Atlantic Gold

Appendix A.2a

Mine Waste Stockpile Geotechnical Design - April 1, 2021 as Completed for the Updated 2021 Beaver Dam Mine EIS)

REPORT

Mine Waste Stockpile Geotechnical Design

Beaver Dam Mine

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Distribution List

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1.0 INTRODUCTION

Atlantic Mining NS Inc., a wholly owned subsidiary of St. Barbara Ltd. (Atlantic), has retained Golder Associates Ltd. (Golder) to provide geotechnical design of mine waste material stockpiles for the proposed Beaver Dam Mine Project (Beaver Dam site) located in Marinette, Nova Scotia.

The current mine plan proposes six material stockpiles on site to manage the following materials: non-acid generating waste rock (NAG), low grade ore (LG), potentially acid generating waste rock (PAG), till overburden, organic material, and topsoil. The topsoil stockpiles have been proposed on site to facilitate stripping and site preparation activities. Because of the small size and height of the topsoil stockpiles, their slope stability was not assessed in this report. Figure 1 provides a general arrangement plan of the proposed stockpile locations at the Beaver Dam site.

This report presents a summary of geotechnical subsurface conditions at the site, liquefaction analyses, slope stability analyses, stockpile hazard classifications, and geotechnical stockpile construction recommendations.

2.0 OBJECTIVE

The objective of the Geotechnical Stockpile Design Report is to provide geotechnical recommendations for the proposed stockpiles on site. The scope of the work presented in this report includes the following:

- Summary of subsurface conditions
- Seismic site classification and seismic hazard parameters
- Assessment of liquefaction potential
- Geotechnical design parameters for foundation and stockpiled materials
- Limit equilibrium slope stability analyses for static and seismic (pseudo-static) loading conditions
- General recommendations for site preparation and stockpile material placement

3.0 SUBSURFACE CONDITIONS

Borehole and test pit investigation locations in the stockpile areas are illustrated in plan on Figure 1. A summary of the geotechnical investigation, including Record of Borehole and Test Pit sheets, is presented in the *Preliminary Infrastructure Engineering Report, Beaver Dam Mine* (Golder, 2021). In general, the overburden across the site consists of a thin layer of organic topsoil over dense to very dense sand and gravel with silt and some cobbles and boulders over bedrock.

4.0 PROPOSED STOCKPILE LOCATIONS

Table 1 summarizes the proposed stockpiles locations.

Table 1: Stockpile General Locations

Stockpile	General Location Description
Non-Acid Generating Stockpile (NAG)	Located in the most Western extent of site, accessed by existing public roadways off Beaver Dam Road.
Low Grade Stockpile (LGS)	Located in the Western portion of site directly East in near proximity to the NAG stockpile, accessed by existing public roadways off Beaver Dam Road.
Topsoil Stockpiles (TSS)	Four small topsoil stockpiles are planned for the site. They are spaced across the site near areas requiring topsoil stripping.
Till Stockpiles (TLS)	Two till stockpiles are planned. They are both located East of the originally proposed crusher pad in the Central-East end of site.
Potential Acid Generating Stockpile (PAG)	Located in the North-Central section of site, directly North of the originally proposed crusher pad, accessed by Beaver Dam Road.
Organic Material Stockpile (OMS)	Located on the South-East section of site, accessed by public roads off Beaver Dam Road.

5.0 SEISMIC SITE CLASSIFICATION

The level of importance of seismic loading at any site is related to factors such as the subsoil conditions and their soil behaviour during an earthquake, the magnitude, duration, and frequency level of strong ground motion, and the probable intensity and likelihood of occurrence of an earthquake.

The *Canadian Foundation Engineering Manual* (CFEM, 2006) contains seismic analysis and design methodology. The seismic Site Class value, as defined in Table 6.1A (CFEM, 2006), depends on the average shear wave velocity and/or average standard penetration testing (SPT) N-values of the upper 30 m of soil and/or rock below founding level. The CFEM permits the Site Class to be specified based solely on the stratigraphy and in-situ testing data.

For the upper 30 m of soil and/or rock below founding level, average of SPT N-values is more than 50; and results of Vertical Seismic Profiling (VSP) performed at the location of Borehole BH2020-03B also suggest an average shear wave velocity of 413 m/s, which both suggest a Site Class C for seismic design analysis. Based on the in-situ testing data, this site can be assigned a Site Class of C for seismic design purposes.

Table 2 summarizes seismic parameters for the site, based on a 10% probability of exceedance in 50 years and a 2% probability of exceedance in 50 years from the NBCC (2015).

Table 2: 2015 National Building Code Seismic Hazard Calculation (NBCC, 2015)

Probability of Exceedance	PGA	Sa(0.05)	Sa(0.1)	Sa(0.2)	Sa(0.5)	Sa(1.0)	Sa(2.0)
10% in 50 years (475 AEP)	0.023 g	0.025 g	0.039 g	0.042 g	0.036 g	0.023 g	0.012 g
2% in 50 years (2,475 AEP)	0.061 g	0.075 g	0.105 g	0.105 g	0.079 g	0.051 g	0.028 g

Notes: AEP = annual exceedance probability
PGA = peak ground acceleration
g = acceleration due to gravity
Sa = spectral acceleration

6.0 GEOTECHNICAL DESIGN PARAMATERS

Geotechnical soil parameters were obtained from laboratory testing results following the geotechnical investigation (Golder, 2021) and from typical soil parameters based on previous project experience.

A summary of geotechnical parameters that were used in the effective stress slope stability analyses are summarised in Table 3. Effective stress parameters are considered appropriate to represent the geotechnical behaviour of the till overburden foundation, which is generally comprised of silt, sand, gravel and cobbles (i.e., non-cohesive granular material).

Table 3: Effective Stress Geotechnical Material Parameters Used in the Slope Stability Analyses

Material Type	Unit Weight (kN/m^3)	Cohesion (kPa)	Friction Angle (degrees)
Organics/Topsoil (In-Situ)	18	0	10
Organics (Stockpiled)	16	0	10
Till (In-Situ)	22	0	34
Till (Stockpiled)	21	0	34
Waste Rock	22	0	38
Bedrock	N/A	Impenetrable	Impenetrable

Stockpile slope stability was also checked using total stress parameters for the foundation till. Although the till can be generally described as a non-cohesive material (i.e., silt, sand, gravel and cobbles), there may be stockpile foundation areas with higher fines content (i.e., clayey or cohesive material) that are more appropriately modelled using total stress parameters. A summary of total stress parameters that were used for the till overburden foundation in the slope stability analyses are summarised in Table 4. Stability analyses were carried out modelling the till with a fixed undrained shear strength (S_u) and also using the SHANSEP method (Stress History and Normalized Soil Engineering Properties) to determine the minimum FOS.

Table 4: Total Stress Soil Parameters Used in the Slope Stability Analyses

Material Type	Unit Weight (kN/m³)	Undrained Shear Strength (kPa)	SHANSEP (Ladd and Foote, 1974)	
			Shear Strength Ratio (Tau/Sigma)	Minimum Shear Strength (kPa)
Till (In-Situ)	22	100	0.25	50 kPa

7.0 STOCKPILE DESIGN PARAMETERS

The total projected mine waste material quantities to be placed in each stockpile are summarized in Table 5.

Table 5: Life of Mine Material Quantities[1]

Material Type Life of Mine Material Quantities	Weight (Mt)	Volume (Mm³)
Organics	2.29	1.49
Topsoil	0.82	0.41
West Till Pile (1)	0.69	0.45
East Till Pile (2)	1.97	1.28
NAG	34.28	16.32
Low Grade Ore	2.48	1.17
PAG	2.50	1.19

Proposed stockpile maximum crest elevations and approximate height are summarised in Table 6.

Table 6: Proposed Stockpile Maximum Crest Elevations and Approximate Height[1]

Stockpile	Maximum Crest Elevation (m)	Approximate Height (m)
Organics	165	5
West Till	165	10-20
East Till	165	3-10
NAG	190	30-50
Low Grade (LG)	170	14-25
PAG	180	20

[1] Provided by Atlantic in MS Excel file titled "*Waste and Road Design Specifications (210128)*".

8.0 STOCKPILE DESIGN CRITERIA

8.1 Stability Analysis Factors of Safety

Based on the framework discussed in the *Guidelines for Mine Waste Dump and Stockpile Design* (Hawley and Cunning, 2017), a hazard (i.e., consequence of failure) and confidence level were assigned for each stockpile. The organics and till stockpiles were assessed as low hazard level based on overall fill slope angles less than 25 degrees, maximum stockpile height less than 50 meters, and no critical infrastructure present within the potential runout zone in the event of a slope failure. The waste rock stockpiles (i.e., PAG, NAG, and LG) were assessed as low to moderate hazard level based on the potential for moderate environmental impacts, in the event of a slope failure due to the presence of downstream lakes. Based on the available geotechnical investigation data and understanding of the stockpile foundation conditions, a moderate to high confidence level rating was assigned to all stockpiles. The assigned hazard and confidence levels are summarized in Table 7.

Table 7: Stockpile Hazard and Confidence Level

Stockpile	Hazard Level	Confidence Level
Organics	Low	Moderate to High
East Till	Low	Moderate to High
West Till	Low	Moderate to High
NAG	Low to Moderate	Moderate to High
Low Grade (LG)	Low to Moderate	Moderate to High
PAG	Low to Moderate	Moderate to High

Table 8 summarizes target and minimum factor of safety (FOS) values that were used for the stockpile design. The target FOS values (middle column of Table 8) are suggested design values from the *Mined Rock and Overburden Piles Investigation and Design Manual – Interim Guidelines* (BCMWRPRC, 1991). The minimum FOS values (third/right column of Table 8) are for a “Moderate” stability analysis rating based on the *Guidelines for Mine Waste Dump and Stockpile Design* by Hawley and Cunning (2017). The stockpile designs attempted to achieve the target FOS values (middle column of Table 8) but the minimum FOS values (third/right column of Table 8) are considered acceptable for stockpiles with a “Low” or “Moderate” Hazard Classification (discussed further in Sections 10 and 11 below). It should be noted that the minimum FOS values (third/right column of Table 8) assume that there is at least a moderate level of confidence in the input parameters, which is the case for this site, and that the stability analysis results are credible.

Table 8: Target and Minimum Factor of Safety (FOS) Values

Loading Condition	Target FOS Values (Case A - BCMWRPRC, 1991)	Minimum FOS Values (Moderate Stability Rating - Hawley and Cunning, 2017)
Dump/spoil surface short-term	1.0	-
Dump/spoil surface long-term	1.2	-
Overall global stability short-term (static)	1.3	-
Overall global stability long-term (static)	1.5	1.2
Pseudo-static (earthquake)	1.1	1.05

8.2 Design Earthquake

A design earthquake with a 2% probability of exceedance in 50 years (i.e., return period of 2,475 years) and peak ground acceleration (PGA) of 0.061 g (NBCC, 2015) was selected for design of the stockpiles.

9.0 LIQUEFACTION ASSESSMENT

Liquefaction is a phenomenon whereby seismically induced shaking generates shear stresses within the soil under undrained conditions. These stresses tend to densify the soil (i.e., leading to potentially large surface settlements) and under undrained conditions, generate excess pore pressures. The excess pore pressures can also lead to sudden temporary losses in strength. Where existing static shear stresses are present, the loss of strength can lead to significant lateral movements also referred to as “lateral spreading” or under certain conditions, even catastrophic failure of a slope also referred to as “flow slides”. Lateral spreading and flow slides often accompany liquefaction along rivers and other shorelines.

The liquefaction susceptibility of granular foundation soils was evaluated by comparing the penetration resistance required to trigger liquefaction with the available penetration resistance. Liquefaction is predicted to occur when the available penetration resistance is less than the resistance required. The susceptibility of the cohesive soils to cyclic mobility was also assessed.

The methodology used to assess liquefaction potential at the site is consistent with the approach outlined in Boulanger and Idriss (2014). It involves comparing the cyclic shear stresses applied to the soil by the design earthquake, represented as the cyclic stress ratio (CSR), to the cyclic shear strength, represented as the cyclic resistance ratio (CRR) provided by the soil.

Assessment of liquefaction susceptibility was carried out using the recommended procedure presented by Boulanger and Idriss (2014), which is a stress-based approach based on available geotechnical investigation data. The stress-based approach compares the earthquake induced cyclic stress with the cyclic strength of the foundation material. The earthquake-induced stresses and the cyclic resistance are normalized with respect to the vertical effective consolidation stress to obtain the induced CSR and the CRR. The factor of safety against liquefaction (FS_{Liq}) is calculated as follows:

$$FS_{Liq} = \frac{CRR}{\mathrm{CSR}}$$

If FS_{Liq} is less than 1, the foundation soils are considered to be susceptible to liquefaction.

The CRR of the foundation soil at each depth were calculated using the borehole SPT data collected as part of the investigation. The results of the liquefaction analyses indicate that the foundation soils at the site are not liquefiable during the 2,475-year design earthquake.

9.1 Earthquake-Induced Cyclic Stress Ratio

One-dimensional ground response analyses were carried out for the representative soil profiles at each stockpile to estimate the CSR. The input parameters for the ground response analyses were estimated using field shear wave velocity measurements at BH2020-03B and SPT data. Further details on the development of the spectrum-compatible input acceleration time histories, and the one-dimensional ground response analyses are included in the following sections.

The earthquake-induced CSR was estimated at a given depth using results of one-dimensional ground response analysis and the Seed and Idriss procedure, as described in Idriss and Boulanger (2008) and Boulanger and Idriss (2014).

$$CSR_{M,\ \sigma_v'} = 0.65\frac{\tau_{max}}{\sigma_v'}$$

Where τ_{max} is the maximum earthquake induced shear stress estimated from dynamic response analyses and σ_v' is vertical effective stress. CSR is calculated for earthquake moment magnitude of M and in-situ vertical effective stress (σ_v').

9.1.1 One-Dimensional Ground Response Analysis

One-dimensional ground response analyses were undertaken to assess the ground response at the site. Two stratigraphic profiles were selected for analysis that are representative of stockpile foundation conditions (i.e., borehole locations) with lowest SPT N-values (Table 9) and deepest overburden thickness (Table 10).

Based on the results of the field investigation, representative index properties and shear wave velocity variations of the overburden soil were developed for the two representative soil profiles and are summarized in the table below. The bedrock quality is variable across the site and includes fresh to highly weathered, medium bedded, weak to strong zones. As a result, Site Class C for soft rock (NBCC, 2015) was considered to be appropriate for this site, and an average shear wave velocity of 560 m/s was selected for the bedrock.

