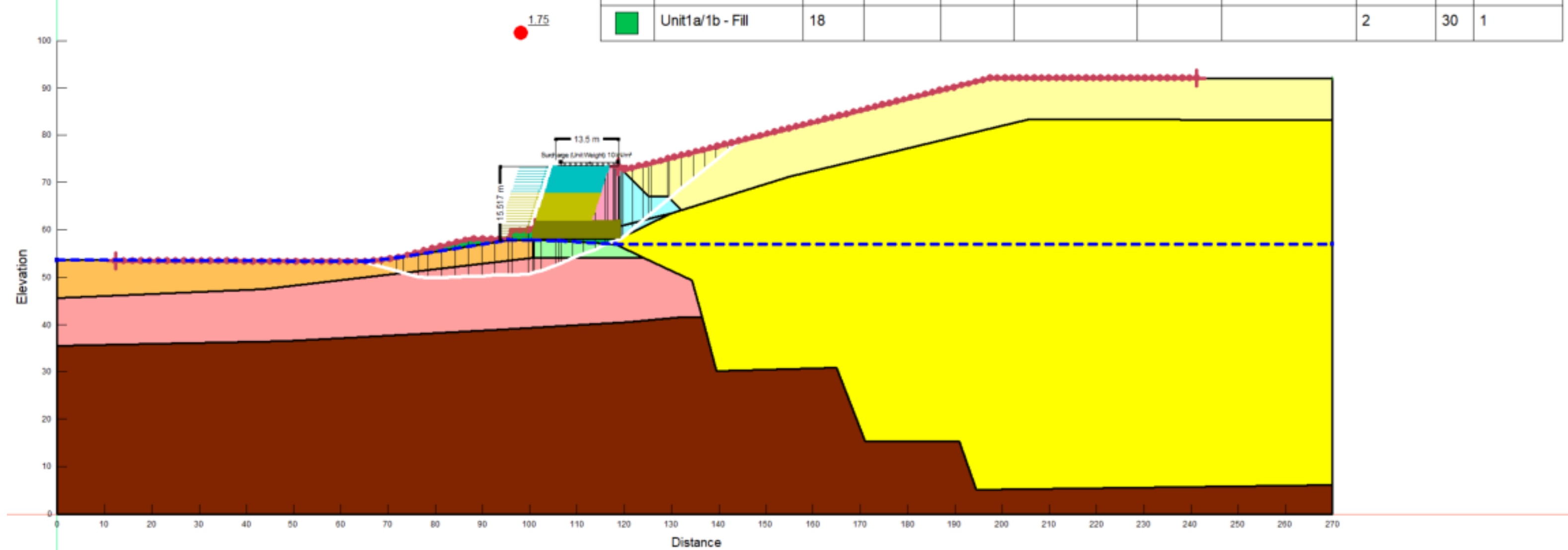
Date: 21/04/2020
Name: 5. Long Term_Extreme WT
Method: Morgenstern-Price



GOLDER	CLIENT Enviroguard Pty Ltd	PROJECT Erskine Park Landfill	A4
	DRAWN KT / DATE 22/04/2020	TITLE CS8-Long Term_Extreme Water Table	
	CHECKED LP / DATE 22/04/2020		
	SCALE N.T.S	PROJECT No 19135652 / CONTROL 19135652-006-R-RevA	FIFURE E.29

Color	Name	Unit Weight (kN/m³)	Cohesion (kPa)	Minimum Strength (kPa)	Non-constrained undrained strength ratio	SHANSEP function exponent	Overconsolidation Ratio Fn.	Cohesion' (kPa)	Phi' (°)	Piezometric Line
	Bed Rock									1
	Engineered Fill	20						0	32	1
	Engineered Fill - BGL	20						0	32	1
	RSW-Unit 4	21						0	32	1
	RSW-Unit 4 - BGL	21						0	32	1
	Unit 1c- Existing Landfill	17						1	27	1
	Unit 1c- Future Landfill	16						1	27	1
	Unit 2 - Residual Clay_SHANSEP - LT	19		100	0.22	0.95	OCR vs Depth - LT			1
	Unit 2 - Residual Clay_Undrained	19	150							1
	Unit 3a/5- VL & L strength Rock	22						10	33	1
	Unit1a/1b - Fill	18						2	30	1

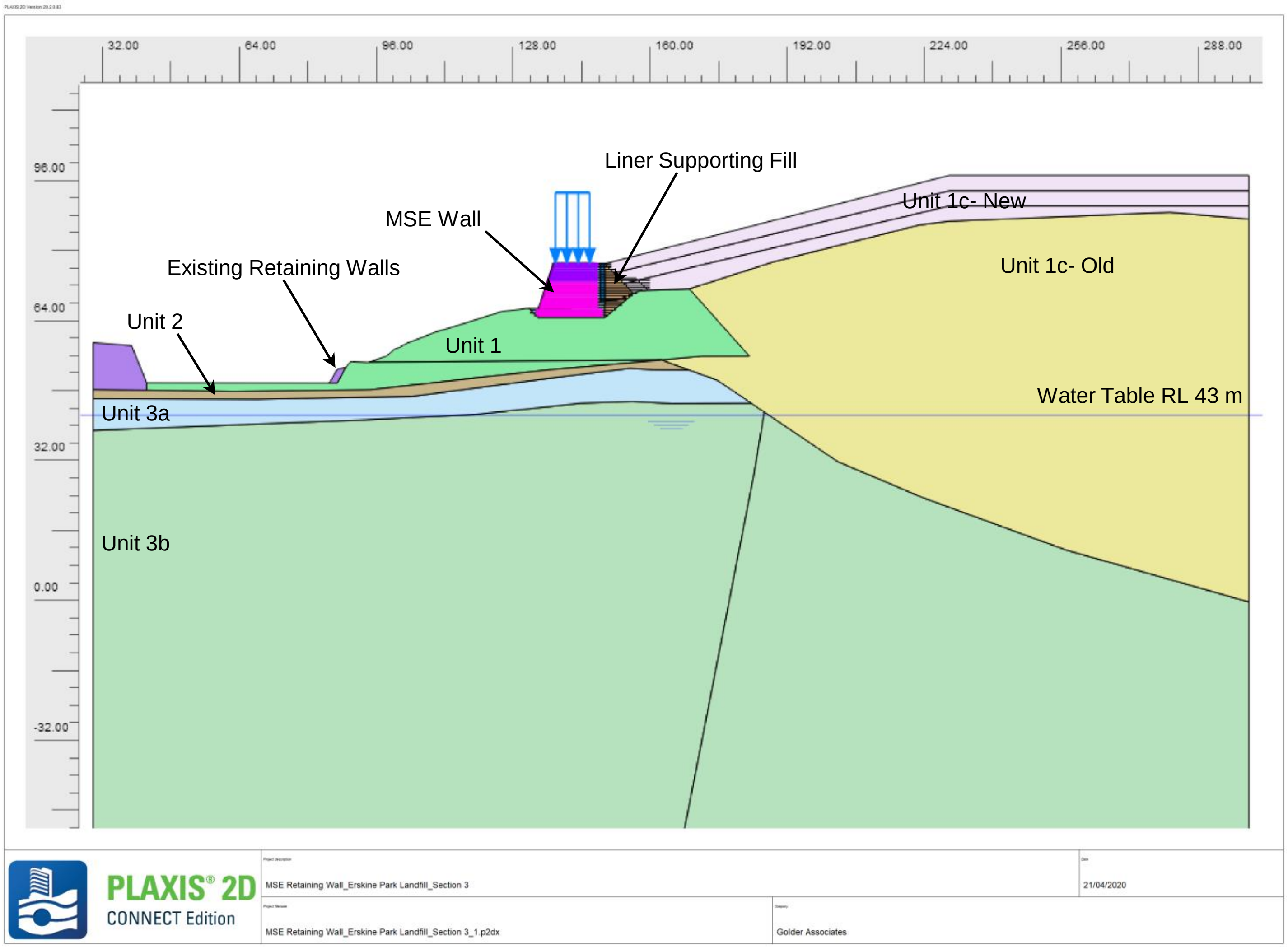
Date: 21/04/2020
Name: 6. Earthquake
Method: Morgenstern-Price



GOLDER	CLIENT Enviroguard Pty Ltd		PROJECT Erskine Park Landfill	A4
	DRAWN KT	DATE 22/04/2020	TITLE CS8-Earthquake	
	CHECKED LP	DATE 22/04/2020		
	SCALE N.T.S		PROJECT No 19135652 / CONTROL 19135652-006-R-RevA	FIFURE E.30

APPENDIX F

Wall Movement

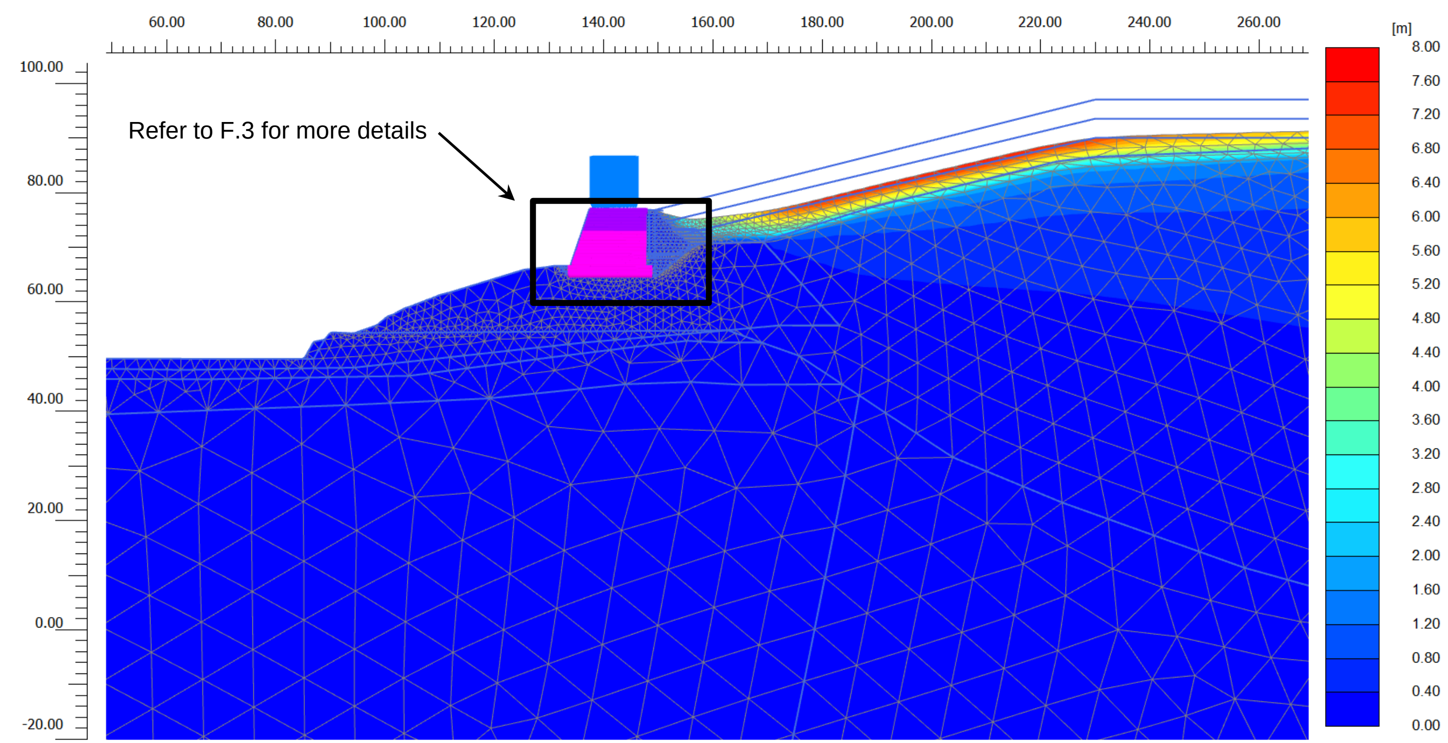


Section 3 – PLAXIS Model

Cross Section View – Output from PLAXIS 2D

GOLDER	CLIENT Enviroguard Pty Ltd		PROJECT MSE Retaining Wall	A4
	DRAWN KT	DATE 22/04/2020	TITLE Preliminary Design Report	
	CHECKED LP	DATE 22/04/2020		
	SCALE N.T.S		PROJECT No 19135652 / CONTROL 19135652-006-R-RevA	FIFURE F.1

Output Version 20.2.0.83



Total displacements |u| (scaled up 5.00 times) (Time 39.49*10^3 day)

Maximum value = 7.697 m (Element 34 at Node 8995)

PLAXIS® 2D CONNECT Edition	*Project description* MSE Retaining Wall_Erskine Park Landfill_Section 3		*Date* 21/04/2020
	Project filename MSE Retaining Wall_Erskine ...	*Step* 663	*Company* Golder Associates

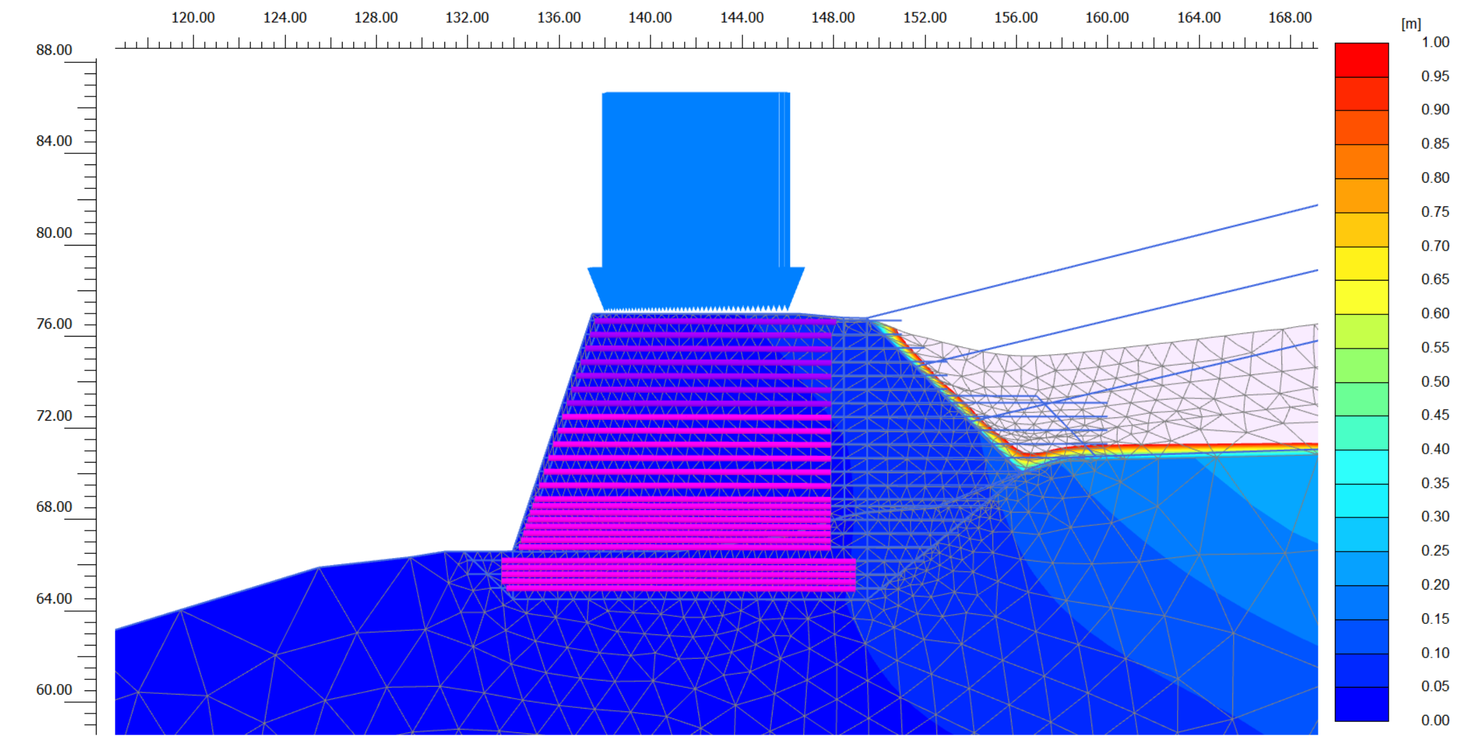
Section 3 – Predicted Total Settlement after 100 yrs

Cross Section View – Output from PLAXIS 2D

GOLDER	CLIENT Enviroguard Pty Ltd		PROJECT MSE Retaining Wall	A4
	DRAWN KT	DATE 22/04/2020	TITLE Preliminary Design Report	
	CHECKED LP	DATE 22/04/2020		
	SCALE N.T.S		PROJECT No 19135652 · CONTROL 19135652-006-R-RevA	FIGURE F.2

Document Set ID: 9209113
Version: 1, Version Date: 09/07/2020

Output Version 20.2.0.83



Total displacements |u| (scaled up 5.00 times) (Time 39.49*10^3 day)

Maximum value = 7.697 m (Element 34 at Node 8995)

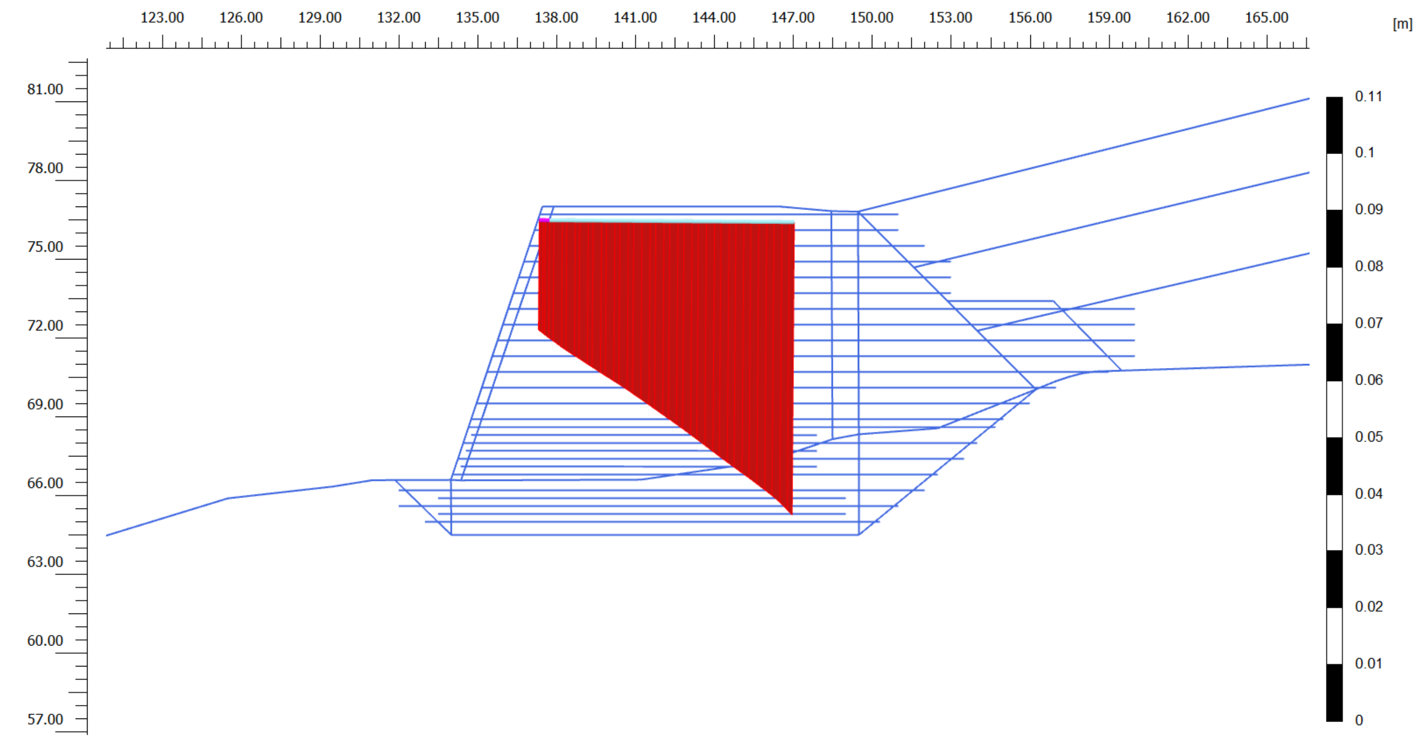
PLAXIS® 2D CONNECT Edition	*Project description* MSE Retaining Wall_Erskine Park Landfill_Section 3			*Date* 21/04/2020
	Project filename MSE Retaining Wall_Erskine ...	*Step* 663	*Company* Golder Associates	

Section 3 – Predicted Total Settlement after 100 yrs

Cross Section View – Output from PLAXIS 2D

GOLDER	CLIENT Enviroguard Pty Ltd		PROJECT MSE Retaining Wall	A4
	DRAWN KT	DATE 22/04/2020	TITLE Preliminary Design Report	
	CHECKED LP	DATE 22/04/2020		
	SCALE N.T.S		PROJECT No 19135652 / CONTROL 19135652-006-R-RevA	FIGURE F.3

Output Version 20.2.0.83



Total displacements u_y (scaled up 200 times) (Time $39.49*10^3$ day)