Table 9: Summary of Representative Stratigraphy and Material Properties for Profile TLS-BH20-03 (Vs values correlated from SPT N-values)

Soil Unit	g (kN/m^3)	Depth (m)	Vs (m/s)
TOPSOIL (OH) ORGANIC SILT	17	0 – 0.4	228
(ML) CLAYEY SILT	18	0.4 – 2.3	216 - 222
(CL) gravelly SILTY CLAY	19	2.3 – 9.6	244 - 378
Bedrock	23	> 9.6	560

Table 10: Summary of Representative Stratigraphy and Material Properties for Profile NAG- BH20-07 (Vs values correlated from SPT N-values)

Soil Unit	g (kN/m^3)	Depth (m)	Vs (m/s)
TOPSOIL (CL) SILTY CLAY	17	0 – 0.6	222
(CL) SILTY CLAY	18	0.6 – 2.9	278 - 302
(ML) gravelly sandy CLAYEY SILT	19	2.9 – 8.9	334 - 368
(CL) SILTY CLAY	18	8.9 – 9.6	356
(CL) gravelly SILTY CLAY	19	9.6 – 12.6	305 - 342
(CL) SILTY CLAY	18	12.6 – 13.3	305
Bedrock	23	> 13.3	560

Where required for analysis, the small-strain shear modulus (G_{max}) for the site soils were estimated using the site-specific shear wave velocity (V_s) measurements obtained from the results of the VSP testing or correlated from SPT N-values. The values of G_{max} and V_s are related through the following expression:

$$G_{max} = \rho\ (V_s)^2, where\ \rho = material\ density$$

9.1.1.1 Target Spectrum

In accordance with NBCC (2015) seismic hazard data for the site and underlying soft bedrock at depth, the Site Class C seismic hazard values for the 2% probability of exceedance in the 50-year design earthquake event given in Section 5.0 were used as the target spectrum for the input ground motions.

9.1.1.2 Spectrum-Compatible Earthquake Time Histories

To develop time histories compatible with the target firm-ground spectrum, a hazard de-aggregation was first carried out to identify the primary contributors of earthquake magnitude and hypocentral distance for the 2,475-year design earthquake event. A suite of representative seed time histories that matched the primary contributors were selected for each design earthquake. The time histories were then linearly scaled to match the Site Class C target spectra to represent the site-specific design firm-ground accelerations, for use in the site-specific ground response analyses. Time histories were obtained from either the Engineering Seismology Toolbox (EST) or the Pacific Earthquake Engineering Research (PEER) databases.

A summary of the earthquake records used in the site-specific ground response analyses for each design earthquake are provided in the table below. The earthquake mean magnitudes and hypocentral distances are also provided for reference.

Table 11: Summary of Input Time History Earthquake Events – 2,475-Year Design Earthquake

Database	Event Name	Event Year	Station / Suite Name	Mag.	Dist. (km)	Scaling Method
EST	Motion # 31	-	East6c2 Suite	6.0	26	Linear Scaling
EST	Motion # 7	-	East7a2 Suite	7.0	45	Linear Scaling
EST	Motion # 11	-	East7c2 Suite	7.0	50	Linear Scaling
EST	Motion # 16	-	East7c2 Suite	7.0	63	Linear Scaling
EST	Motion # 30	-	East7c2 Suite	7.0	48	Linear Scaling
EST	Motion # 35	-	East7c2 Suite	7.0	100	Linear Scaling
EST	Motion # 36	-	East7c2 Suite	7.0	100	Linear Scaling
EST	Motion # 37	-	East7a2 Suite	7.0	96	Linear Scaling
EST	Motion # 41	-	East7a2 Suite	7.0	94	Linear Scaling
EST	Motion # 44	-	East7a2 Suite	7.0	99	Linear Scaling
PEER	Sparks	2011	Sparks	5.7	60	Linear Scaling

9.1.1.3 SHAKE Analysis

The one-dimensional soil columns and soil parameters described above were used for the ground response analyses. For all soil columns, the input motions established for the site were applied at the top of the bedrock as outcropping motions to account for the overburden effects. All ground response analyses were carried out using the software Shake2000 (Version 10.1.1, November 2018, part of the Professional Suite of ground response software by GeoMotions, LLC).

The shear modulus reduction and damping versus shear strain curves used for the main soil strata are as follows:

- Clayey Silt: Vucetic and Dobry (1991) for Plasticity Index (Ip) = 0%
- Silty Clay: Vucetic and Dobry (1991) for Plasticity Index (Ip) = 15%
- Bedrock: EPRI, 1993

The ground response (SHAKE) analysis results were an input to calculate CSR values with depth and used for the liquefaction assessment described below.

9.2 Cyclic Resistance Ratio

The CRR of non-plastic soils is generally obtained with semi-empirical relationships developed from in-situ testing compiled from case histories where liquefaction has or has not been observed. Idriss and Boulanger (2008) and Boulanger and Idriss (2014) provide details of the procedure to estimate the CRR of non-plastic soils using SPT data, which is formulated as follows:

$$CRR_{M=7.5,\sigma'_{vc}=1\mathrm{atm}} = exp\left(\frac{(N_1)_{60cs}}{14.1} + \left(\frac{(N_1)_{60cs}}{126}\right)^2 - \left(\frac{(N_1)_{60cs}}{23.6}\right)^3 + \left(\frac{(N_1)_{60cs}}{25.4}\right)^4 - 2.8\right)$$

Where $CRR_{M=7.5,\sigma'_{vc}=1\mathrm{atm}}$ is the cyclic resistance of the soil subjected to a magnitude M7.5 earthquake, and normalized to vertical effective stress, $\sigma'_{vc} = 1\mathrm{atm}$; $(N_1)_{60cs}$ is the penetration resistance corrected for SPT hammer efficiency, overburden pressure, and soil fines content.

The correction for fines content is based on Idriss and Boulanger (2008) using average fines content measurements from laboratory testing of samples collected during field investigation.

The CRR can be extended to other values of earthquake magnitude and effective overburden stress by using correction factors to adjust for the site characteristics:

$$CRR_{M,\sigma'_{vc}} = CRR_{M=7.5,\sigma'_{vc}=1} \cdot MSF \cdot K_\sigma$$

Where $CRR_{M,\sigma'_{vc}}$ is the cyclic resistance ratio at the specific values of earthquake magnitude M and overburden effective stress σ'_{vc}. MSF is the magnitude scaling factor and K_σ is the overburden correction factor. Values for these factors are presented in Idriss and Boulanger (2008) and Boulanger and Idriss (2014).

CRR values calculated in accordance with the above method were used for the liquefaction assessment described below.

9.3 Results of Liquefaction Susceptibility Assessment

Liquefaction susceptibility assessment results for the foundation materials are presented in Appendix B. The liquefaction susceptibility of the two representative soil profiles was assessed by comparing earthquake induced CSR and CRR values to calculate factor of safety against liquefaction (FSL) with depth.

The liquefaction assessment indicates that the stockpile foundation soils at the site are not expected to liquefy following the 2,475-year return period design earthquake event.

10.0 SLOPE STABILITY ANALYSIS

Slope stability analyses were completed for each stockpile using the program SLOPE/W™ Ver. 2019, which is a two-dimensional limit equilibrium computer software program developed by Geo-Slope International Ltd. The Morgenstern-Price method of slices was employed to analyse potential failure surfaces through the stockpile slopes and underlying foundations. The analyses were conducted to locate the most critical failure surfaces, resulting in the most conservative FOS. Slope stability analyses were conducted using both effective and total stress analysis parameters. Slope stability analysis results for each stockpile are included in Appendix C.

Post-earthquake analyses (i.e., using residual shear strengths for liquefied foundation materials) were not carried out for any of the stockpiles because none of the foundation soils were determined to be susceptible to liquefaction under the design earthquake (as outlined in Section 9). Pseudo-static analyses were carried out for all stockpiles because the foundation materials are not expected to experience liquefaction. The pseudo-static analyses were carried out in accordance with the method proposed by Hynes-Griffin and Franklin (1984). In this method, a horizontal acceleration coefficient of 0.0305 g (equal to half of the bedrock PGA) is applied.

Table 12 summarizes the results of slope stability analyses for each stockpile. All values meet the minimum FOS values (outlined in Table 8 above) for a “Moderate” slope stability rating in accordance with the *Guidelines for Mine Waste Dump and Stockpile Design* by Hawley and Cunning (2017). The calculated FOS values are considered sufficient to accommodate some variability in foundation conditions and material properties (i.e., moderate confidence level).

Slope stability analysis of the organics stockpile was initially checked for the proposed 7H:1V slope, which calculated a FOS below 1.0 (i.e., a 7H:1V organics slope would not meet the design criteria). However, stability analyses determined that the organics stockpile slope could achieve the required FOS (see Table 12) with a 10 m wide zone of till on the exterior slope and be steepened to 3H:1V (see Figure C-7 in Appendix C).

Slope stability analysis of the West Till (1) stockpile was initially checked for the proposed 3H:1V overall slope (e.g., 7 m high inter-bench slopes and 21 m wide benches) which calculated acceptable FOS values (outlined in Table 8). The West Till (1) stockpile slopes were then optimized by checking stability with 9 m high inter-bench slopes and 16 m wide benches (as illustrated in Cross-Section A on Figure 2). The revised West Till (1) stability analysis results calculated acceptable FOS values (as summarized in Table 12 and presented in Appendix C). The updated West Till (1) stockpile stability analyses indicate that bench widths for the East Till (2) stockpile can also be reduced from 21 m to 16 m (as illustrated in Cross-Section B on Figure 2).

Slope stability of the NAG stockpile north and south slopes was checked with bench geometry that achieved an overall 3H:1V slope. The NAG stockpile slopes were analysed with 10 m high inter-bench slopes at 1.5H:1V and 21 m wide benches (as illustrated in Cross-Sections E and F on Figure 3). These bench dimensions and overall slopes for the NAG stockpiles calculated acceptable minimum FOS values (as summarized in Table 12 and presented in Appendix C).

Slope stability analyses indicate that a 3H:1V overall slope for the other mine waste stockpiles will meet minimum factor of safety requirements (as outlined in Table 12 and presented in Appendix C). Recommended slope configurations (e.g., bench heights, inter-bench slopes, and overall slopes) for each stockpile are summarized in Table 13 and illustrated in cross-section on Figures 2 and 3. The north slope of the NAG stockpile has the lowest FOS values (e.g., static FOS = 1.35 and pseudo-static FOS = 1.19), which meet the minimum FOS values for a “Moderate” stability rating based on the *Guidelines for Mine Waste Dump and Stockpile Design* by Hawley and Cunning (2017).

Table 12: Stockpile Slope Stability Analysis Results

Stockpile	Minimum Static FOS	Calculated Static FOS	Minimum Pseudo-Static FOS	Calculated Pseudo-Static FOS
Organics	1.20	1.85	1.05	1.64
West Till (1)	1.20	1.74	1.05	1.58
East Till (2)	1.20	1.80	1.05	1.62
NAG (South Slope)	1.20	1.49	1.05	1.31
NAG (North Slope)	1.20	1.35	1.05	1.19
LG	1.20	1.94	1.05	1.73
PAG	1.20	1.61	1.05	1.44

11.0 STOCKPILE HAZARD CLASSIFICATION

Waste dump and stockpile stability rating and hazard classification (WSRHC) assessments were carried out for the proposed NAG, PAG, LG, West Till (1), East Till (2), and Organics stockpiles in accordance with the *Guidelines for Mine Waste Dump and Stockpile Design* by Hawley and Cunning (2017). All stockpiles were assessed as waste dump and stockpile hazard classification (WHC) III Moderate Hazard, except for the LG stockpile, which was assessed as WHC II Low Hazard (just above the WHC III Moderate Hazard line). Appendix D presents the stockpile hazard classification assessments.

12.0 GROUND PREPARATION AND STOCKPILE DEVELOPMENT

12.1 Ground Preparation and Initial Lift Placement

Recommendations for ground preparation and stockpile development are summarized in Table 13. Cross-sections of each stockpile illustrating the recommended topsoil stripping width are shown on Figure 2 and Figure 3. Topsoil should be stripped from the specified width within the perimeter of the stockpile footprints prior to placing the initial lift of waste, to improve slope stability and prevent shear failures through the weak organic topsoil layer. The initial lift of waste placement should be limited to 2 m in height to confirm foundation stability and should extend across the entire stockpile footprint prior to placing the next vertical lift above.

Table 13: Recommendations for Ground Preparation and Stockpile Development

Stockpile	Topsoil Stripping Width	Inter-bench Slope (H:V)	Steepest Overall Slope (H:V)	Maximum Vertical Bench Height (m)	Minimum Bench Width (m)	Development Recommendations
Organics	10 m	N/A	3:1	N/A	N/A	10 m wide till exterior slope required for stability
West Till (1)	45 m	1.5:1	2.4:1	9	16	At least one mid-slope bench
East Till (2)	40 m	1.5:1	2.6:1	7	16	At least one mid-slope bench
NAG	100 m wide (South slope) 160 m wide (North slope)	1.5:1	3:1	10	21	Topsoil stripping width = ultimate stockpile height x 3.2 = 100 to 160 m wide
LG	40 m	1.5:1	3:1	10	21	At least one mid-slope bench
PAG	70 m	1.5:1	3:1	7	21	At least one mid-slope bench

12.2 Surface Water Management

A surface water management plan should be developed for all stockpile areas that ties into the site-wide water management plan. Surface water management should include upstream diversions to prevent run-on to the stockpiles and downstream water collection systems. Surface water management and/or sediment control measures should be implemented prior to beginning stockpile ground preparation and waste placement.

12.3 Stockpile Dumping Operations

The stockpiles should be developed from the bottom up, in 2 to 3 m thick lifts to achieve the overall slopes summarized in Table 13. Each lift shall extend across the entire stockpile footprint before starting the next lift. Figures 2 and 3 illustrate cross-sections and typical bench dimensions for each stockpile slope. Bench heights should be reduced, where required, to ensure that the specified overall (i.e., crest to toe) stockpile slope is maintained. Vertical bench heights should be limited to 5 m where the total stockpile height is 10 m or less (e.g., East Till stockpile). Vertical bench heights should be limited to 7 m where the total stockpile height is between 10 and 25 m (e.g., East Till and PAG stockpiles), except at the West Till stockpile where the vertical bench height can be up to 9 m. Stockpiles with an ultimate height greater than 25 m can be constructed with 10 m vertical bench heights. All stockpiles, other than the organics stockpile, shall have at least one mid-slope bench.

Waste materials should be dumped well away from the bench crest edge and pushed with a bulldozer to achieve the recommended bench dimensions and slopes. Safety berms should be maintained on all dump crests and haul roads of sufficient height, to prevent the largest mine equipment from inadvertently driving over the crest. The height of the safety berms should be no less than half the height of the largest haul truck tire.

Safety berms should not be used as a wheel stop when backing up to dump. Haul trucks should dump short of the crest and the dumped materials pushed over the crest with a dozer. Some of the dumped material should be retained on the crest for ongoing safety berm construction.

The condition of the dump platform must be monitored visually for any signs of instability. The dozer operator responsible for spreading dumped waste materials should ensure that the surface of the dump and dump platform is maintained in good condition. The dump platform should be maintained with an uphill grade to the crest. A grade of not less than 2% should be maintained to facilitate surface water drainage away from the crest edge.

The dumping sequence should consider haul road configuration and stockpile foundation conditions. In addition, foundation conditions may require that waste materials be placed preferentially in particular areas to achieve adequate slope stability.

Stockpile stability is influenced by many factors, including dump height, dump materials, dump geometry, climatic conditions, foundation materials, and surface and groundwater conditions. However, the rate of crest edge (horizontal) and stockpile height (vertical) advancement have a significant influence on slope stability. Maximum rates of horizontal and vertical advancement should be defined based on available site-specific foundation conditions, design information, and dump operational experience.

12.4 Stockpile Visual Monitoring

Regularly scheduled inspections and monitoring of the stockpiles is critical to early detection of concerns relating to physical stability. The visual inspection program should include informal observations by operations staff, formal monthly inspections by a site engineer, and annual external visual inspections by a qualified geotechnical engineer. Visual inspection of ramps and haul roads near dump crests or slopes should be carried out on a frequent basis during stockpile development operations. Haul truck operators, dozer operators, and any others who routinely visit the dumps should be trained in the recognition of hazards and reporting procedures. Operations staff, equipment operators, surveyors, and other personnel that regularly visit the waste dumps should be trained to recognise the following potential indications of instability:

- excessive or abnormal cracking
- excessive crest deformation or settlement
- excessive over-steepening of the crest
- abnormal platform tilting
- seepage breakout on the face
- bulging of the face
- toe spreading

Observations of any of these indicators should be evaluated to determine if there is a developing slope instability issue.

12.5 Geotechnical Monitoring Instrumentation

Monitoring of the physical performance of stockpiles is recommended to confirm that performance is consistent with design assumptions. The monitoring program should consider potential failure mechanisms. Foundation instability is the primary potential mechanism of stockpile failure. Consideration should be given to the installation of vibrating wire piezometers in clayey foundation materials that may be susceptible to excess pore water pressure generation during loading (i.e., fill placement). In addition, installation of slope inclinometers could be considered to monitor slope and foundation deformation. A trigger action response plan (TARP) should be established for the piezometers and slope inclinometers.

12.6 Operational Guidelines

The operation of a waste dump or stockpile must be consistent with the design basis and assumptions. Operational guidelines or standard operating procedures should be developed using the design basis and reviewed by the design engineer.

13.0 IMPORTANT INFORMATION AND LIMITATIONS OF THIS REPORT

Standard of Care: Golder Associates Ltd. (Golder) has prepared this report in a manner consistent with that level of care and skill ordinarily exercised by members of the engineering and science professions currently practicing under similar conditions in the jurisdiction in which the services are provided, subject to the time limits and physical constraints applicable to this report. No other warranty expressed or implied is made.

Basis and Use of the Report: This report has been prepared for the specific site, design objective, development, and purpose described to Golder by the Client: Atlantic Mining NS Inc. The factual data, interpretations, and recommendations pertain to a specific project, as described in this report and are not applicable to any other project or site location. Any change of site conditions, purpose, development plans or if the project is not initiated within eighteen months of the date of the report may alter the validity of the report. Golder cannot be responsible for use of this report, or portions thereof, unless Golder is requested to review and, if necessary, revise the report.

The information, recommendations, and opinions expressed in this report are for the sole benefit of the Client. No other party may use or rely on this report or any portion thereof without Golder's express written consent. If the report was prepared to be included for a specific permit application process, then the client may authorize the use of this report for such purpose by the regulatory agency as an Approved User for the specific and identified purpose of the applicable permit review process, provided this report is not noted to be a draft or preliminary report, and is specifically relevant to the project for which the application is being made. Any other use of this report by others is prohibited and is without responsibility to Golder. The report, all plans, data, drawings, and other documents, as well as all electronic media prepared by Golder are considered its professional work product and shall remain the copyright property of Golder, who authorizes only the Client and Approved Users to make copies of the report, but only in such quantities as are reasonably necessary for the use of the report by those parties. The Client and Approved Users may not give, lend, sell, or otherwise make available the report or any portion thereof to any other party without the express written permission of Golder. The Client acknowledges that electronic media is susceptible to unauthorized modification, deterioration, and incompatibility and therefore the Client cannot rely upon the electronic media versions of Golder's report or other work products.

The report is of a summary nature and is not intended to stand alone without reference to the instructions given to Golder by the Client, communications between Golder and the Client, and to any other reports prepared by Golder for the Client relative to the specific site described in the report. In order to properly understand the suggestions, recommendations, and opinions expressed in this report, reference must be made to the whole of the report. Golder cannot be responsible for use of portions of the report without reference to the entire report.

Unless otherwise stated, the suggestions, recommendations, and opinions given in this report are intended only for the guidance of the Client in the design of the specific project. The extent and detail of investigations, including the number of test holes, necessary to determine all of the relevant conditions which may affect construction costs would normally be greater than has been carried out for design purposes. Contractors bidding on, or undertaking the work, should rely on their own investigations, as well as their own interpretations of the factual data presented in the report, as to how subsurface conditions may affect their work, including but not limited to proposed construction techniques, schedule, safety, and equipment capabilities.

Soil, Rock and Groundwater Conditions: Classification and identification of soils, rocks, and geologic units have been based on commonly accepted methods employed in the practice of geotechnical engineering and related disciplines. Classification and identification of the type and condition of these materials or units involves judgment, and boundaries between different soil, rock or geologic types or units may be transitional, rather than abrupt. Accordingly, Golder does not warrant or guarantee the exactness of the descriptions.

Special risks occur whenever engineering or related disciplines are applied to identify subsurface conditions and even a comprehensive investigation, sampling, and testing program may fail to detect all or certain subsurface conditions. The environmental, geologic, geotechnical, geochemical, and hydrogeologic conditions that Golder interprets to exist between and beyond sampling points may differ from those that actually exist. In addition to soil variability, fill of variable physical and chemical composition can be present over portions of the site or on adjacent properties. **The professional services retained for this project include only the geotechnical aspects of the subsurface conditions at the site, unless otherwise specifically stated and identified in the report.** The presence or implication(s) of possible surface and/or subsurface contamination resulting from previous activities or uses of the site and/or resulting from the introduction onto the site of materials from off-site sources are outside the terms of reference for this project and have not been investigated or addressed.

Soil and groundwater conditions shown in the factual data and described in the report are the observed conditions at the time of their determination or measurement. Unless otherwise noted, those conditions form the basis of the recommendations in the report. Groundwater conditions may vary between and beyond reported locations and can be affected by annual, seasonal, and meteorological conditions. The condition of the soil, rock and groundwater may be significantly altered by construction activities (traffic, excavation, groundwater level lowering, pile driving, blasting, etc.) on the site or on adjacent sites. Excavation may expose the soils to changes due to wetting, drying, or frost. Unless otherwise indicated, the soil must be protected from these changes during construction.

Follow-Up and Construction Services: All details of the design were not known at the time of submission of Golder's report. Golder should be retained to review the final design, project plans, and documents, prior to construction, to confirm that they are consistent with the intent of Golder's report.

During construction, Golder should be retained to perform sufficient and timely observations of encountered conditions to confirm and document that the subsurface conditions do not materially differ from those interpreted conditions considered in the preparation of Golder's report and to confirm and document that construction activities do not adversely affect the suggestions, recommendations, and opinions contained in Golder's report. Adequate field review, observation, and testing during construction are necessary for Golder to be able to provide letters of assurance, in accordance with the requirements of many regulatory authorities. In cases where this recommendation is not followed, Golder's responsibility is limited to interpreting accurately, the information encountered at the borehole locations, at the time of their initial determination or measurement during the preparation of the Report.

Changed Conditions and Drainage: Where conditions encountered at the site differ significantly from those anticipated in this report, either due to natural variability of subsurface conditions or construction activities, it is a condition of this report that Golder be notified of any changes and be provided with an opportunity to review or revise the recommendations within this report. Recognition of changed soil and rock conditions requires experience and it is recommended that Golder be employed to visit the site with sufficient frequency to detect if conditions have changed significantly.

Drainage of subsurface water is commonly required, either for temporary or permanent installations for the project. Improper design or construction of drainage or dewatering can have serious consequences. Golder takes no responsibility for the effects of drainage, unless specifically involved in the detailed design and construction monitoring of the system.

Signature Page

Golder Associates Ltd.

Naemeh Naghavi, PhD., P.Eng. (ON)
Geotechnical Engineer

Craig Kelly, P.Eng.
Project Manager, Geotechnical Engineer

Darrin Johnson, P.Eng.
Associate, Geotechnical Engineer

BG/NN/CK/DCJ/MM/sm

https://golderassociates.sharepoint.com/sites/125672/project files/6 deliverables/20142100-008-p200-wsra stockpile design/final report_01apr2021/20142100-008-r-rev0_stockpile design report_01apr_2021.docx

REFERENCES

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Electric Power Research Institute (EPRI, 1993), “*Guidelines for determining design basis ground motions*” Technical Report EPRI TR-102293, Electric Power Research Institute.

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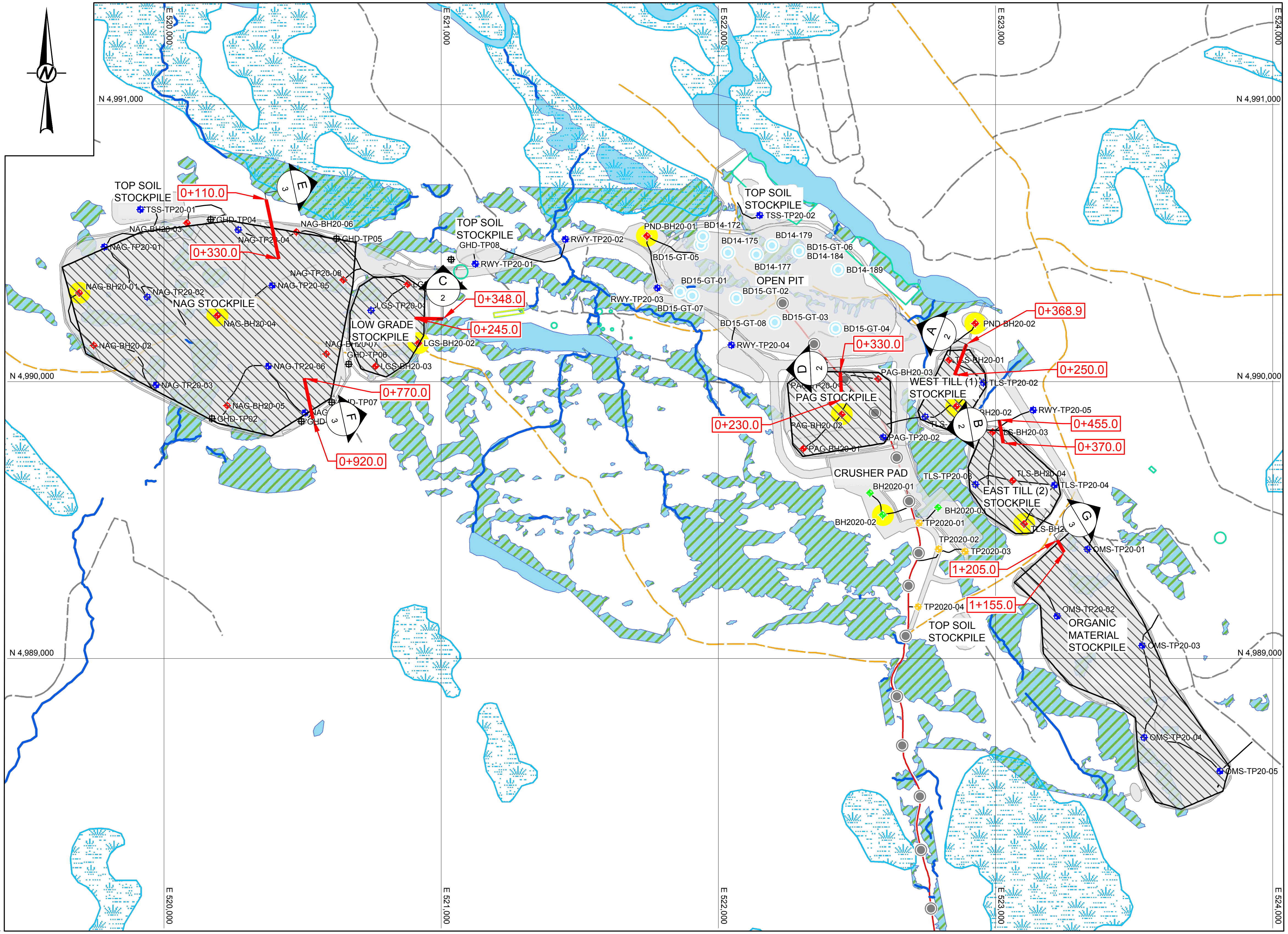


KEY MAP

LEGEND

- TESTPIT LOCATION (GOLDER)
- BOREHOLE LOCATION (GOLDER)
- TESTPIT LOCATION (AUSENCO)
- BOREHOLE LOCATION (AUSENCO)
- TESTPIT LOCATION (GHD)
- PREVIOUS INVESTIGATION LOCATION (WSP)
- PREVIOUS INVESTIGATION LOCATION (AMNS)
- NSE WATERCOURSE
- ARCHEOLOGY SITES
- PROPOSED TRAILS
- PRELIMINARY PROPERTY BOUNDARY
- NSE WETLANDS
- FIELD DELINEATED WETLANDS
- OPEN WATER
- LOCAL ROAD
- DRY WEATHER/SEASONAL ROAD
- TRACK/FORESTRY
- PROPOSED STOCKPILE
- STOCKPILE SECTION LOCATION

NOTE(S)

1. ALL LOCATIONS ARE APPROXIMATE.

REFERENCE(S)

1. BOREHOLE AND TEST PIT LOCATIONS AND BASE INFORMATION PROVIDED BY MCCALLUM ENVIRONMENTAL LTD. ON SEPTEMBER 24, 2020.
2. PROJECTION: TRANSVERSE MERCATOR, DATUM: NAD 83, COORDINATE SYSTEM: UTM ZONE 20.