Maximum value = -0.02086 m

Minimum value = -0.05580 m

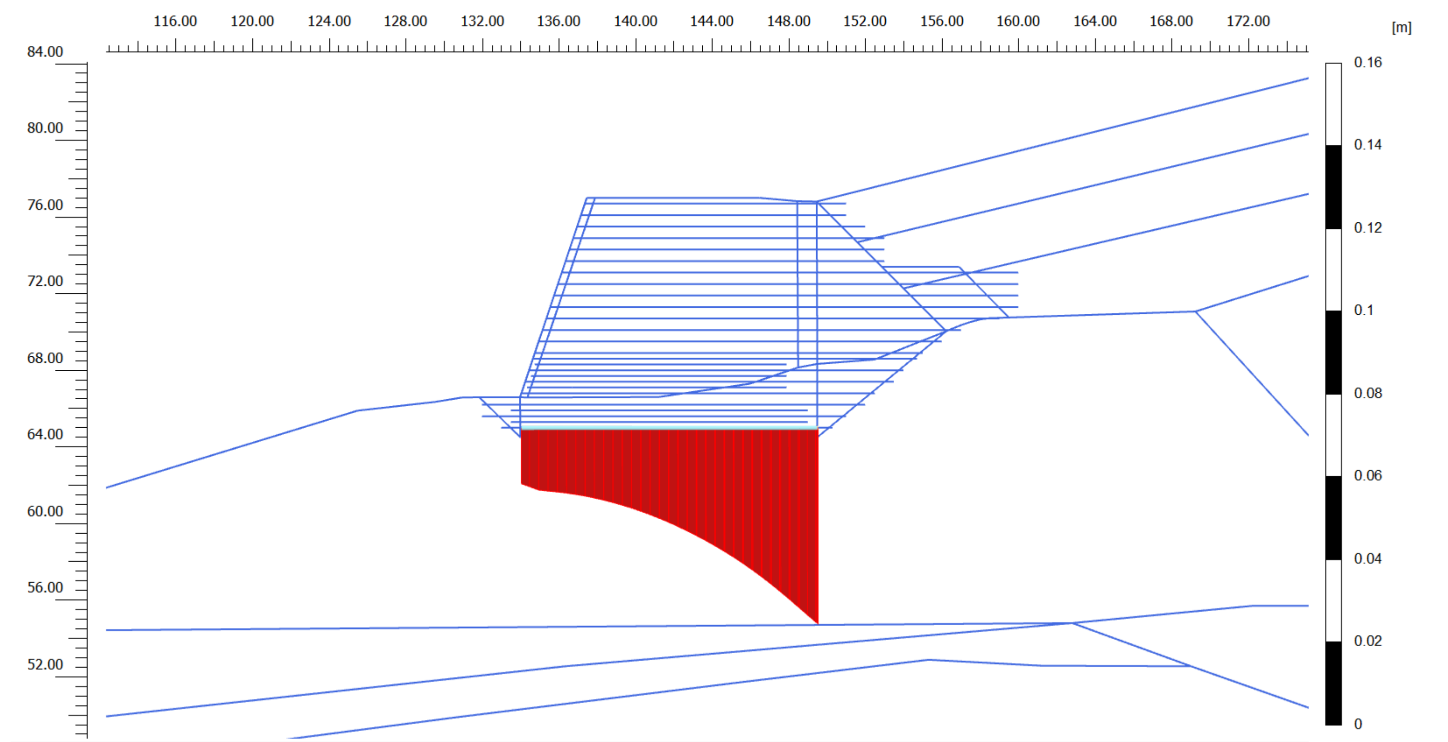
	Project description		Date
PLAXIS® 2D CONNECT Edition	MSE Retaining Wall_Erskine Park Landfill_Section 3		22/04/2020
	Project filename: MSE Retaining Wall_Erskine ...	Step: 663	Company: Golder Associates

Section 3 – Predicted Vertical Settlement after 100 yrs at the Top of the Wall

Cross Section View – Output from PLAXIS 2D

GOLDER	CLIENT: Enviroguard Pty Ltd	PROJECT: MSE Retaining Wall	A4
	DRAWN: KT / DATE: 22/04/2020	TITLE: Preliminary Design Report	
	CHECKED: LP / DATE: 22/04/2020		
	SCALE: N.T.S	PROJECT No: 19135652 / CONTROL: 19135652-006-R-RevA	FIGURE: F.4

Output Version 20.2.0.83



Total displacements u_y (scaled up 200 times) (Time 39.49×10^3 day)

Maximum value = -0.01455 m

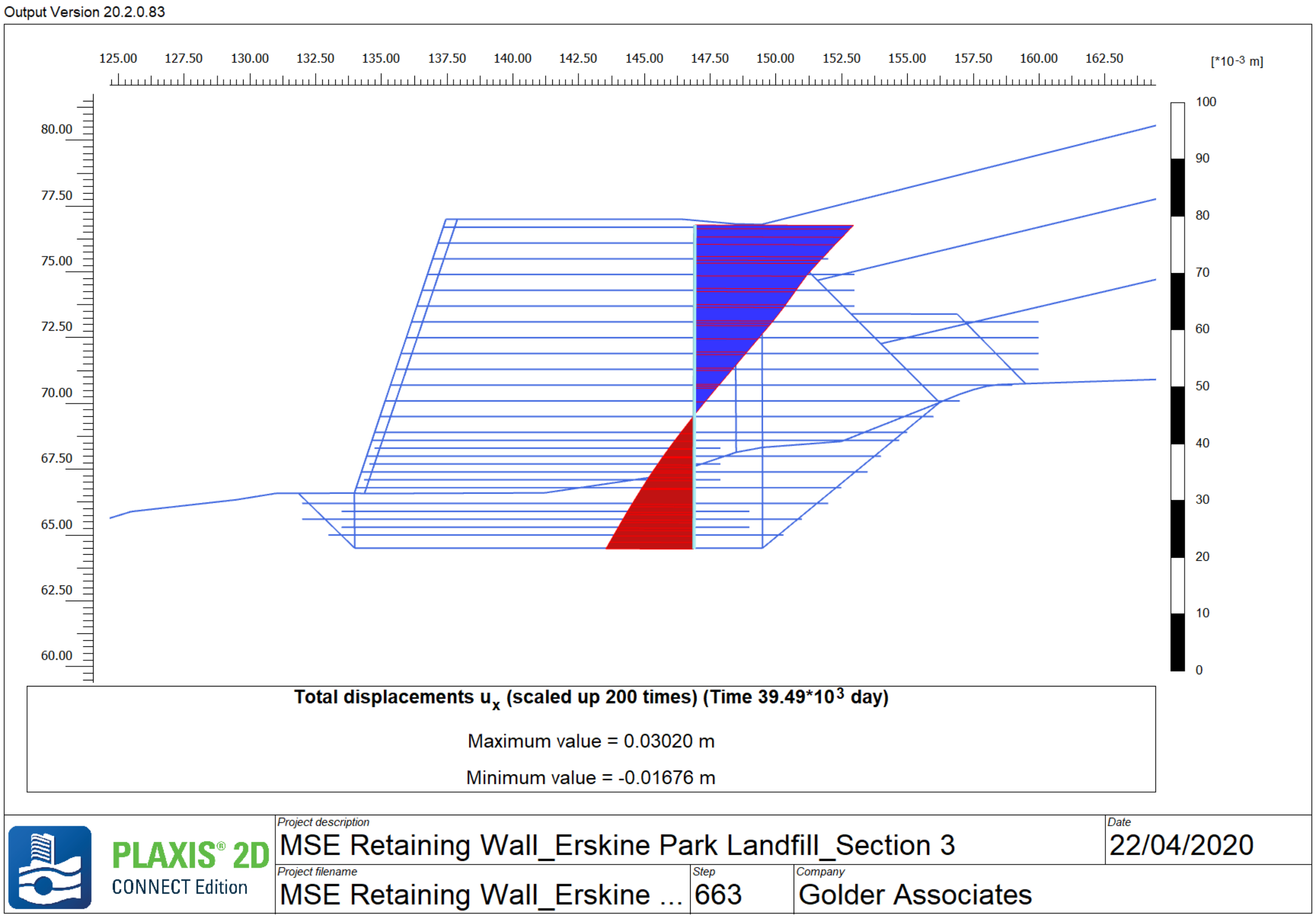
Minimum value = -0.05125 m

PLAXIS® 2D CONNECT Edition	*Project description* MSE Retaining Wall_Erskine Park Landfill_Section 3		*Date* 21/04/2020
	Project filename MSE Retaining Wall_Erskine ...	*Step* 663	*Company* Golder Associates

Section 3 – Predicted Vertical Settlement after 100 yrs at the Bottom of the Wall

Cross Section View – Output from PLAXIS 2D

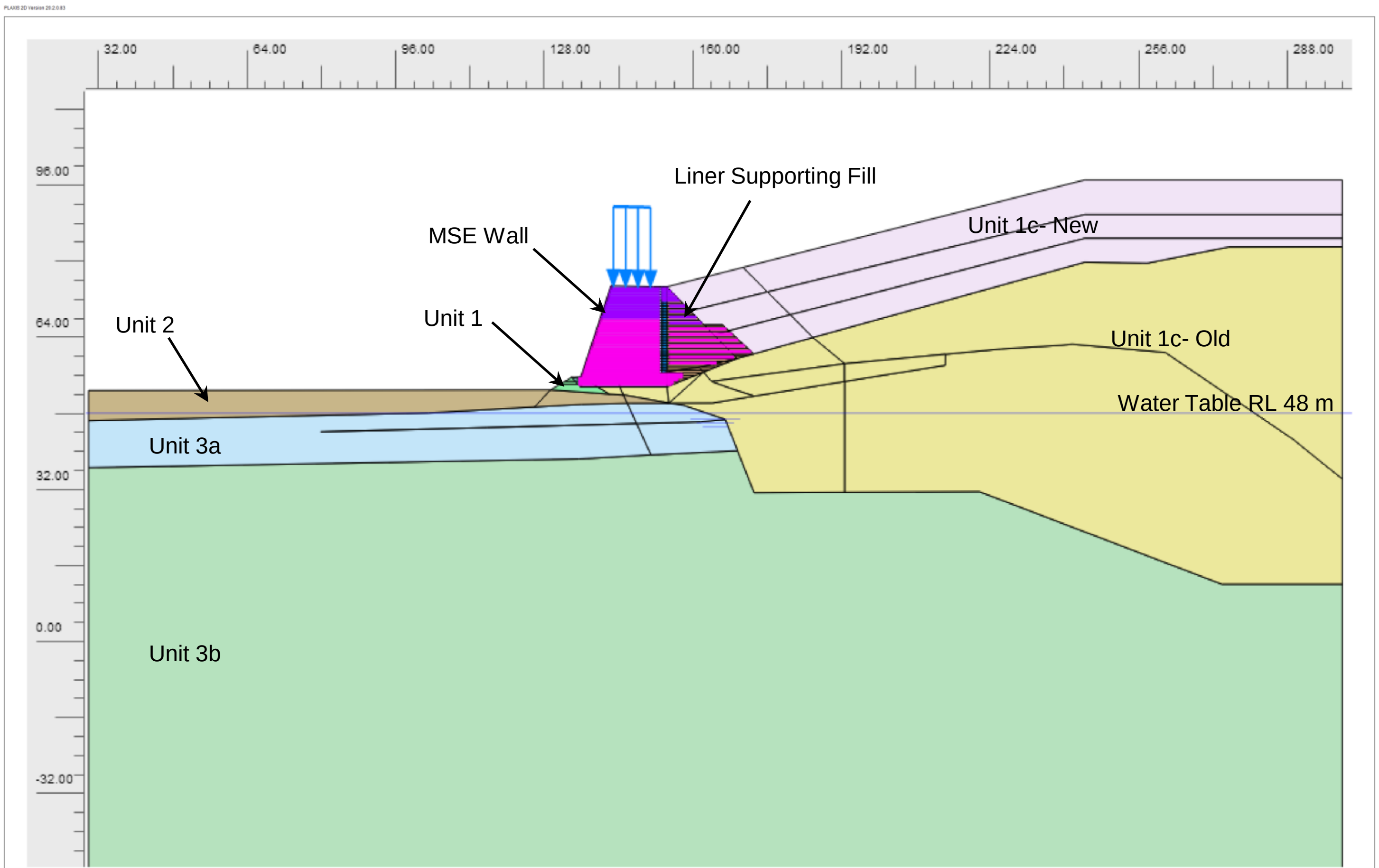
GOLDER	CLIENT Enviroguard Pty Ltd		PROJECT MSE Retaining Wall	A4
	DRAWN KT	DATE 22/04/2020	TITLE Preliminary Design Report	
	CHECKED LP	DATE 22/04/2020		
	SCALE N.T.S		PROJECT No 19135652 / CONTROL 19135652-006-R-RevA	FIFURE F.5