0 250 500
1:15,000 METRES

CLIENT
ATLANTIC MINING NS CORPORATION

PROJECT
BEAVER DAM MINE WASTE STOCKPILE GEOTECHNICAL DESIGN

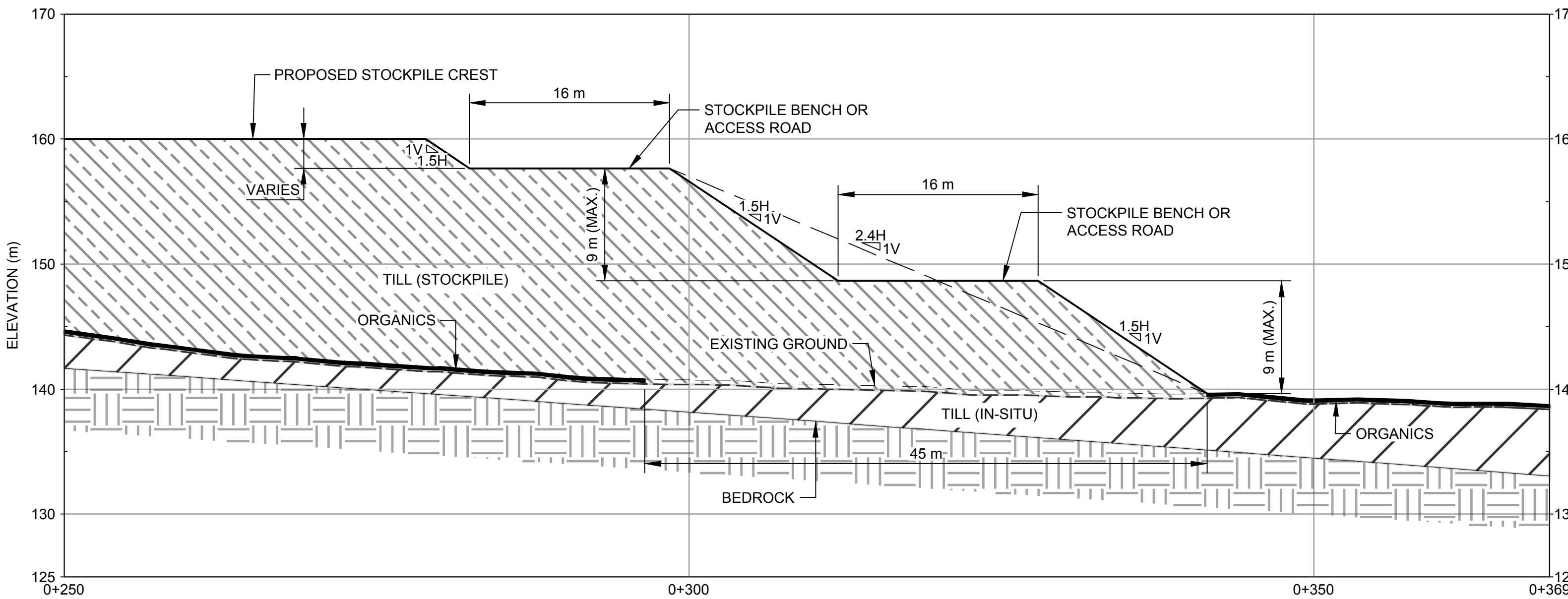
CONSULTANT

GOLDER

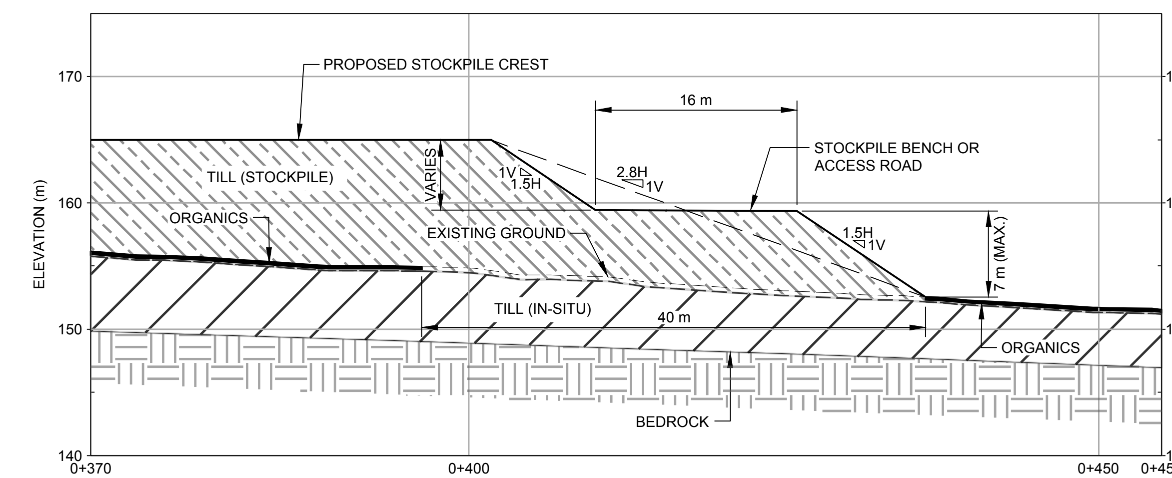
YYYY-MM-DD	2021-03-30
DESIGNED	BG
PREPARED	WS
REVIEWED	MGM
APPROVED	DCJ

TITLE
MINE WASTE STOCKPILE LOCATION PLAN

PROJECT NO.	CONTROL	REV.	FIGURE
20142100	0001	0	1



SCALE 1:600 m A 2 CROSS SECTION - WEST TILL (1) STOCKPILE NORTHEAST SLOPE



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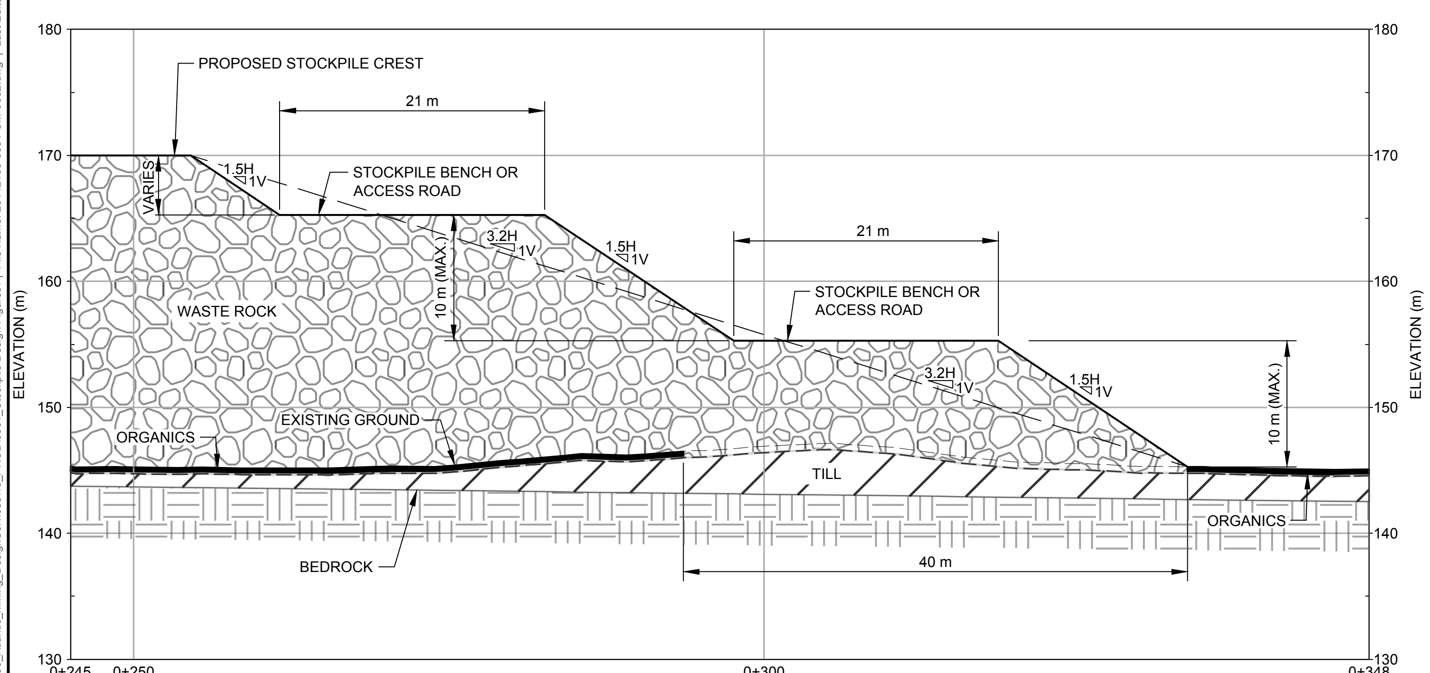

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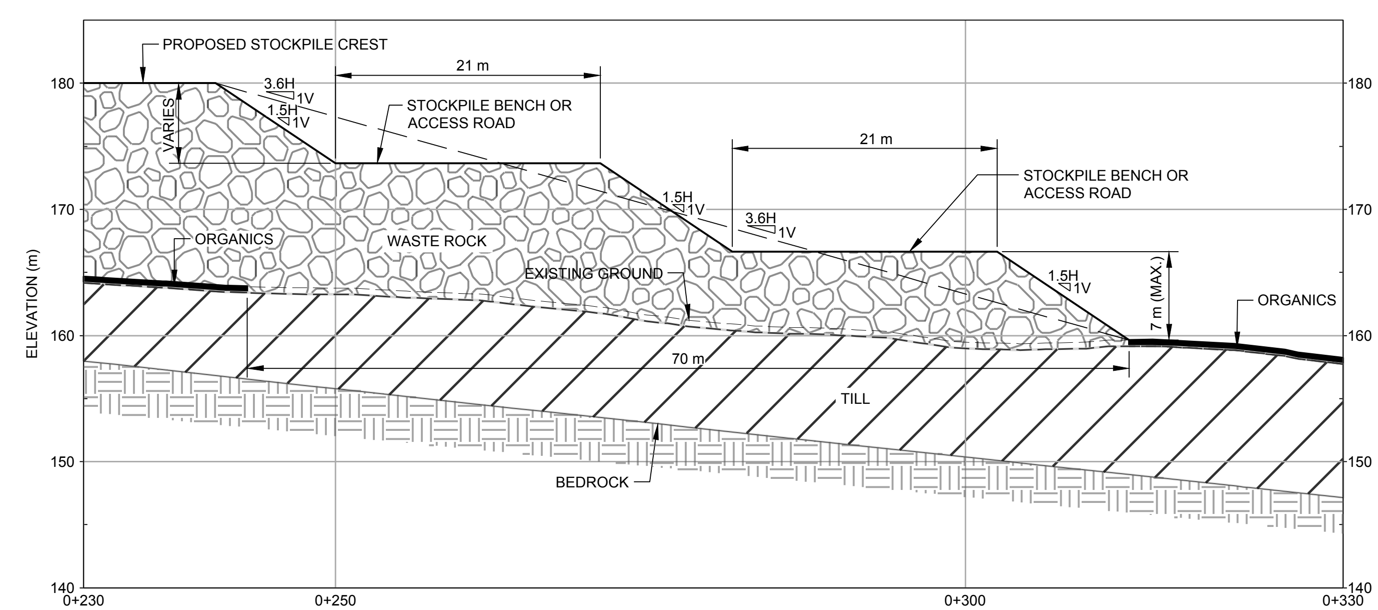


SCALE 1:600 m D 2 CROSS SECTION - PAG STOCKPILE NORTH SLOPE

0 10 20
1:600 METRES

MATERIALS

- BEDROCK
- TILL (IN-SITU)
- ORGANICS
- TILL (STOCKPILE)
- WASTE ROCK

CLIENT
ATLANTIC MINING NS CORPORATION

PROJECT
BEAVER DAM MINE WASTE STOCKPILE GEOTECHNICAL DESIGN

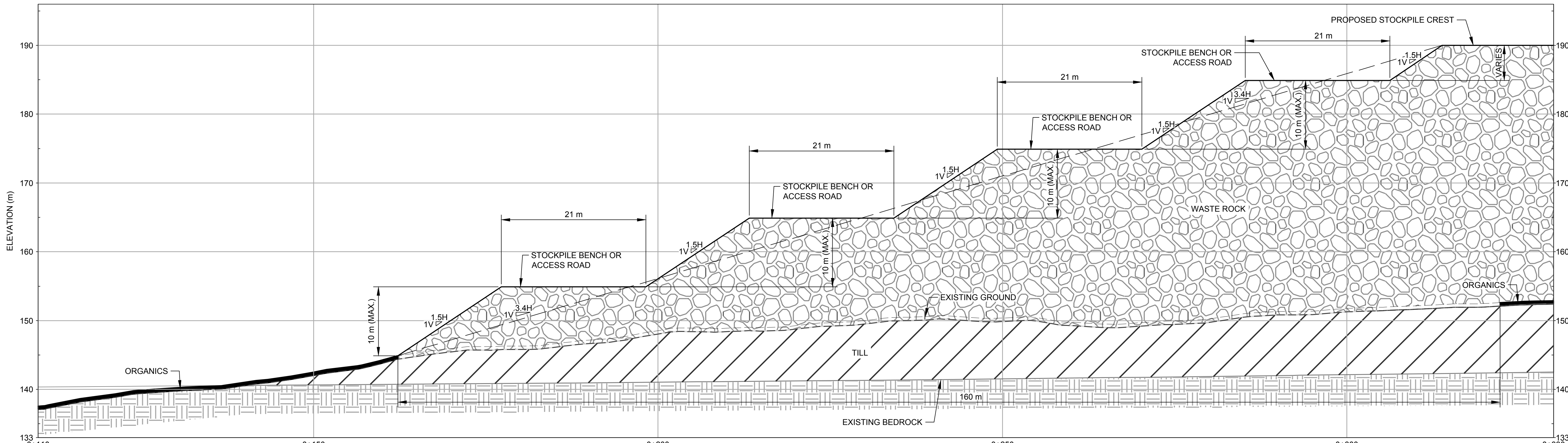
CONSULTANT
GOLDER

YYYY-MM-DD	2021-03-30
DESIGNED	BG
PREPARED	WS
REVIEWED	MGM
APPROVED	DCJ

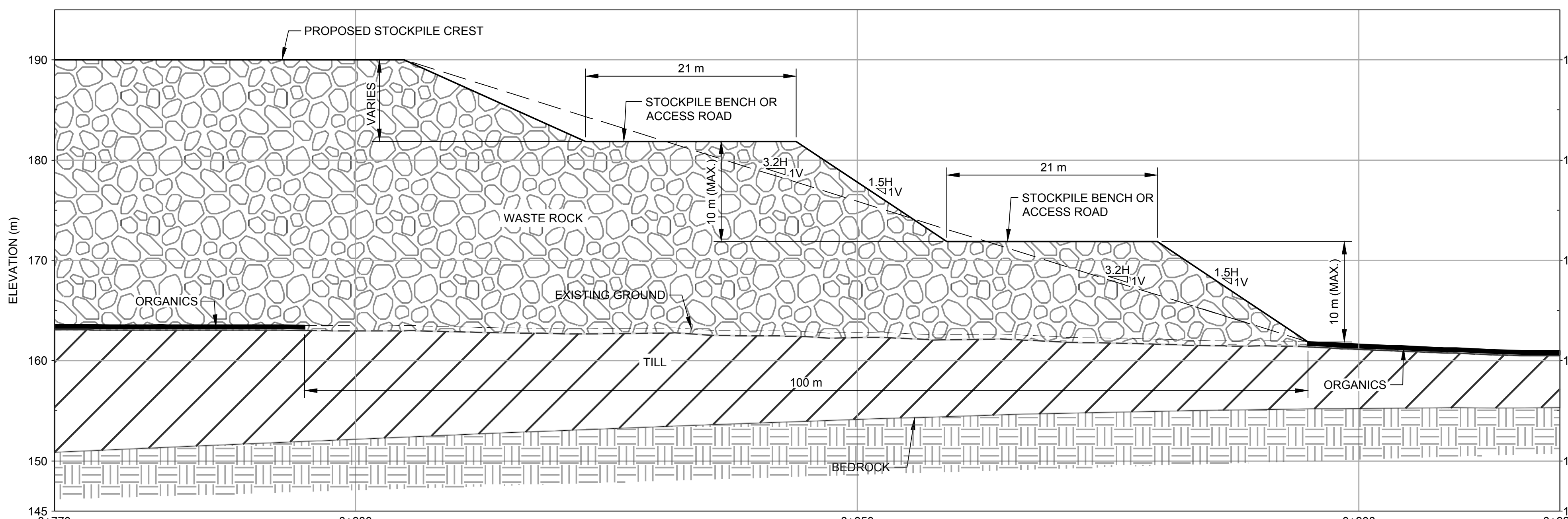
TITLE
MINE WASTE STOCKPILE CROSS SECTIONS AND DETAILS 1

PROJECT NO.	CONTROL	REV.	FIGURE
20142100	0001	0	2

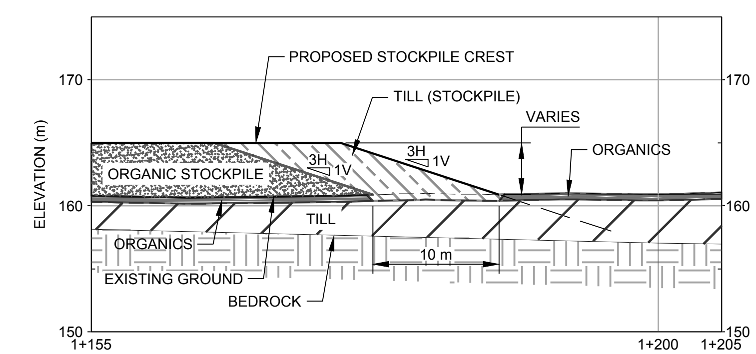
25 mm IF THIS MEASUREMENT DOES NOT MATCH WHAT IS SHOWN, THE SHEET SIZE HAS BEEN MODIFIED FROM: ANSI D



SCALE 1:600 m E/3 CROSS SECTION - NAG STOCKPILE NORTH SLOPE



SCALE 1:600 m F/3 CROSS SECTION - NAG STOCKPILE SOUTH SLOPE



SCALE 1:600 m G/3 CROSS SECTION - ORGANIC STOCKPILE NORTH SLOPE

0 10 20
1:600 METRES

MATERIALS

- BEDROCK
- TILL (IN-SITU)
- ORGANICS
- TILL (STOCKPILE)
- WASTE ROCK
- ORGANIC STOCKPILE

CLIENT	ATLANTIC MINING NS CORPORATION
PROJECT	BEAVER DAM MINE WASTE STOCKPILE GEOTECHNICAL DESIGN
CONSULTANT	GOLDER
YYYY-MM-DD	2021-03-30
DESIGNED	BG
PREPARED	WS
REVIEWED	MGM
APPROVED	DCJ
TITLE	**MINE WASTE STOCKPILE CROSS SECTIONS AND DETAILS 2**
PROJECT NO.	20142100
CONTROL	0001
REV.	0
FIGURE	3

APPENDIX A

Seismic Hazard Calculation

2015 National Building Code Seismic Hazard Calculation

INFORMATION: Eastern Canada English (613) 995-5548 français (613) 995-0600 Facsimile (613) 992-8836
Western Canada English (250) 363-6500 Facsimile (250) 363-6565

Site: 45.066N 62.718W **User File Reference:** Beaver Dam Mine 2021-01-19 12:40 UT

Requested by: Craig Kelly, Golder Associates

Probability of exceedance per annum	0.000404	0.001	0.0021	0.01
Probability of exceedance in 50 years	2 %	5 %	10 %	40 %
Sa (0.05)	0.075	0.041	0.025	0.009
Sa (0.1)	0.105	0.061	0.039	0.014
Sa (0.2)	0.105	0.064	0.042	0.017
Sa (0.3)	0.092	0.058	0.040	0.016
Sa (0.5)	0.079	0.052	0.036	0.014
Sa (1.0)	0.051	0.034	0.023	0.008
Sa (2.0)	0.028	0.018	0.012	0.004
Sa (5.0)	0.007	0.004	0.003	0.001
Sa (10.0)	0.003	0.002	0.001	0.001
PGA (g)	0.061	0.035	0.023	0.008
PGV (m/s)	0.067	0.042	0.027	0.008

Notes: Spectral (Sa(T), where T is the period in seconds) and peak ground acceleration (PGA) values are given in units of g (9.81 m/s^2). Peak ground velocity is given in m/s. Values are for "firm ground" (NBCC2015 Site Class C, average shear wave velocity 450 m/s). NBCC2015 and CSAS6-14 values are highlighted in yellow. Three additional periods are provided - their use is discussed in the NBCC2015 Commentary. Only 2 significant figures are to be used. **These values have been interpolated from a 10-km-spaced grid of points. Depending on the gradient of the nearby points, values at this location calculated directly from the hazard program may vary. More than 95 percent of interpolated values are within 2 percent of the directly calculated values.**

References

National Building Code of Canada 2015 NRCC no. 56190; Appendix C: Table C-3, Seismic Design Data for Selected Locations in Canada

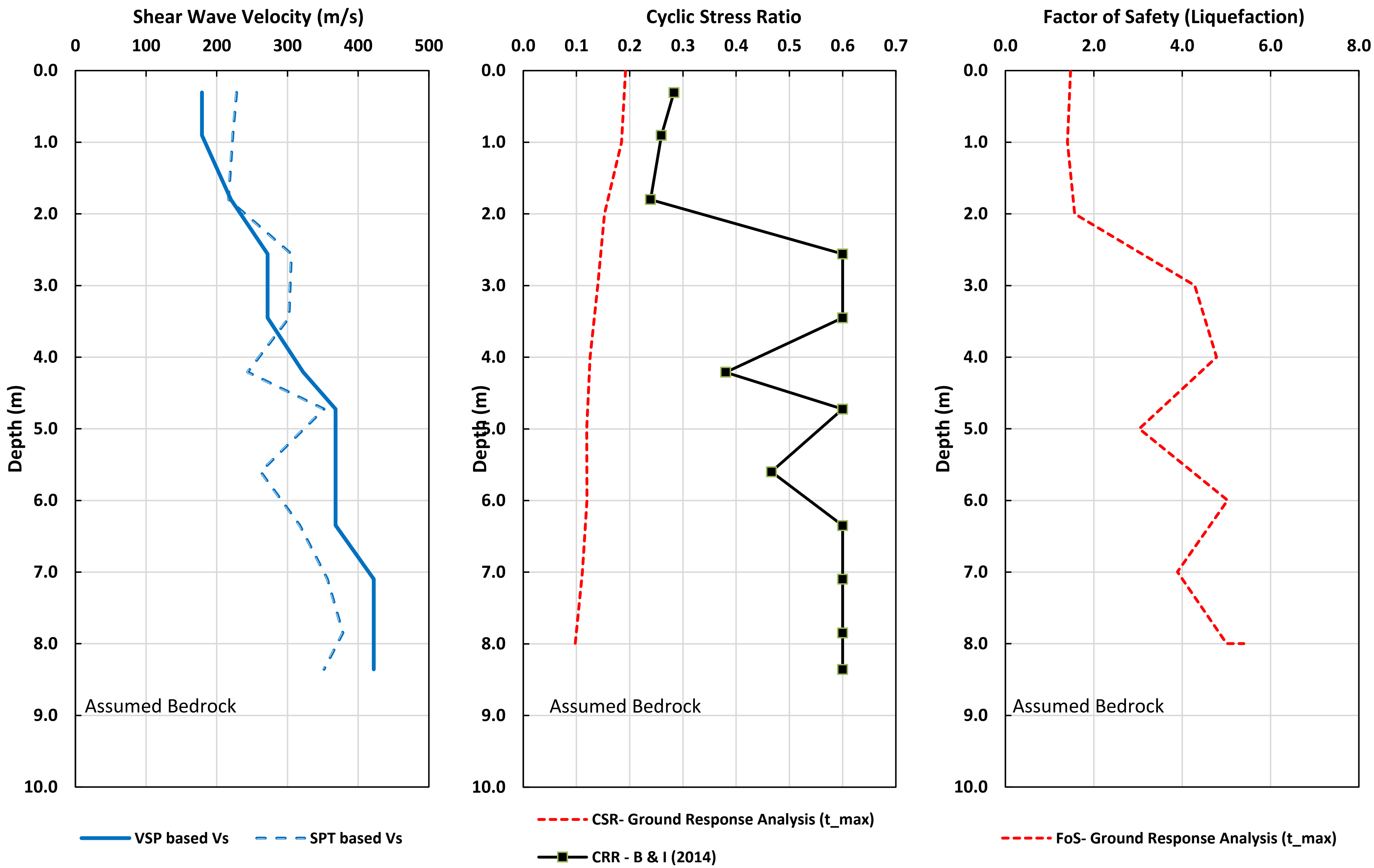
Structural Commentaries (User's Guide - NBC 2015: Part 4 of Division B)
Commentary J: Design for Seismic Effects

Geological Survey of Canada Open File 7893 Fifth Generation Seismic Hazard Model for Canada: Grid values of mean hazard to be used with the 2015 National Building Code of Canada

See the websites www.EarthquakesCanada.ca and www.nationalcodes.ca for more information

APPENDIX B

Liquefaction Assessment Results



CLIENT
ATLANTIC MINING NS Inc.

PROJECT
MINE WASTE STOCKPILE GEOTECHNICAL DESIGN
BEAVER DAM MINE

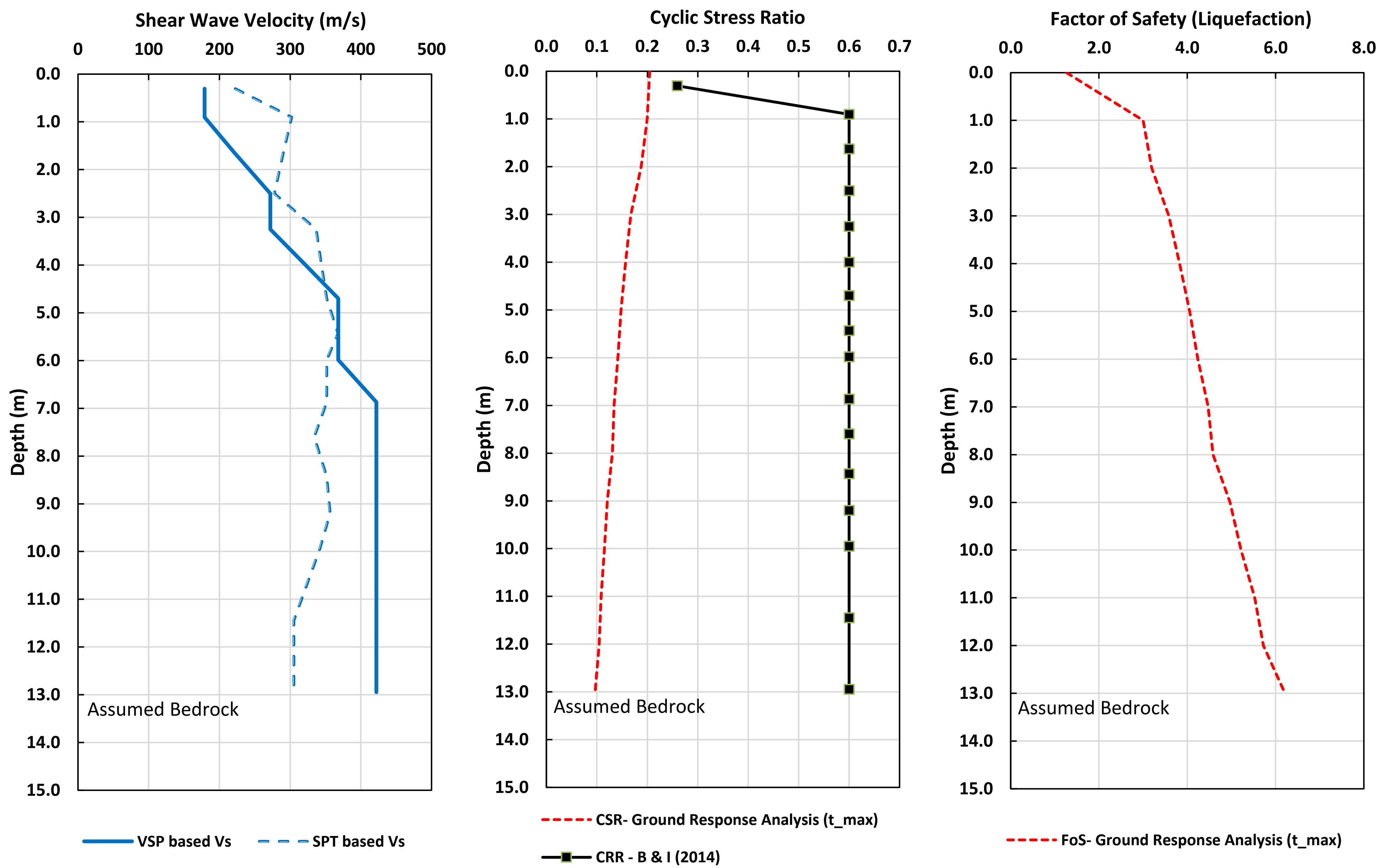
CONSULTANT

GOLDER

YYYY-MM-DD	2021-03-17
DESIGNED	NNA
PREPARED	NNA
REVIEWED	DCJ
APPROVED	DCJ

TITLE
SPT-based Liquefaction Assessment at TLS BH20-03
1:2475 AEP, Mw=6.6, PGA=0.061g

TITLE	PHASE	REV	FIGURE
20142100		**A**	**B-1**



CLIENT **ATLANTIC MINING NS Inc.**		PROJECT **MINE WASTE STOCKPILE GEOTECHNICAL DESIGN** **BEAVER DAM MINE**	
CONSULTANT GOLDER	YYYY-MM-DD	2021-03-17	TITLE **SPT-based Liquefaction Assessment at NAG BH20-07** **1:2475 AEP, Mw=6.6, PGA=0.061g**
	DESIGNED	NNA	
	PREPARED	NNA	
	REVIEWED	DCJ	TITLE **20142100** PHASE REV **A** FIGURE **B-2**
	APPROVED	DCJ	