Section 3 – Predicted Horizontal Movement after 100 yrs

GOLDER	CLIENT Enviroguard Pty Ltd		PROJECT MSE Retaining Wall	A4
	DRAWN KT	DATE 22/04/2020	TITLE Preliminary Design Report	
	CHECKED LP	DATE 22/04/2020		
	SCALE N.T.S		PROJECT No 19135652 / CONTROL 19135652-006-R-RevA	FIGURE F.6

Cross Section View – Output from PLAXIS 2D



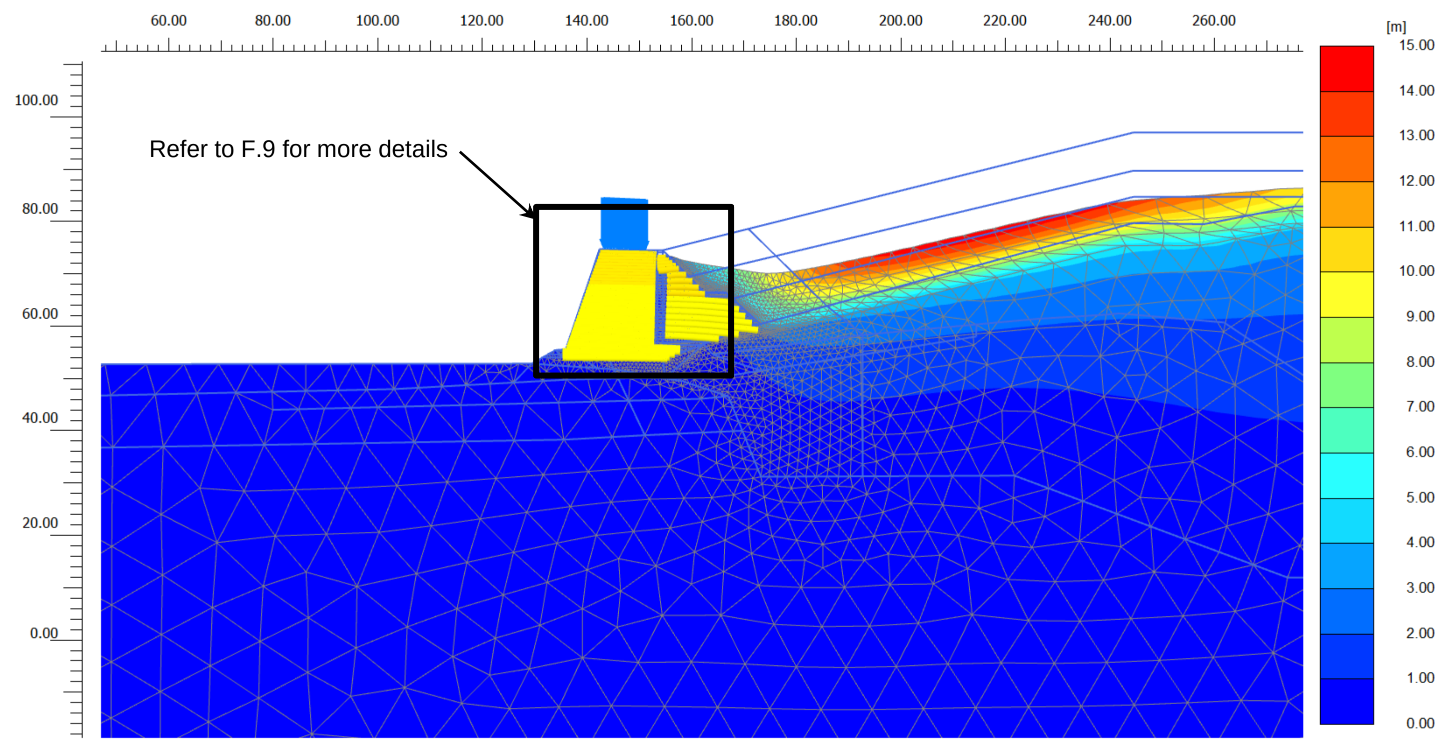
PLAXIS® 2D CONNECT Edition	MSE Retaining Wall_Erskine Park Landfill_Section 3A	21/04/2020
	4. MSE_Section 3A_FullLandfill.p2dx	Golder Associates

Section 3A – PLAXIS Model

Cross Section View – Output from PLAXIS 2D

GOLDER	CLIENT Enviroguard Pty Ltd		PROJECT MSE Retaining Wall	A4
	DRAWN KT	DATE 22/04/2020	TITLE Preliminary Design Report	
	CHECKED LP	DATE 22/04/2020		
	SCALE N.T.S		PROJECT No 19135652 / CONTROL 19135652-006-R-RevA	FIFURE F.7

Output Version 20.2.0.83



Total displacements |u| (scaled up 0.500 times) (Time 39.53*10^3 day)

Maximum value = 14.97 m (Element 18 at Node 2542)

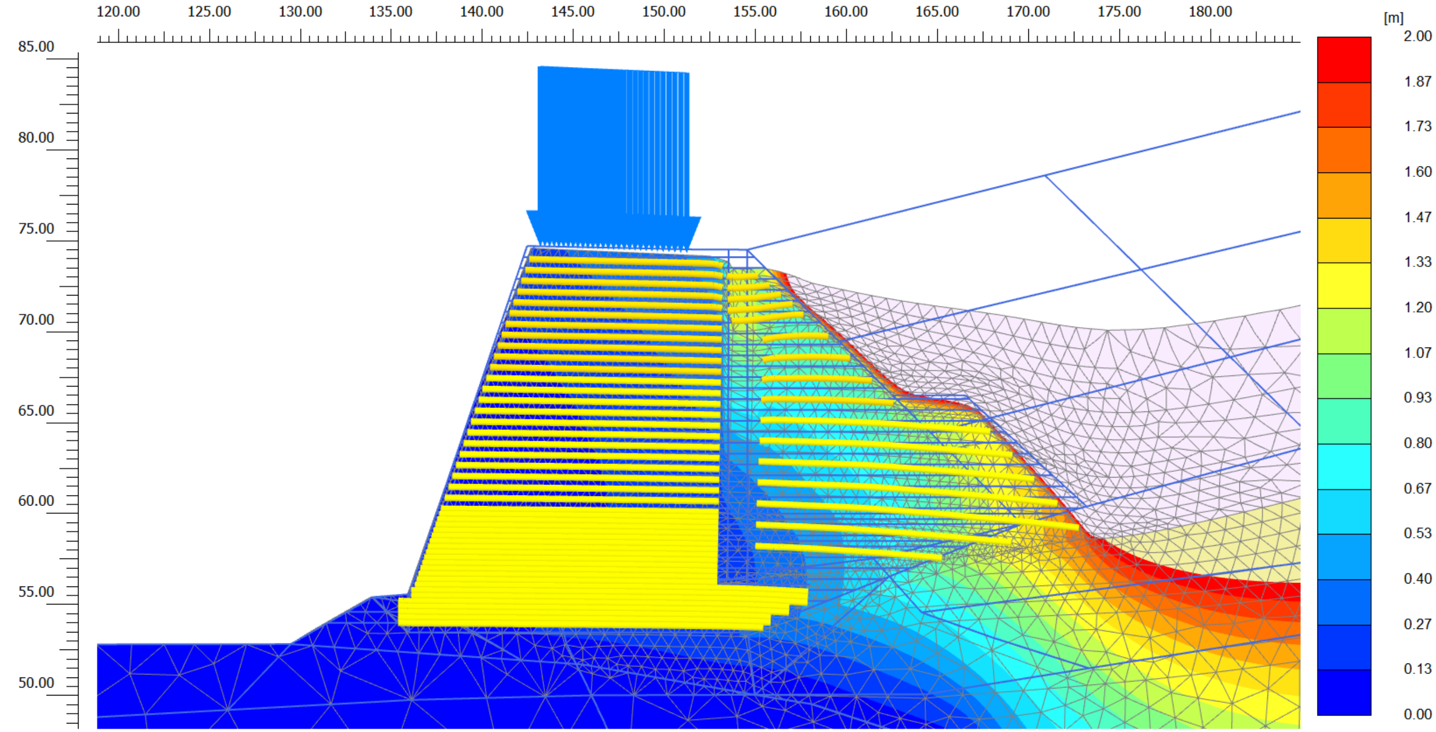
PLAXIS® 2D CONNECT Edition	*Project description* MSE Retaining Wall_Erskine Park Landfill_Section 3A			*Date* 21/04/2020
	Project filename 4. MSE_Section 3A_FullLandfill	*Step* 910	*Company* Golder Associates	

Section 3A – Predicted Total Settlement after 100 yrs

Cross Section View – Output from PLAXIS 2D

GOLDER	CLIENT Enviroguard Pty Ltd		PROJECT MSE Retaining Wall	A4
	DRAWN KT	DATE 22/04/2020	TITLE Preliminary Design Report	
	CHECKED LP	DATE 22/04/2020		
	SCALE N.T.S		PROJECT No 19135652 CONTROL 19135652-006-R-RevA	FIFURE F.8

Output Version 20.2.0.83



Total displacements |u| (scaled up 0.500 times) (Time $39.53*10^3$ day)

Maximum value = 14.97 m (Element 18 at Node 2542)

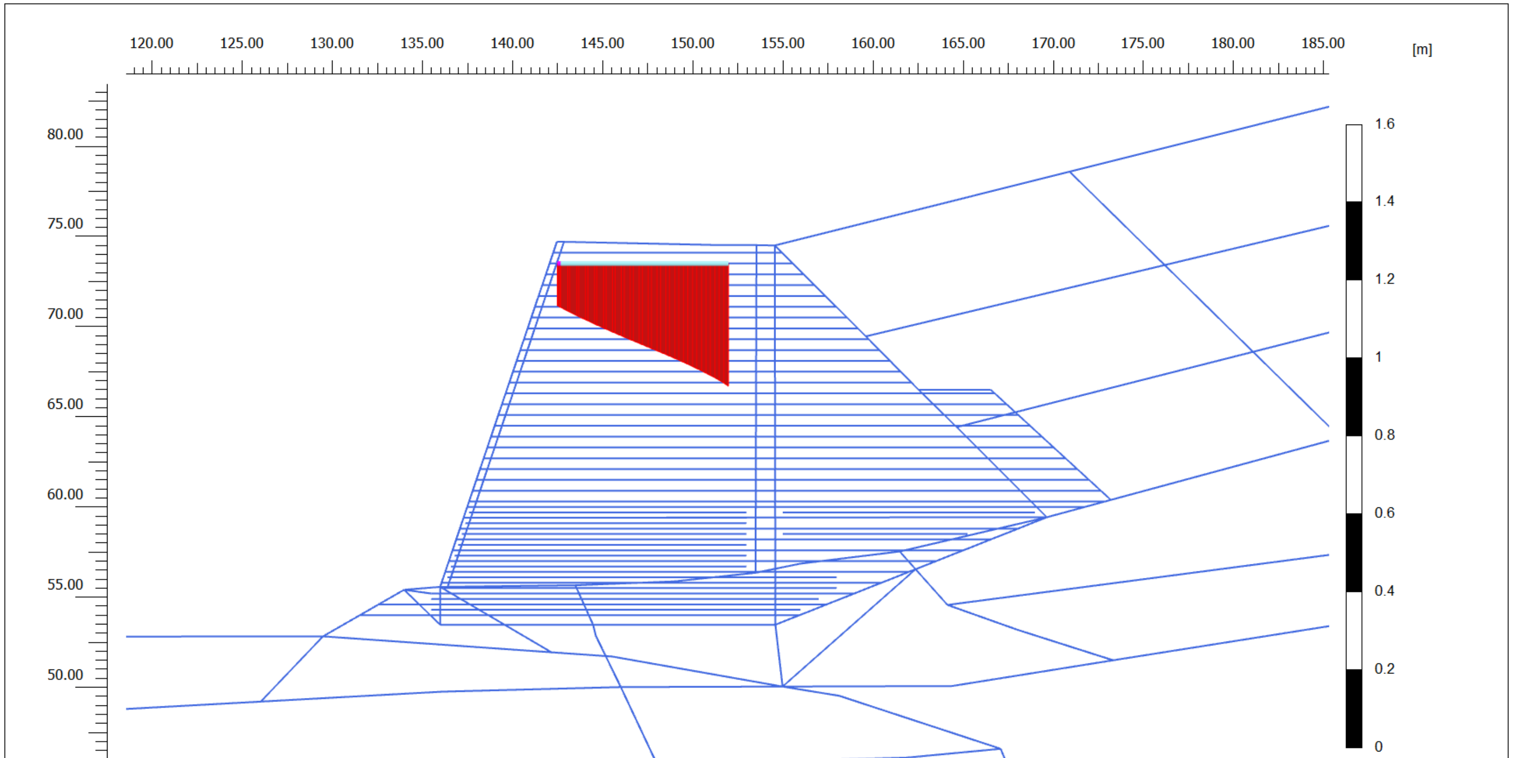
PLAXIS® 2D CONNECT Edition	*Project description* MSE Retaining Wall_Erskine Park Landfill_Section 3A			*Date* 21/04/2020
	Project filename 4. MSE_Section 3A_FullLandfill	*Step* 910	*Company* Golder Associates	

Section 3A – Predicted Total Settlement after 100 yrs

Cross Section View – Output from PLAXIS 2D

GOLDER	CLIENT Enviroguard Pty Ltd		PROJECT MSE Retaining Wall	A4
	DRAWN KT	DATE 22/04/2020	TITLE Preliminary Design Report	
	CHECKED LP	DATE 22/04/2020		
	SCALE N.T.S		PROJECT No 19135652 · CONTROL 19135652-006-R-RevA	FIFURE F.9

Output Version 20.2.0.83



Total displacements u_y (scaled up 20.0 times) (Time 39.53*10^3 day)

Maximum value = -0.1167 m

Minimum value = -0.3401 m

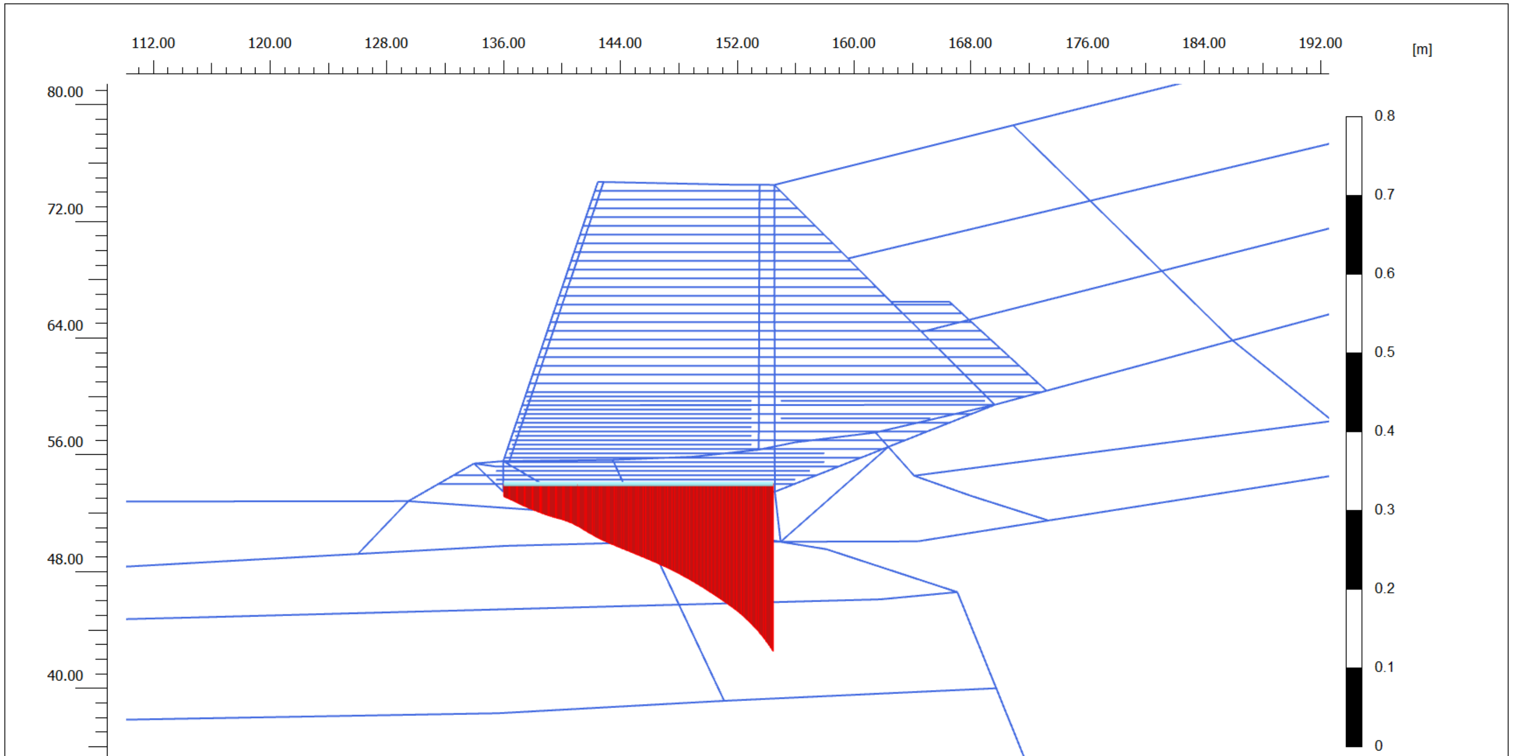
PLAXIS® 2D CONNECT Edition	Project description: MSE Retaining Wall_Erskine Park Landfill_Section 3A		Date: 21/04/2020
	Project filename: 4. MSE_Section 3A_FullLandfill	Step: 910	Company: Golder Associates

Section 3A – Predicted Vertical Settlement after 100 yrs at the Top of the Wall

Cross Section View – Output from PLAXIS 2D

GOLDER	CLIENT: Enviroguard Pty Ltd		PROJECT: MSE Retaining Wall	A4
	DRAWN: KT	DATE: 22/04/2020	TITLE: Preliminary Design Report	
	CHECKED: LP	DATE: 22/04/2020		
	SCALE: N.T.S		PROJECT No: 19135652 / CONTROL: 19135652-006-R-RevA	FIFURE: F.10

Output Version 20.2.0.83



Total displacements u_y (scaled up 50.0 times) (Time 39.53*10^3 day)

Maximum value = -0.01716 m

Minimum value = -0.2297 m

PLAXIS® 2D CONNECT Edition

Project description			Date
MSE Retaining Wall_Erskine Park Landfill_Section 3A			21/04/2020
Project filename	Step	Company	
4. MSE_Section 3A_FullLandfill	910	Golder Associates	