APPENDIX C

Slope Stability Analysis Results

Table C-1: Summary of Stability Analyses

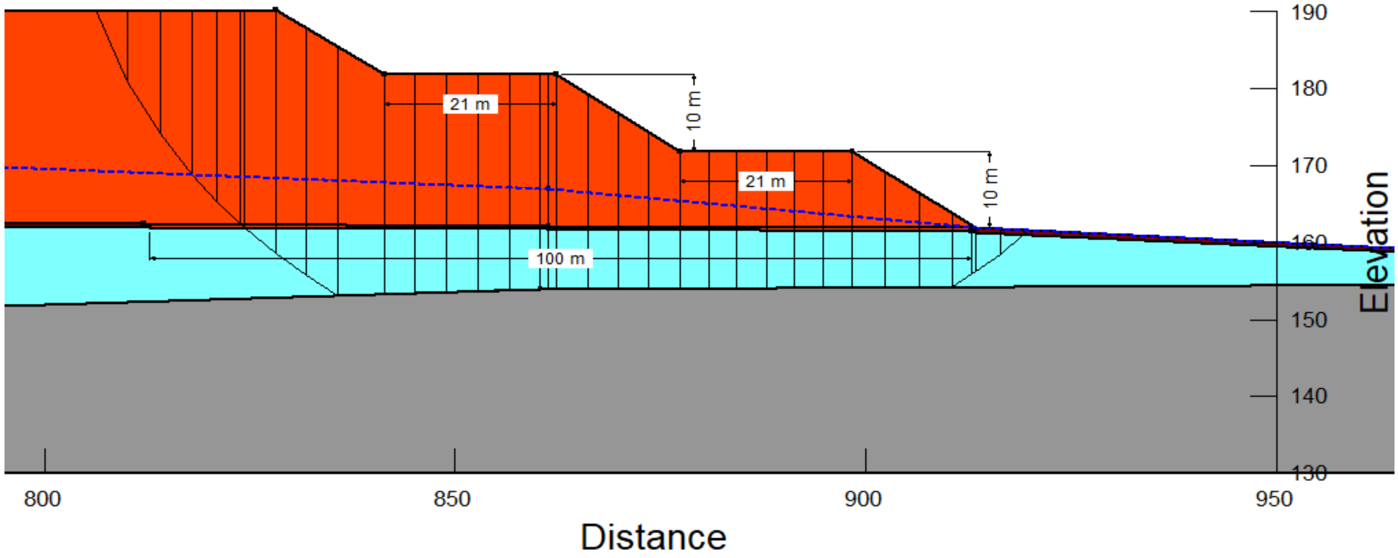
Stockpile	Loading Condition	Crest Level (m)	Max Material Height (m)	Minimum Calculated FOS	Target FOS	Minium FOS	Figure No
NAG	Long-term (steady-state) - Effective Stress	190	45	2.21	1.5	1.2	-
(South Slope)	Long-term (steady-state) - Total Stress			1.49	1.5	1.2	C-1
	Pseudo-Static - Effective Stress			1.98	1.1	1.05	-
	Pseudo-Static - Total Stress			1.31	1.1	1.05	C-1
	Post Seismic			N/A	N/A	N/A	N/A
NAG	Long-term (steady-state) - Effective Stress	190	45	2.02	1.5	1.2	-
(North Slope)	Long-term (steady-state) - Total Stress			1.35	1.5	1.2	C-2
	Pseudo-Static - Effective Stress			1.83	1.1	1.05	-
	Pseudo-Static - Total Stress			1.19	1.1	1.05	C-2
	Post Seismic			N/A	N/A	N/A	N/A
LG	Long-term (steady-state) - Effective Stress	170	25	2.25	1.5	1.2	-
(North Slope)	Long-term (steady-state) - Total Stress			1.94	1.5	1.2	C-3
	Pseudo-Static - Effective Stress			2.05	1.1	1.05	-
	Pseudo-Static - Total Stress			1.73	1.1	1.05	C-3
	Post Seismic			N/A	N/A	N/A	N/A
PAG	Long-term (steady-state) - Effective Stress	180	20	2.06	1.5	1.2	-
(North Slope)	Long-term (steady-state) - Total Stress			1.61	1.5	1.2	C-4
	Pseudo-Static - Effective Stress			1.86	1.1	1.05	-
	Pseudo-Static - Total Stress			1.44	1.1	1.05	C-4
	Post Seismic			N/A	N/A	N/A	N/A
West Till (1)	Long-term (steady-state) - Effective Stress	160 (Northeast)	20	1.74 (1.54 bench)	1.5	1.2	C-5
(Northeast Slope)	Long-term (steady-state) - Total Stress	165 (Southwest)		1.90	1.5	1.2	-
	Pseudo-Static - Effective Stress			1.58 (1.27 bench)	1.1	1.05	C-6
	Pseudo-Static - Total Stress			1.71	1.1	1.05	-
	Post Seismic			N/A	N/A	N/A	N/A
East Till (2)	Long-term (steady-state) - Effective Stress	165	10	1.80	1.5	1.2	C-7
(North Slope)	Long-term (steady-state) - Total Stress			2.21	1.5	1.2	-
	Pseudo-Static - Effective Stress			1.62	1.1	1.05	C-7
	Pseudo-Static - Total Stress			2.01	1.1	1.05	-
	Post Seismic			N/A	N/A	N/A	N/A
Organic	Long-term (steady-state) - Effective Stress	165	5	1.85	1.5	1.2	C-8
(North Slope)	Long-term (steady-state) - Total Stress			1.93	1.5	1.2	-
	Pseudo-Static - Effective Stress			1.64	1.1	1.05	C-8
	Pseudo-Static - Total Stress			1.69	1.1	1.05	-
	Post Seismic			N/A	N/A	N/A	N/A

Notes:
1. Ground topography survey provided by Atlantic Gold.
2. Overburden thicknessed inferred from borehole and test pits data from 2020 geotechnical investigation program.
3. Material strength paramaters based on results obtianed from geotechnical investigation and typical soil parameters from previous project experience.
4. This table should be read in conjunction with the accompanying report.

Long Term (steady-state)

1.49

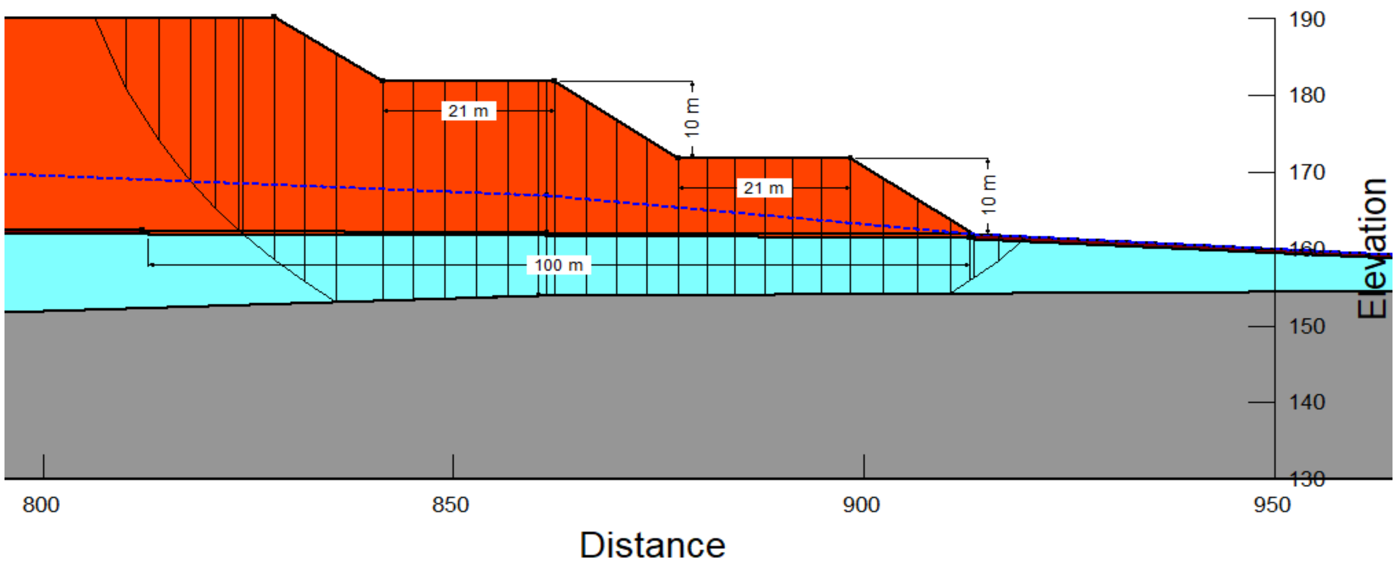
Color	Name	Model	Unit Weight (kN/m³)	Cohesion (kPa)	Effective Cohesion (kPa)	Effective Friction Angle (°)
	Bedrock	Bedrock (Impenetrable)				
	Organics (In-Situ)	Mohr-Coulomb	18		0	10
	Till (Undrained)	Undrained (Phi=0)	22	100		
	Wasterock	Mohr-Coulomb	22		0	38



Pseudo-Static

1.31

Color	Name	Model	Unit Weight (kN/m³)	Cohesion (kPa)	Effective Cohesion (kPa)	Effective Friction Angle (°)
	Bedrock	Bedrock (Impenetrable)				
	Organics (In-Situ)	Mohr-Coulomb	18		0	10
	Till (Undrained)	Undrained (Phi=0)	22	100		
	Wasterock	Mohr-Coulomb	22		0	38



GOLDER

Scale:	N.T.S	**Stability Analysis - Total Stress** **NAG Stockpile South Slope (Crest El. 190 m)**	
Date:	Mar-21		
Design:	MM		
Check:	NN	**ATLANTIC GOLD - BEAVER MINE**	Figure #: C-1
Review:	DCJ		

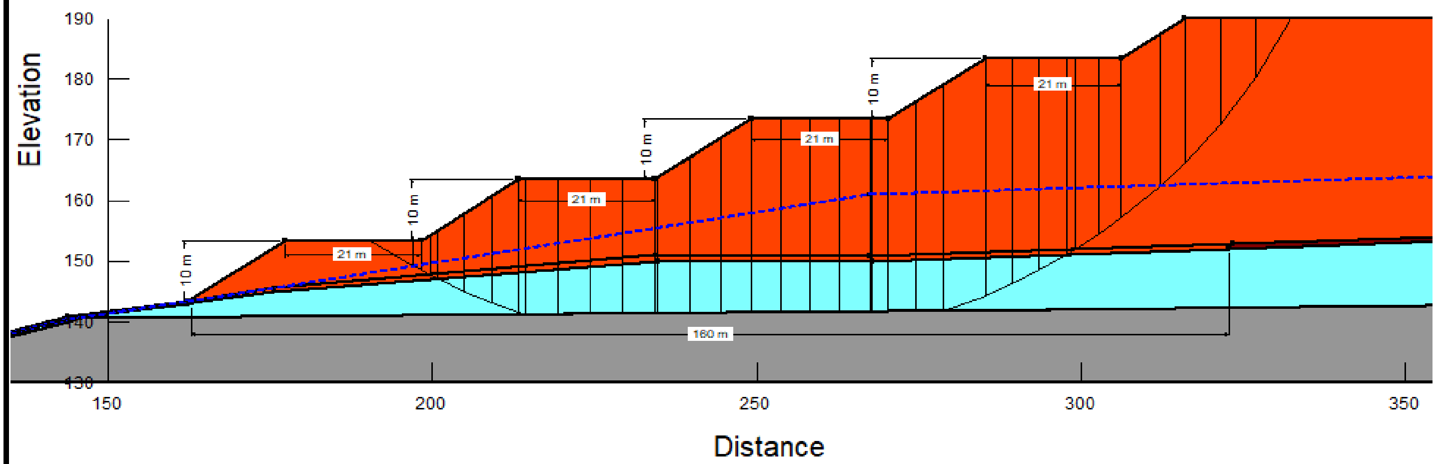
File Name: AppC_Stability Analyses Figures.x
Project No.: 20142100 | Version: 1

Golder Associates

Long Term (steady-state)

1.35

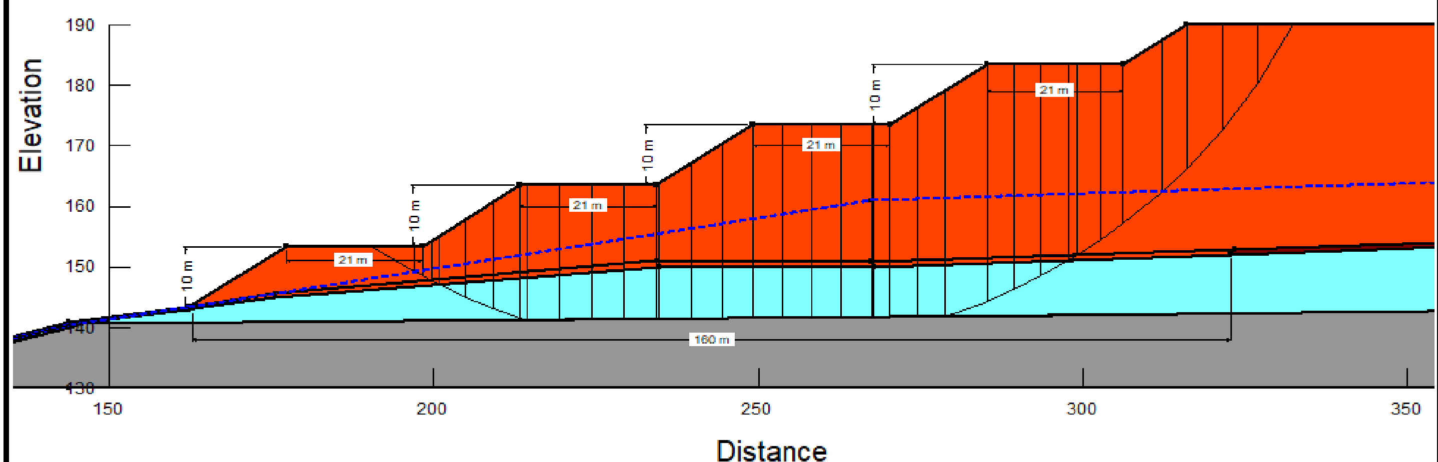
Color	Name	Model	Unit Weight (kN/m³)	Cohesion (kPa)	Effective Cohesion (kPa)	Effective Friction Angle (°)
	Bedrock	Bedrock (Impenetrable)				
	Organics (In-Situ)	Mohr-Coulomb	18		0	10
	Till (Undrained)	Undrained (Phi=0)	22	100		
	Wasterock	Mohr-Coulomb	22		0	38



Pseudo-Static

1.19

Color	Name	Model	Unit Weight (kN/m³)	Cohesion (kPa)	Effective Cohesion (kPa)	Effective Friction Angle (°)
	Bedrock	Bedrock (Impenetrable)				
	Organics (In-Situ)	Mohr-Coulomb	18		0	10
	Till (Undrained)	Undrained (Phi=0)	22	100		
	Wasterock	Mohr-Coulomb	22		0	38



GOLDER

Scale:	N.T.S	**Stability Analysis - Total Stress**	
Date:	Mar-21	**NAG Stockpile North Slope (Crest El. 190 m)**	
Design:	MM		
Check:	NN	**ATLANTIC GOLD - BEAVER MINE**	Figure #: C-2
Review:	DCJ		

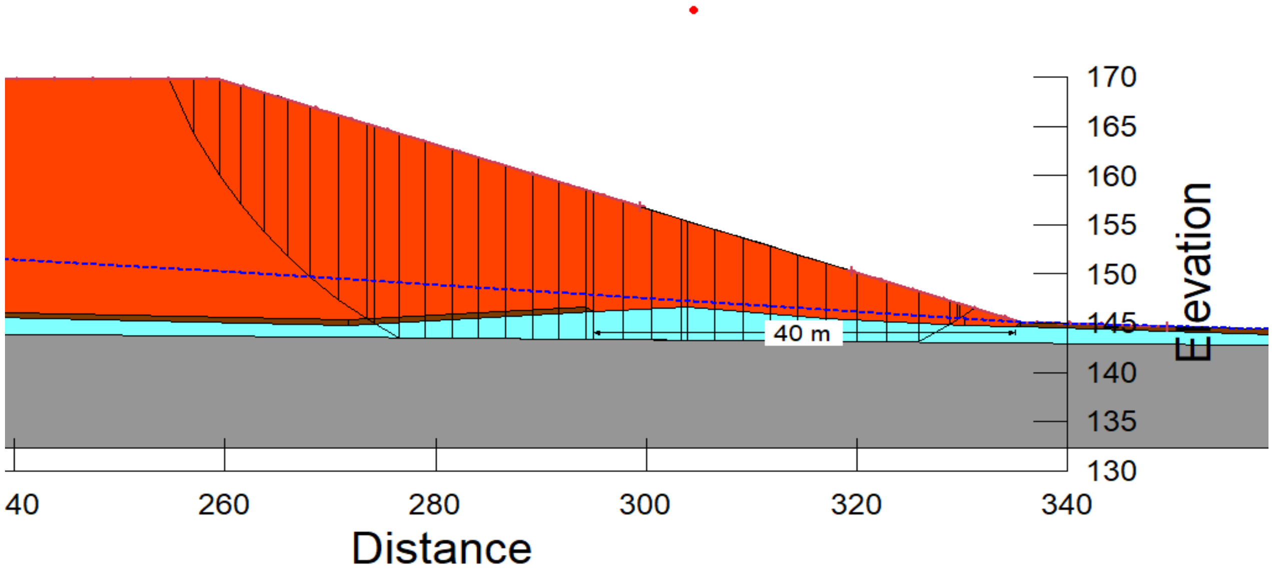
File Name	AppC_Stability Analyses Figures.x		
Project No.	20142100	Version	1

Golder Associates

Long Term (steady-state)

Color	Name	Model	Unit Weight (kN/m³)	Cohesion (kPa)	Effective Cohesion (kPa)	Effective Friction Angle (°)
	Bedrock	Bedrock (Impenetrable)				
	Organics	Mohr-Coulomb	18		0	10
	Till (Undrained)	Undrained (Phi=0)	22	100		
	Wasterock	Mohr-Coulomb	22		0	38

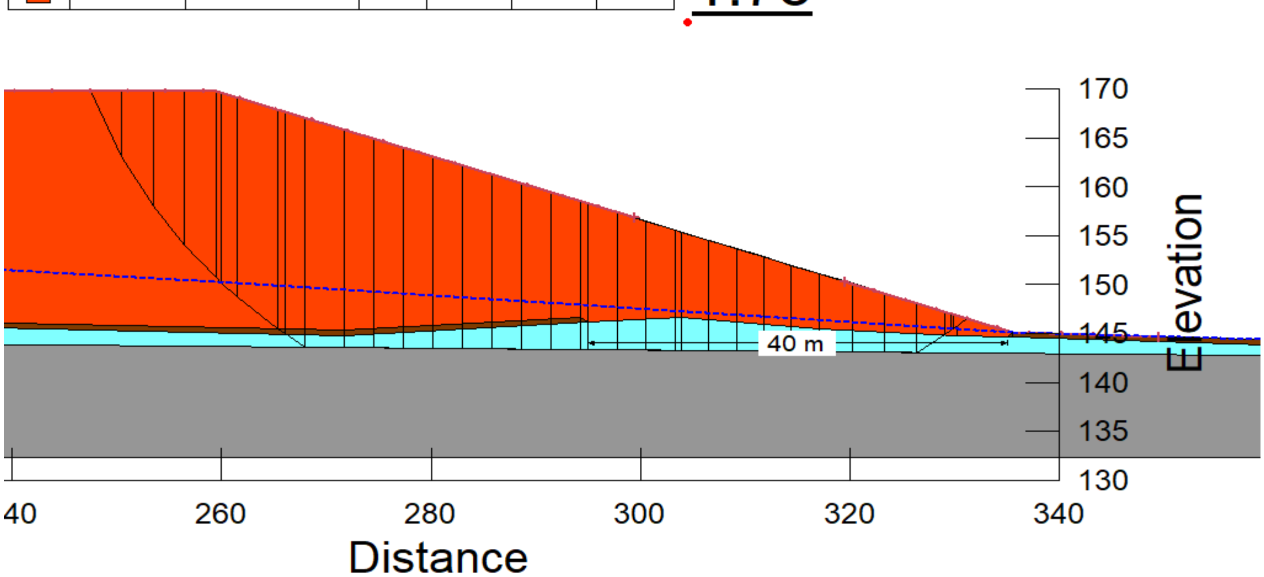
1.94



Pseudo-Static

Color	Name	Model	Unit Weight (kN/m³)	Cohesion (kPa)	Effective Cohesion (kPa)	Effective Friction Angle (°)
	Bedrock	Bedrock (Impenetrable)				
	Organics	Mohr-Coulomb	18		0	10
	Till (Undrained)	Undrained (Phi=0)	22	100		
	Wasterock	Mohr-Coulomb	22		0	38

1.73



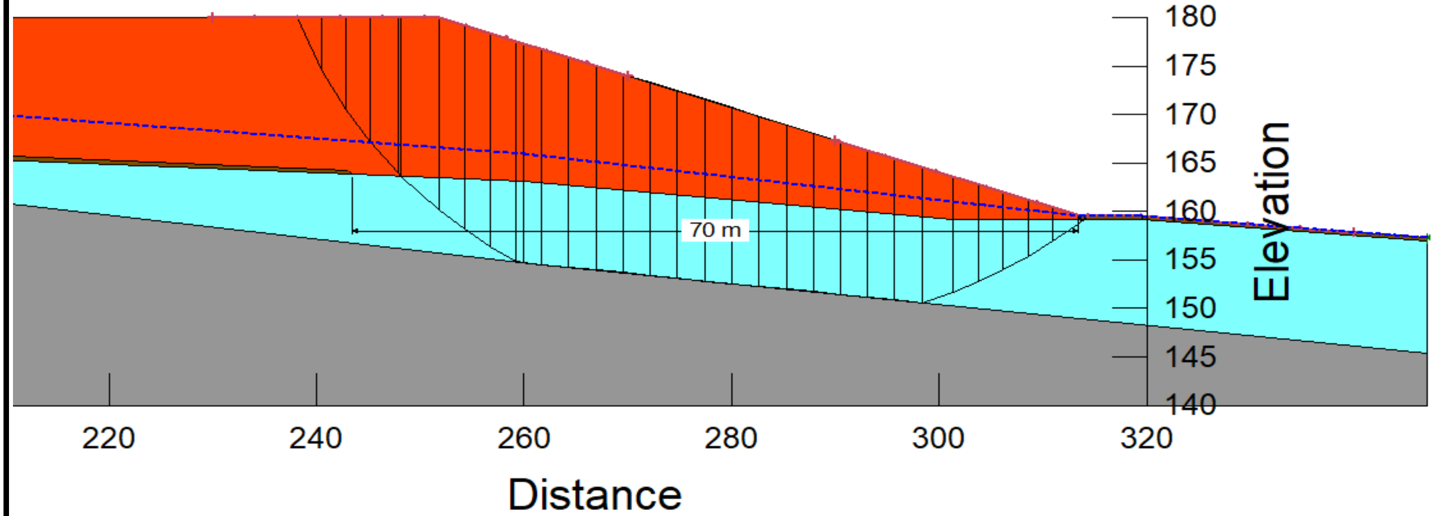
GOLDER	Scale: N.T.S Date: Mar-21 Design: MM	Stability Analysis - Total Stress LG Stockpile East Slope (Crest El. 170 m)	
File Name: AppC_Stability Analyses Figures.x	Check: NN	ATLANTIC GOLD - BEAVER MINE	Figure #: C-3
Project No. 20142100 Version 1	Review: DCJ		

Golder Associates

Long Term (steady-state)

Color	Name	Model	Unit Weight (kN/m³)	Cohesion (kPa)	Effective Cohesion (kPa)	Effective Friction Angle (°)
	Bedrock	Bedrock (Impenetrable)				
	Organics	Mohr-Coulomb	18		0	10
	Till (Undrained)	Undrained (Phi=0)	22	100		
	Wasterock	Mohr-Coulomb	22		0	38

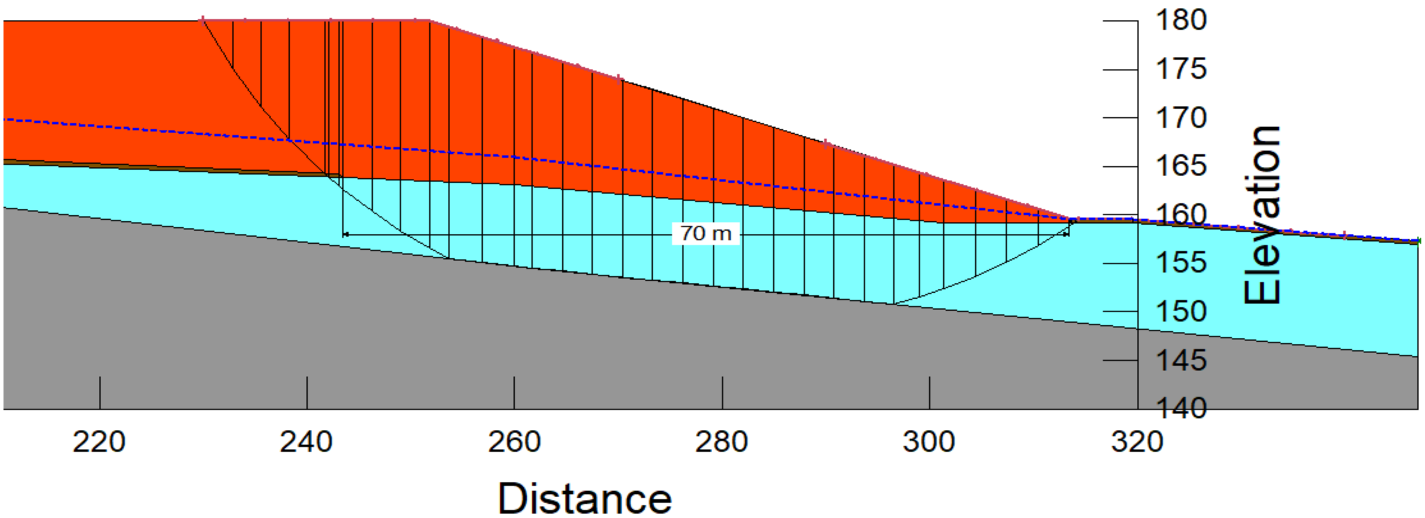
1.61



Pseudo-Static

Color	Name	Model	Unit Weight (kN/m³)	Cohesion (kPa)	Effective Cohesion (kPa)	Effective Friction Angle (°)
	Bedrock	Bedrock (Impenetrable)				
	Organics	Mohr-Coulomb	18		0	10
	Till (Undrained)	Undrained (Phi=0)	22	100		
	Wasterock	Mohr-Coulomb	22		0	38

1.44



GOLDER	Scale: N.T.S Date: Mar-21 Design: MM	Stability Analysis - Total Stress PAG Stockpile North Slope (Crest El. 180 m)	
File Name: AppC_Stability Analyses Figures.x	Check: NN	ATLANTIC GOLD - BEAVER MINE	Figure #: C-4
Project No. 20142100 Version 1	Review: DCJ		

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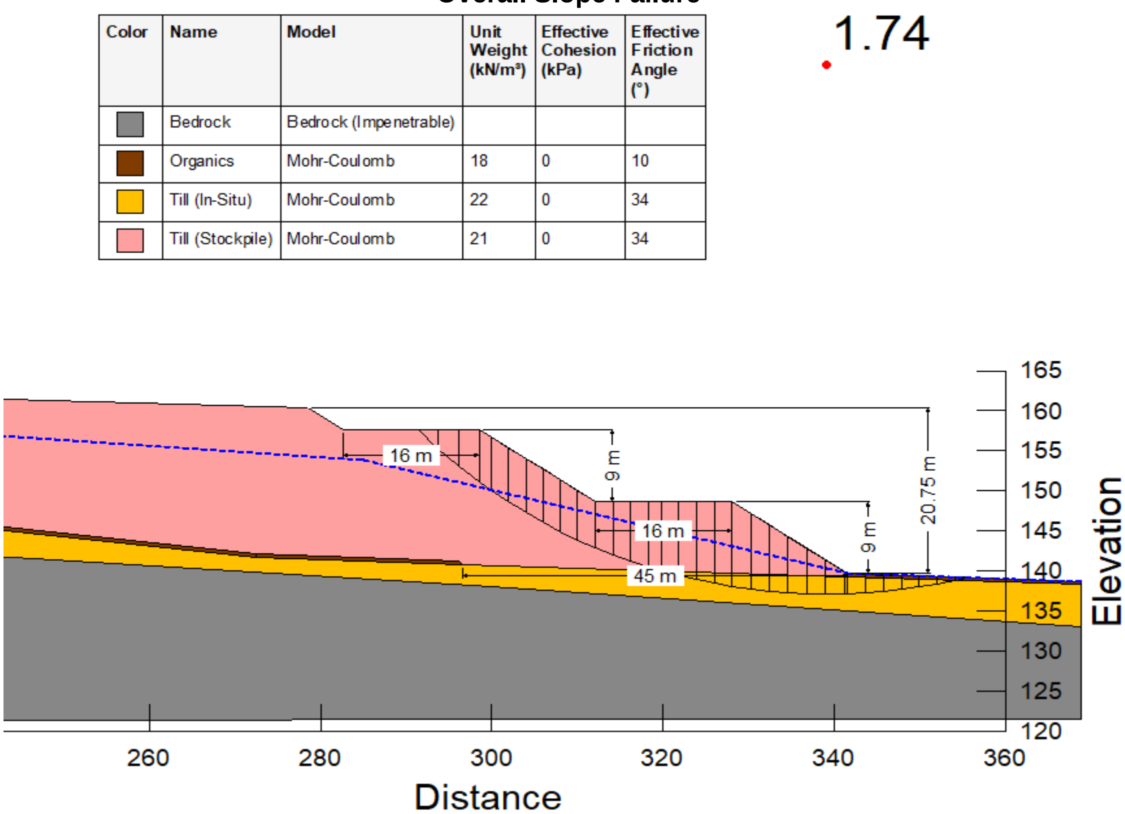
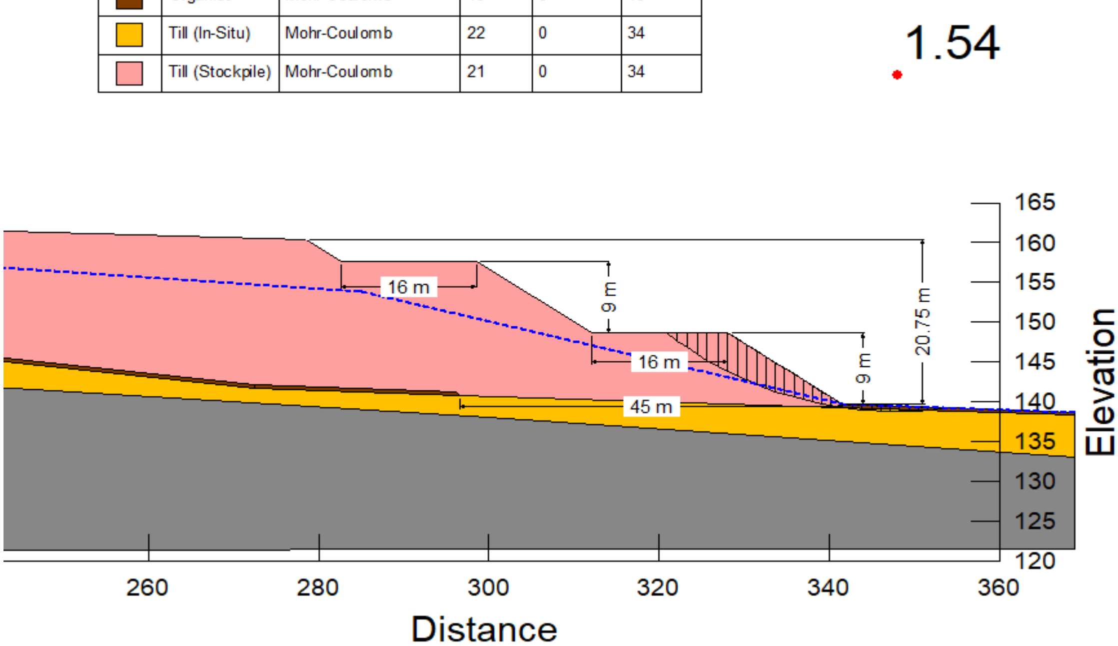
Overall Slope Failure

Color	Name	Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)
	Bedrock	Bedrock (Impenetrable)			
	Organics	Mohr-Coulomb	18	0	10
	Till (In-Situ)	Mohr-Coulomb	22	0	34
	Till (Stockpile)	Mohr-Coulomb	21	0	34