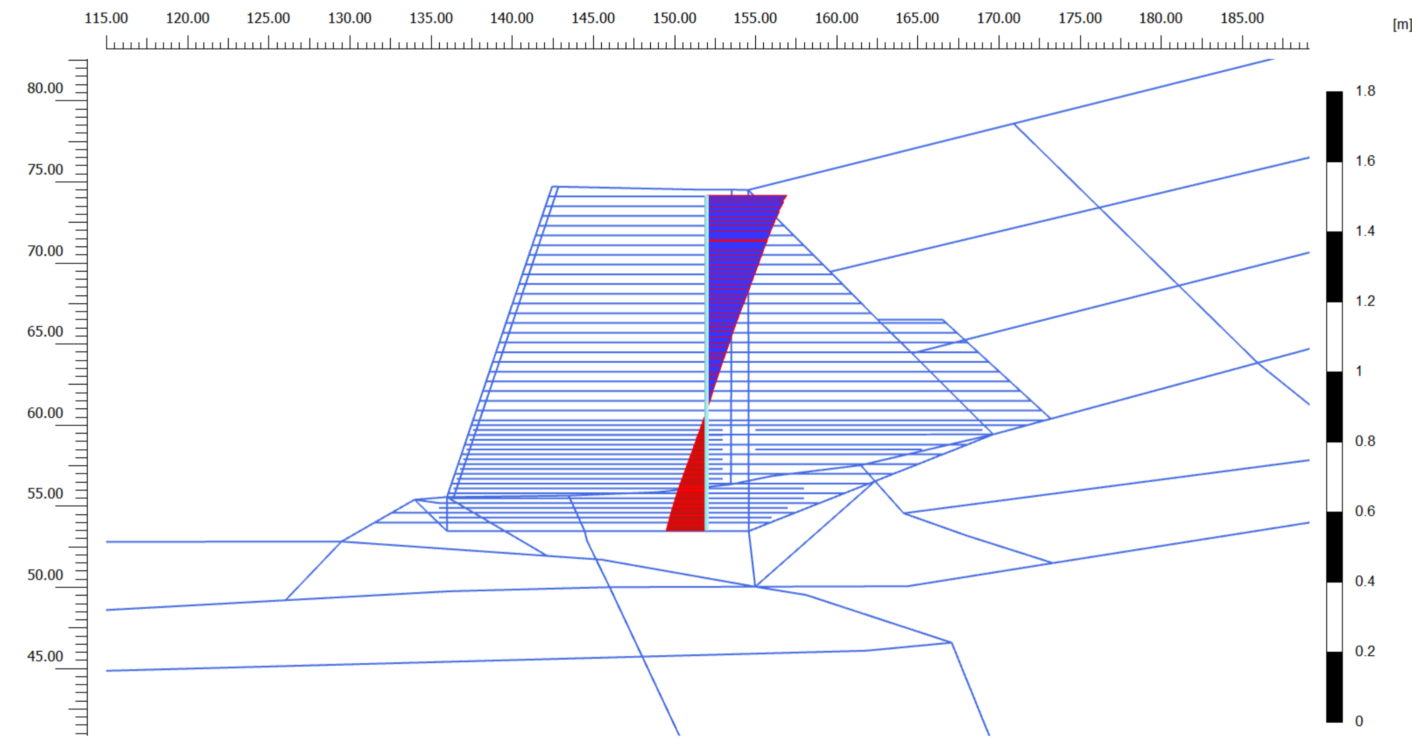
Section 3A – Predicted Vertical Settlement after 100 yrs at the Bottom of the Wall

Cross Section View – Output from PLAXIS 2D

GOLDER

CLIENT Enviroguard Pty Ltd		PROJECT MSE Retaining Wall	A4
DRAWN KT	DATE 22/04/2020	TITLE Preliminary Design Report	
CHECKED LP	DATE 22/04/2020		
SCALE N.T.S		PROJECT No 19135652	CONTROL 19135652-006-R-RevA	FIFURE F.11

Output Version 20.2.0.83



Total displacements u_x (scaled up 20.0 times) (Time 39.53×10^3 day)

Maximum value = 0.2477 m

Minimum value = -0.1245 m

PLAXIS® 2D CONNECT Edition

Project description	Date
MSE Retaining Wall_Erskine Park Landfill_Section 3A	21/04/2020

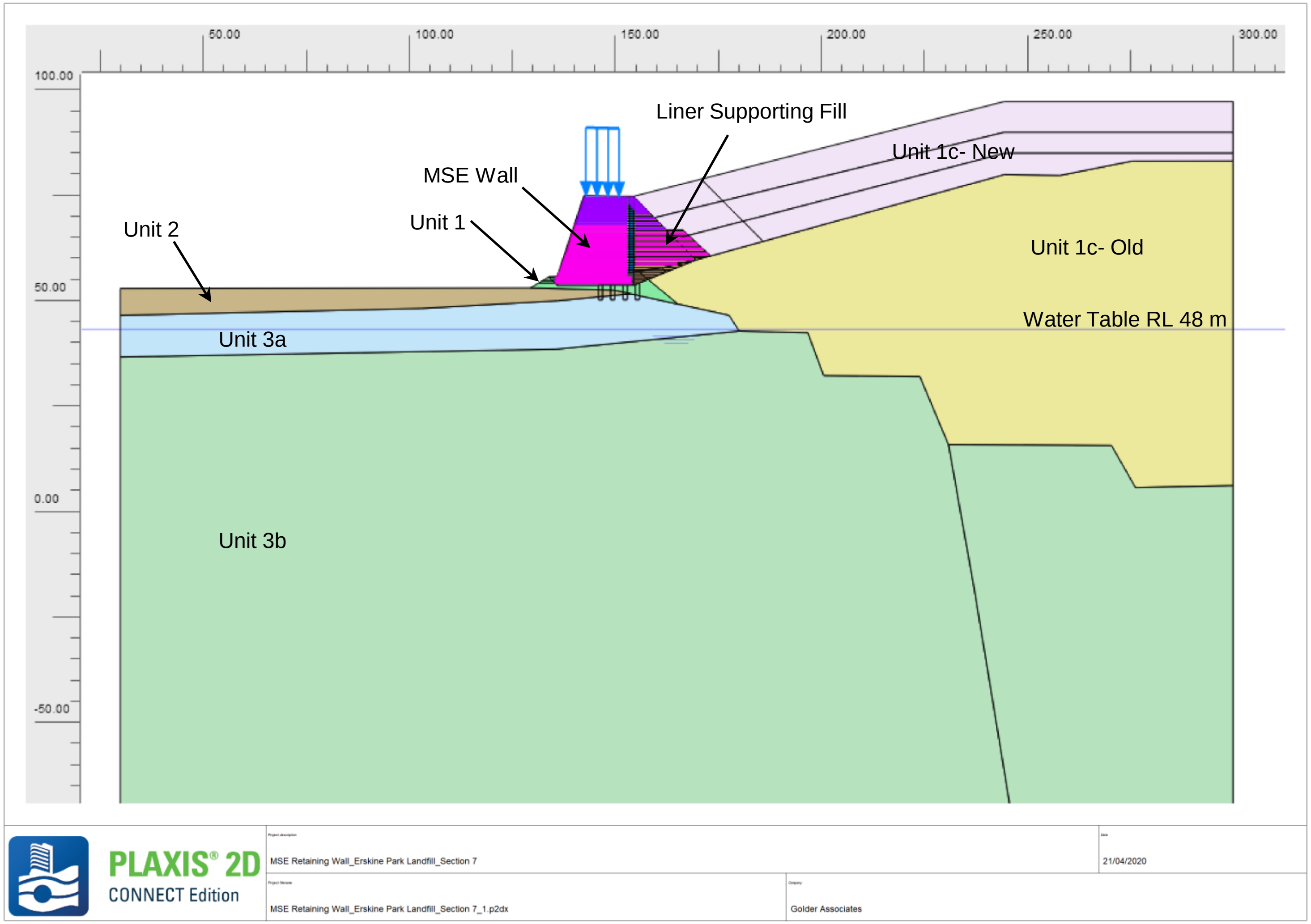
Project filename	Step	Company
4. MSE_Section 3A_FullLandfill	910	Golder Associates

Section 3A – Predicted Horizontal Movement after 100 yrs

Cross Section View – Output from PLAXIS 2D

GOLDER

CLIENT Enviroguard Pty Ltd		PROJECT MSE Retaining Wall		A4
DRAWN KT	DATE 22/04/2020	TITLE Preliminary Design Report		
CHECKED LP	DATE 22/04/2020			
SCALE N.T.S		PROJECT No 19135652	CONTROL 19135652-006-R-RevA	FIGURE F.12



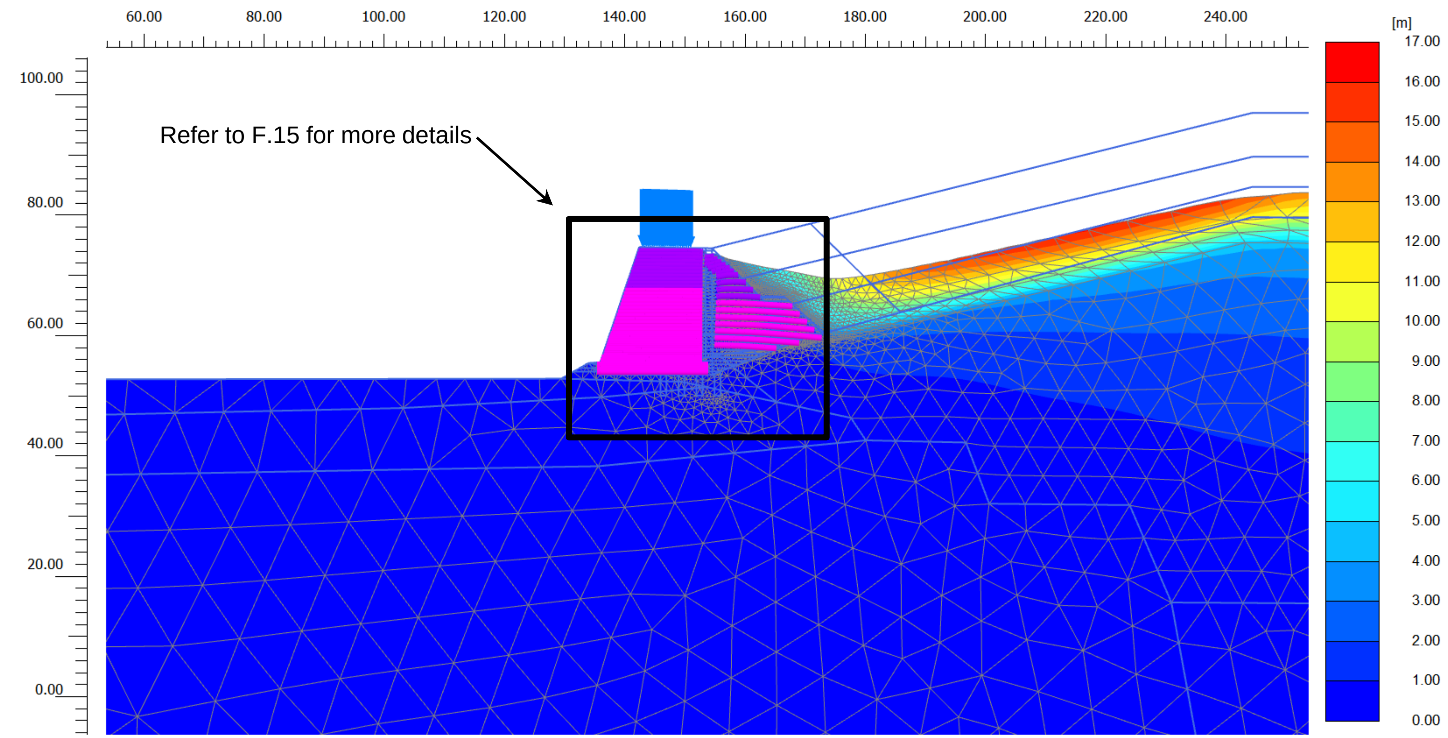
Section 7 – PLAXIS Model

Cross Section View – Output from PLAXIS 2D

GOLDER	CLIENT Enviroguard Pty Ltd		PROJECT MSE Retaining Wall	A4
	DRAWN KT	DATE 22/04/2020	TITLE Preliminary Design Report	
	CHECKED LP	DATE 22/04/2020		
	SCALE N.T.S		PROJECT No 19135652 / CONTROL 19135652-006-R-RevA	FIFURE F.13

Document Set ID: 9209113
Version: 1, Version Date: 09/07/2020

Output Version 20.2.0.83



Total displacements |u| (scaled up 0.500 times) (Time 39.53*10^3 day)

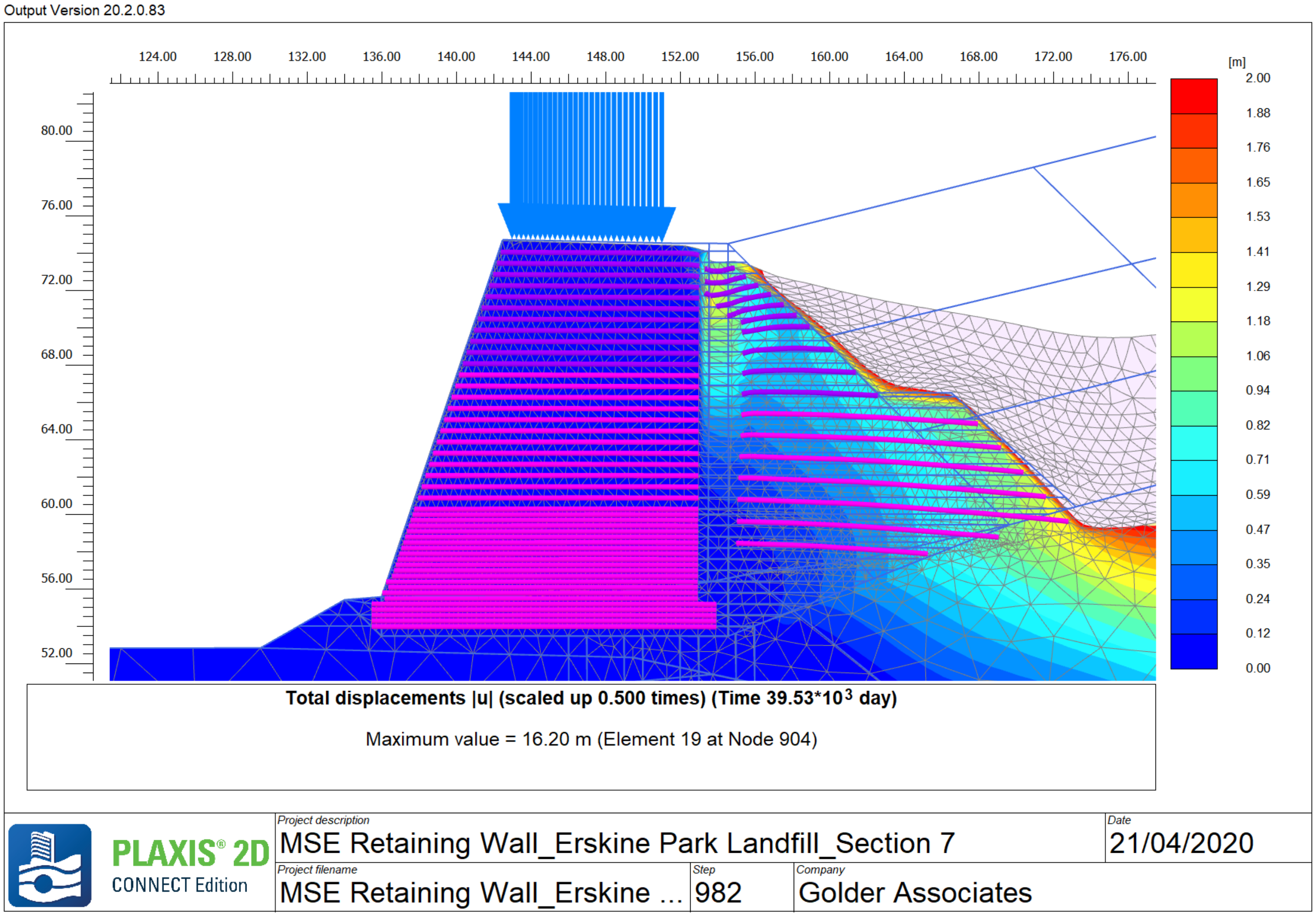
Maximum value = 16.20 m (Element 19 at Node 904)

PLAXIS® 2D CONNECT Edition	Project description: MSE Retaining Wall_Erskine Park Landfill_Section 7		Date: 21/04/2020
	Project filename: MSE Retaining Wall_Erskine ...	Step: 982	Company: Golder Associates

Section 7 – Predicted Total Settlement after 100 yrs

Cross Section View – Output from PLAXIS 2D

GOLDER	CLIENT: Enviroguard Pty Ltd		PROJECT: MSE Retaining Wall	A4
	DRAWN: KT	DATE: 22/04/2020	TITLE: Preliminary Design Report	
	CHECKED: LP	DATE: 22/04/2020		
	SCALE: N.T.S		PROJECT No: 19135652; CONTROL: 19135652-006-R-RevA	FIFURE: F.14



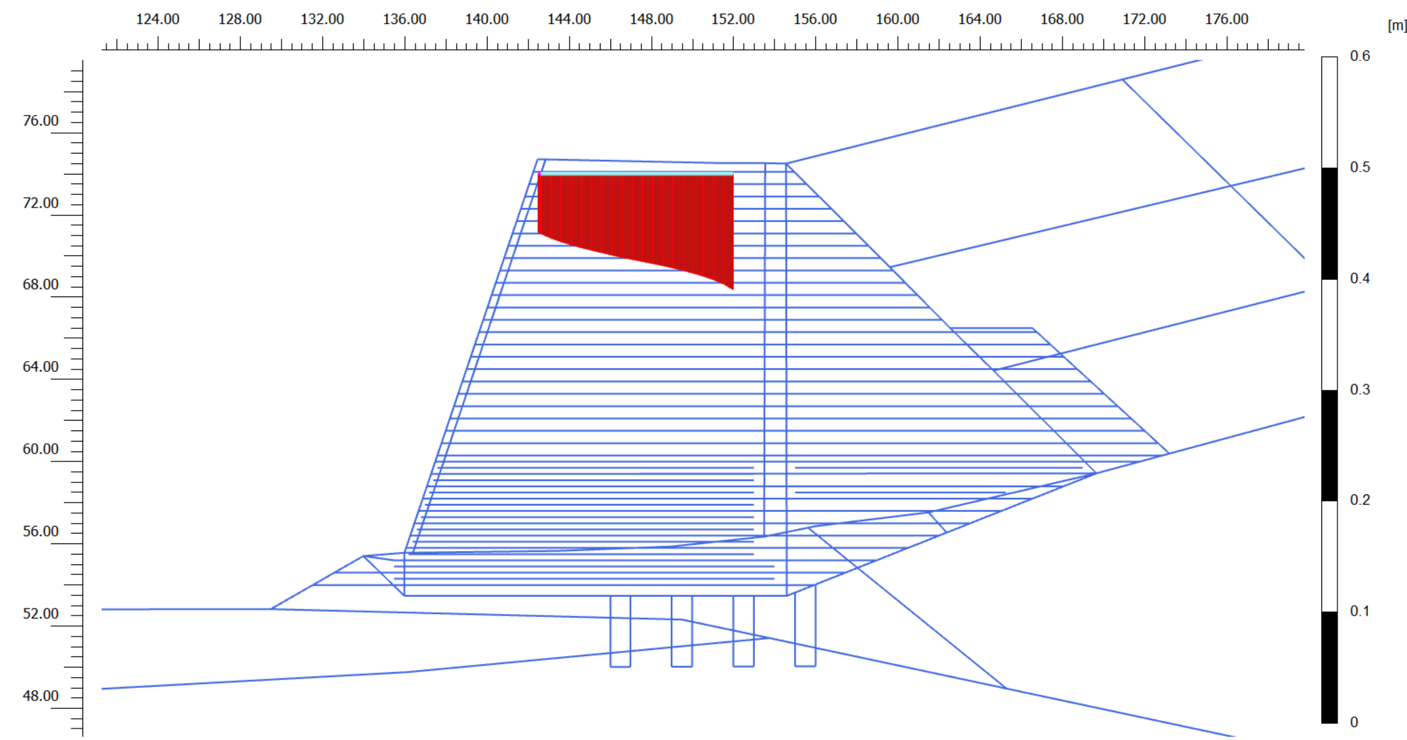
Section 7 – Predicted Total Settlement after 100 yrs

Cross Section View – Output from PLAXIS 2D

GOLDER

CLIENT Enviroguard Pty Ltd		PROJECT MSE Retaining Wall	A4
DRAWN KT	DATE 22/04/2020	TITLE Preliminary Design Report	
CHECKED LP	DATE 22/04/2020		
SCALE N.T.S		PROJECT No 19135652	CONTROL 19135652-006-R-RevA	FIFURE F.15

Output Version 20.2.0.83



Total displacements u_y (scaled up 50.0 times) (Time 39.53×10^3 day)

Maximum value = -0.05686 m

Minimum value = -0.1129 m

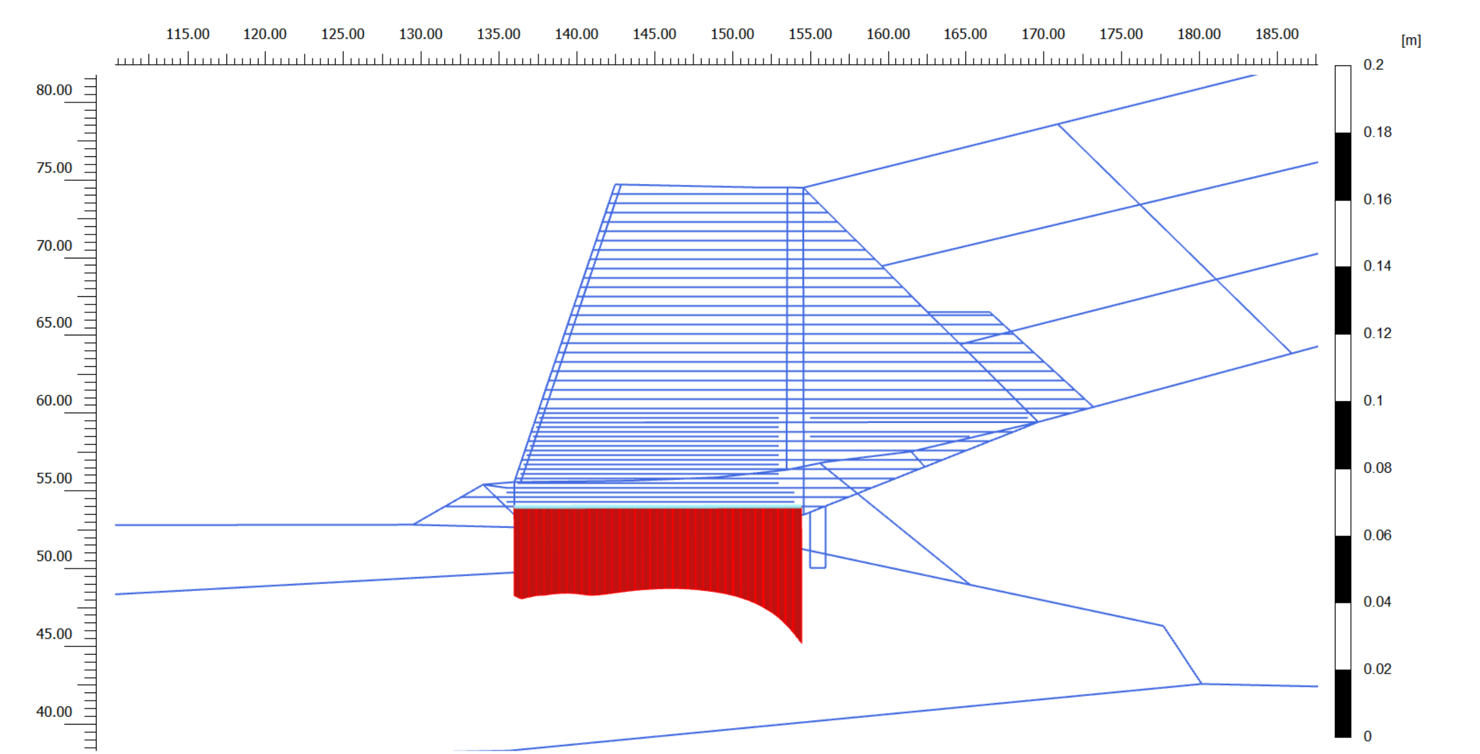
PLAXIS® 2D CONNECT Edition	Project description: MSE Retaining Wall_Erskine Park Landfill_Section 7		Date: 21/04/2020
	Project filename: MSE Retaining Wall_Erskine ...	Step: 982	Company: Golder Associates

Section 7 – Predicted Vertical Settlement after 100 yrs at the Top of the Wall

Cross Section View – Output from PLAXIS 2D

GOLDER	CLIENT: Enviroguard Pty Ltd		PROJECT: MSE Retaining Wall	A4
	DRAWN: KT	DATE: 22/04/2020	TITLE: Preliminary Design Report	
	CHECKED: LP	DATE: 22/04/2020		
	SCALE: N.T.S		PROJECT No: 19135652 / CONTROL: 19135652-006-R-RevA	FIGURE: F.16

Output Version 20.2.0.83



Total displacements u_y (scaled up 200 times) (Time $39.53*10^3$ day)

Maximum value = -0.02632 m

Minimum value = -0.04409 m

PLAXIS® 2D CONNECT Edition

Project description		Date
MSE Retaining Wall_Erskine Park Landfill_Section 7		21/04/2020

Project filename	Step	Company
MSE Retaining Wall_Erskine ...	982	Golder Associates

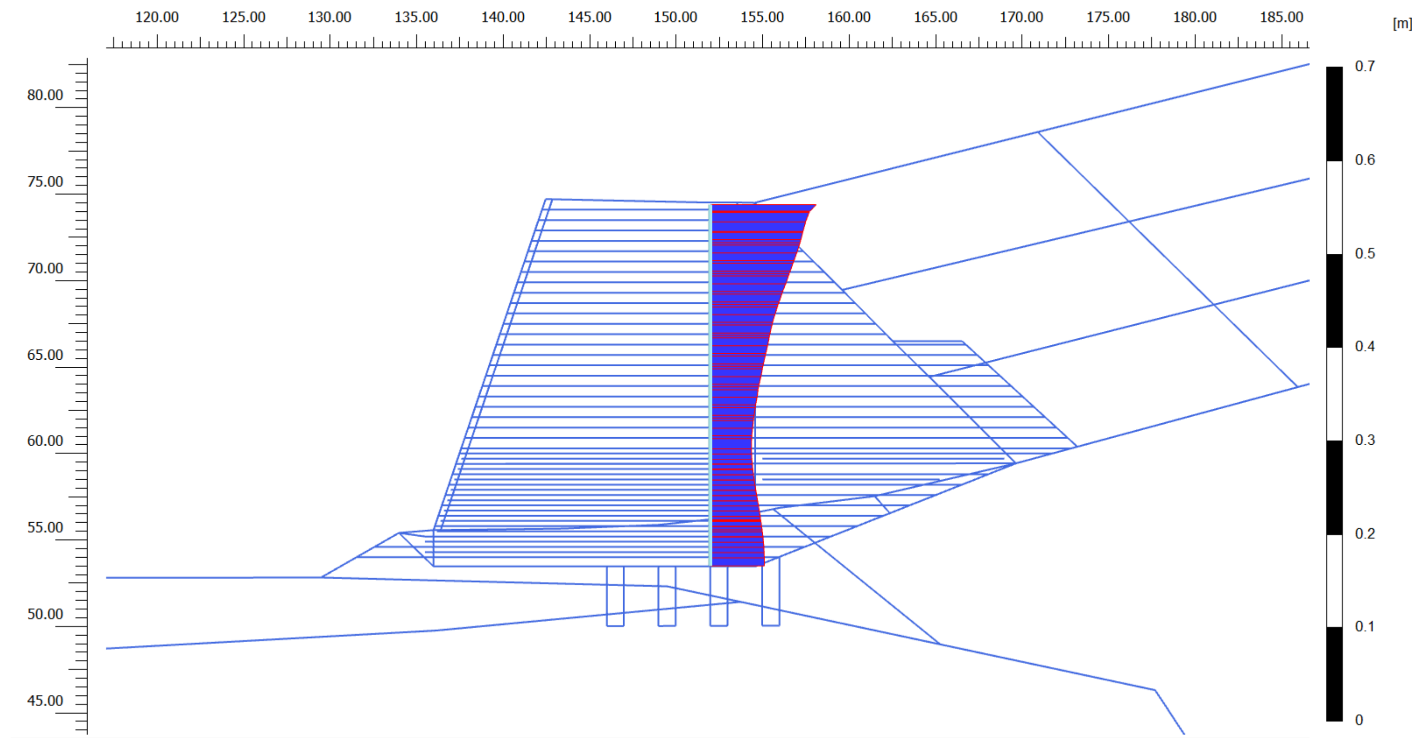
Section 7 – Predicted Vertical Settlement after 100 yrs at the Bottom of the Wall

Cross Section View – Output from PLAXIS 2D

GOLDER

CLIENT Enviroguard Pty Ltd		PROJECT MSE Retaining Wall	A4
DRAWN KT	DATE 22/04/2020	TITLE Preliminary Design Report	
CHECKED LP	DATE 22/04/2020		
SCALE N.T.S		PROJECT No 19135652 / CONTROL 19135652-006-R-RevA	FIFURE F.17

Output Version 20.2.0.83



Total displacements |u| (scaled up 50.0 times) (Time 39.53*10^3 day)

Maximum value = 0.1224 m

PLAXIS® 2D CONNECT Edition	Project description: MSE Retaining Wall_Erskine Park Landfill_Section 7			Date: 21/04/2020
	Project filename: MSE Retaining Wall_Erskine ...	Step: 982	Company: Golder Associates	

Section 7 – Predicted Horizontal Movement after 100 yrs

Cross Section View – Output from PLAXIS 2D

GOLDER	CLIENT: Enviroguard Pty Ltd		PROJECT: MSE Retaining Wall	A4
	DRAWN: KT	DATE: 22/04/2020	TITLE: Preliminary Design Report	
	CHECKED: LP	DATE: 22/04/2020		
	SCALE: N.T.S		PROJECT No: 19135652 / CONTROL: 19135652-006-R-RevA	FIFURE: F.18

APPENDIX G

Important Information

The document ("Report") to which this page is attached and which this page forms a part of, has been issued by Golder Associates Pty Ltd ("Golder") subject to the important limitations and other qualifications set out below.

This Report constitutes or is part of services ("Services") provided by Golder to its client ("Client") under and subject to a contract between Golder and its Client ("Contract"). The contents of this page are not intended to and do not alter Golder's obligations (including any limits on those obligations) to its Client under the Contract.

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This Report has been prepared in the context of the circumstances and purposes referred to in, or derived from, the Contract and Golder accepts no responsibility for use of the Report, in whole or in part, in any other context or circumstance or for any other purpose.

The scope of Golder's Services and the period of time they relate to are determined by the Contract and are subject to restrictions and limitations set out in the Contract. If a service or other work is not expressly referred to in this Report, do not assume that it has been provided or performed. If a matter is not addressed in this Report, do not assume that any determination has been made by Golder in regards to it.

At any location relevant to the Services conditions may exist which were not detected by Golder, in particular due to the specific scope of the investigation Golder has been engaged to undertake. Conditions can only be verified at the exact location of any tests undertaken. Variations in conditions may occur between tested locations and there may be conditions which have not been revealed by the investigation and which have not therefore been taken into account in this Report.

Golder accepts no responsibility for and makes no representation as to the accuracy or completeness of the information provided to it by or on behalf of the Client or sourced from any third party. Golder has assumed that such information is correct unless otherwise stated and no responsibility is accepted by Golder for incomplete or inaccurate data supplied by its Client or any other person for whom Golder is not responsible. Golder has not taken account of matters that may have existed when the Report was prepared but which were only later disclosed to Golder.

Having regard to the matters referred to in the previous paragraphs on this page in particular, carrying out the Services has allowed Golder to form no more than an opinion as to the actual conditions at any relevant location. That opinion is necessarily constrained by the extent of the information collected by Golder or otherwise made available to Golder. Further, the passage of time may affect the accuracy, applicability or usefulness of the opinions, assessments or other information in this Report. This Report is based upon the information and other circumstances that existed and were known to Golder when the Services were performed and this Report was prepared. Golder has not considered the effect of any possible future developments including physical changes to any relevant location or changes to any laws or regulations relevant to such location.

Where permitted by the Contract, Golder may have retained subconsultants affiliated with Golder to provide some or all of the Services. However, it is Golder which remains solely responsible for the Services and there is no legal recourse against any of Golder's affiliated companies or the employees, officers or directors of any of them.

By date, or revision, the Report supersedes any prior report or other document issued by Golder dealing with any matter that is addressed in the Report.

Any uncertainty as to the extent to which this Report can be used or relied upon in any respect should be referred to Golder for clarification

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