Localized Bottom Bench Failure

Color	Name	Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)
	Bedrock	Bedrock (Impenetrable)			
	Organics	Mohr-Coulomb	18	0	10
	Till (In-Situ)	Mohr-Coulomb	22	0	34
	Till (Stockpile)	Mohr-Coulomb	21	0	34

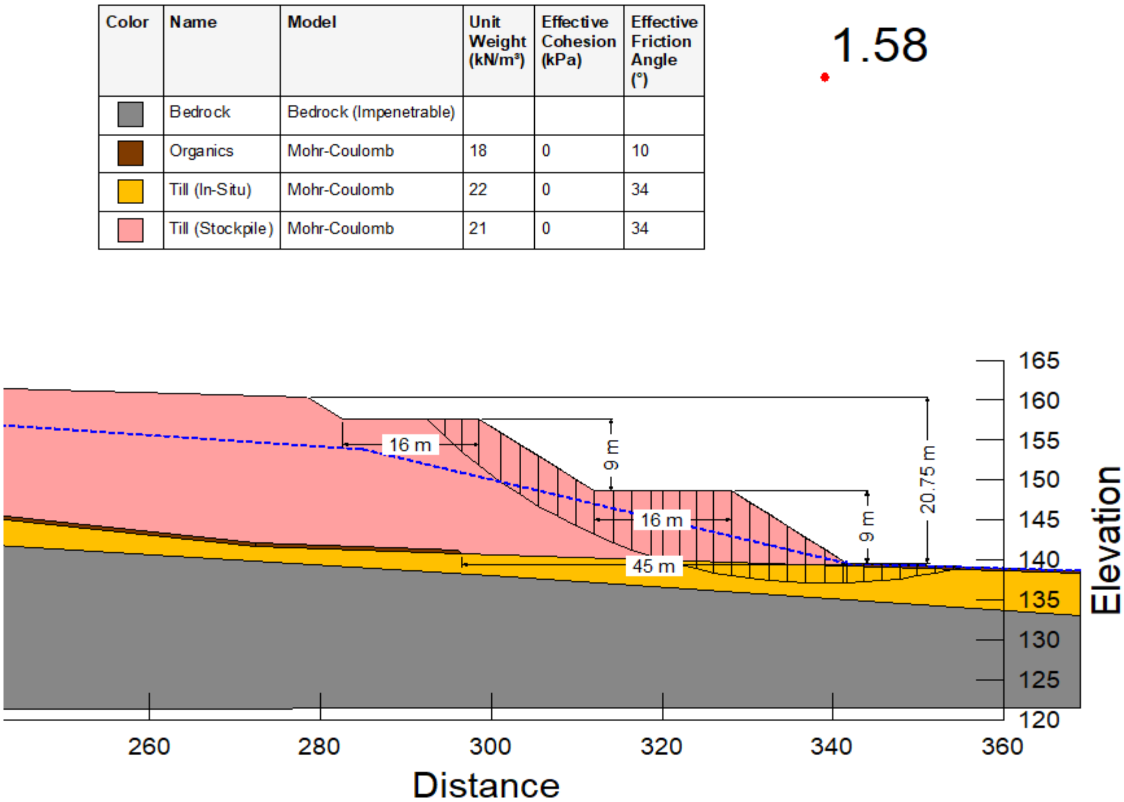


GOLDER		Scale: N.T.S Date: Mar-21 Design: MM		Stability Analysis - Effective Stress Long Term (steady-state) West Till (1) Stockpile Northeast Slope (Crest El. 160 m)	
File Name	AppC_Stability Analyses Figures.x	Check:	NN	ATLANTIC GOLD - BEAVER MINE	Figure #: C-5
Project No.	20142100 Version 1	Review:	DCJ		

Golder Associates

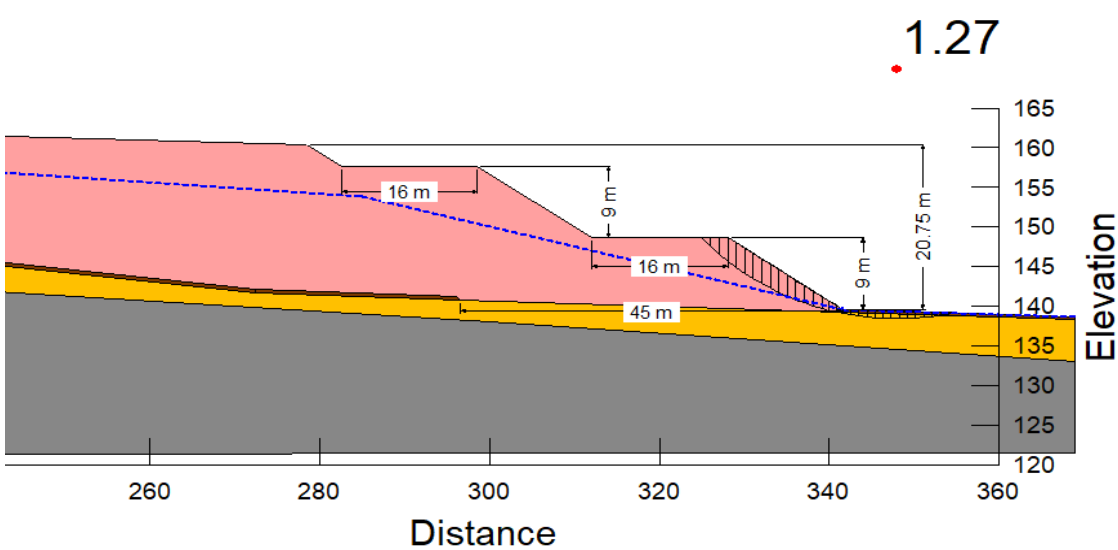
Overall Slope Failure

Color	Name	Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)
	Bedrock	Bedrock (Impenetrable)			
	Organics	Mohr-Coulomb	18	0	10
	Till (In-Situ)	Mohr-Coulomb	22	0	34
	Till (Stockpile)	Mohr-Coulomb	21	0	34



Localized Bottom Bench Failure

Color	Name	Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)
	Bedrock	Bedrock (Impenetrable)			
	Organics	Mohr-Coulomb	18	0	10
	Till (In-Situ)	Mohr-Coulomb	22	0	34
	Till (Stockpile)	Mohr-Coulomb	21	0	34



GOLDER		Scale:	N.T.S	Stability Analysis - Effective Stress Pseudo-Static West Till (1) Stockpile Northeast Slope (Crest El. 160 m)	
		Date:	Mar-21		
		Design:	MM		
File Name	AppC_Stability Analyses Figures.x	Check:	NN	ATLANTIC GOLD - BEAVER MINE	Figure #: C-6
Project No.	20142100 Version 1	Review:	DCJ		

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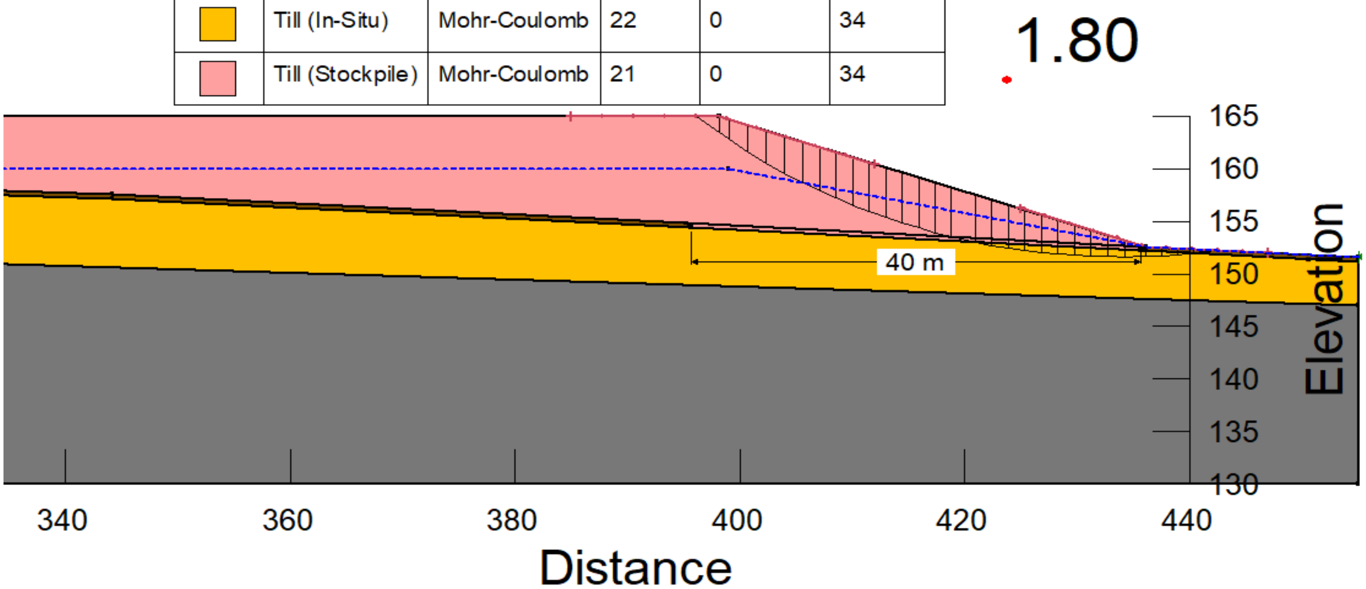
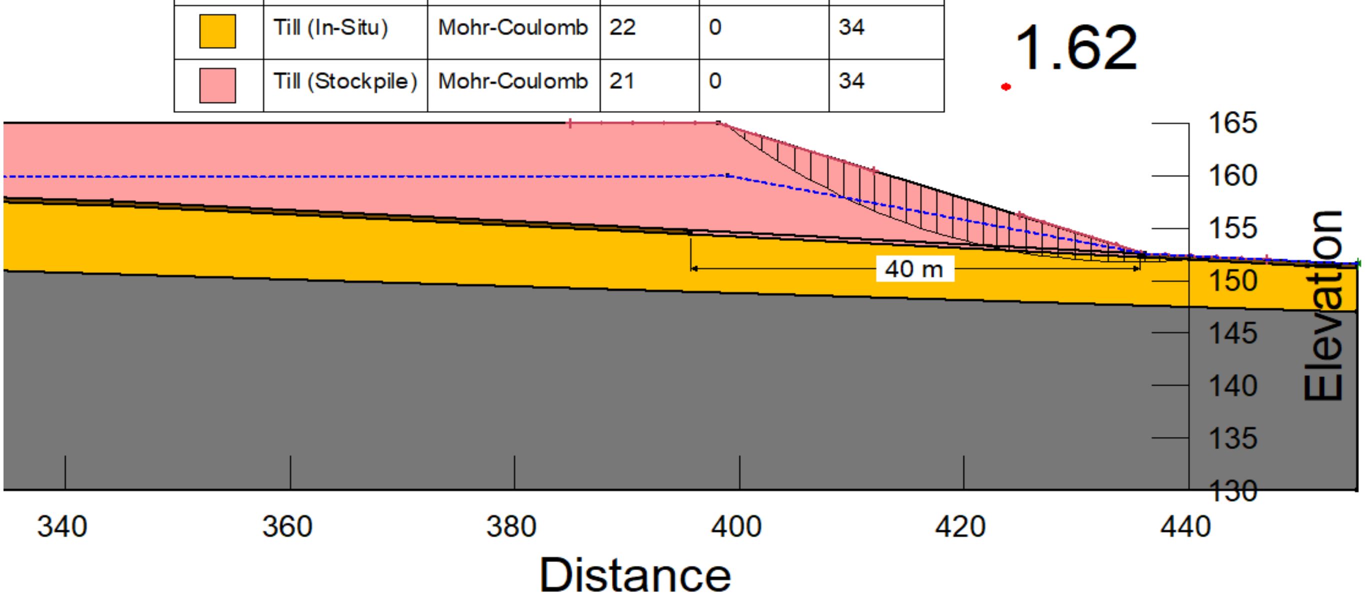
Long Term (steady-state)

Color	Name	Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)
	Bedrock	Bedrock (Impenetrable)			
	Organic	Mohr-Coulomb	18	0	10
	Till (In-Situ)	Mohr-Coulomb	22	0	34
	Till (Stockpile)	Mohr-Coulomb	21	0	34



Pseudo-Static

Color	Name	Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)
	Bedrock	Bedrock (Impenetrable)			
	Organic	Mohr-Coulomb	18	0	10
	Till (In-Situ)	Mohr-Coulomb	22	0	34
	Till (Stockpile)	Mohr-Coulomb	21	0	34

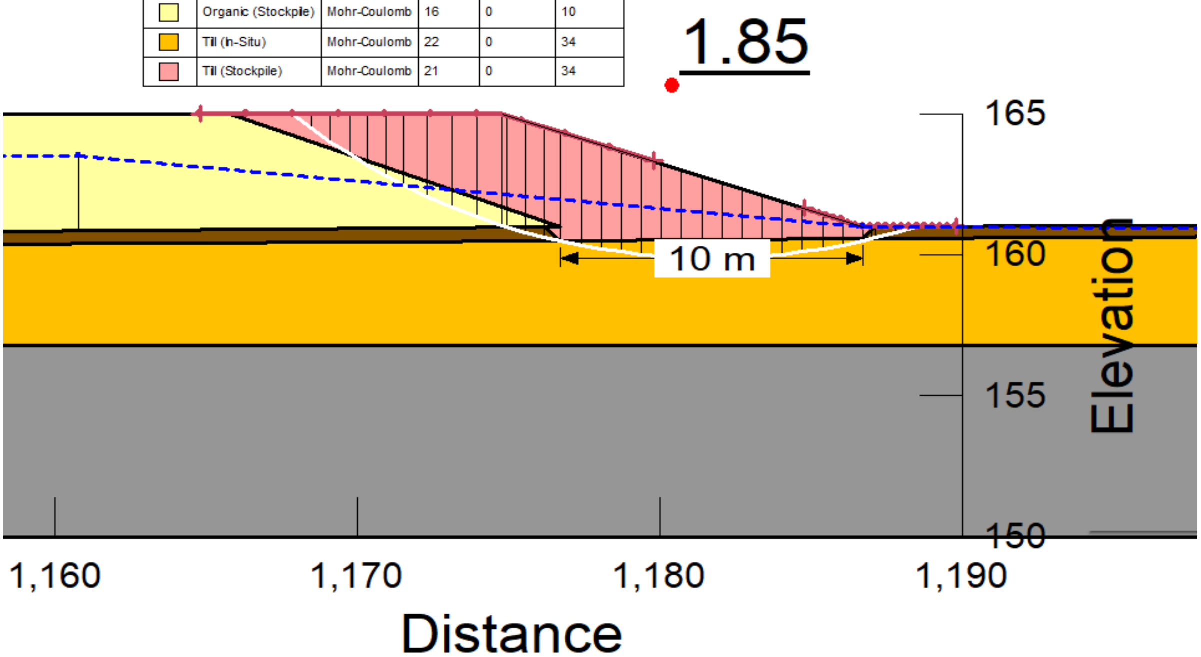


GOLDER		Scale: N.T.S Date: Mar-21 Design: MM	Stability Analysis - Effective Stress East Till (2) Stockpile North Slope (Crest El. 165 m)		
File Name	AppC_Stability Analyses Figures.x	Check: NN	ATLANTIC GOLD - BEAVER MINE	Figure #:	C-7
Project No.	20142100 Version 1	Review: DCJ			

Golder Associates

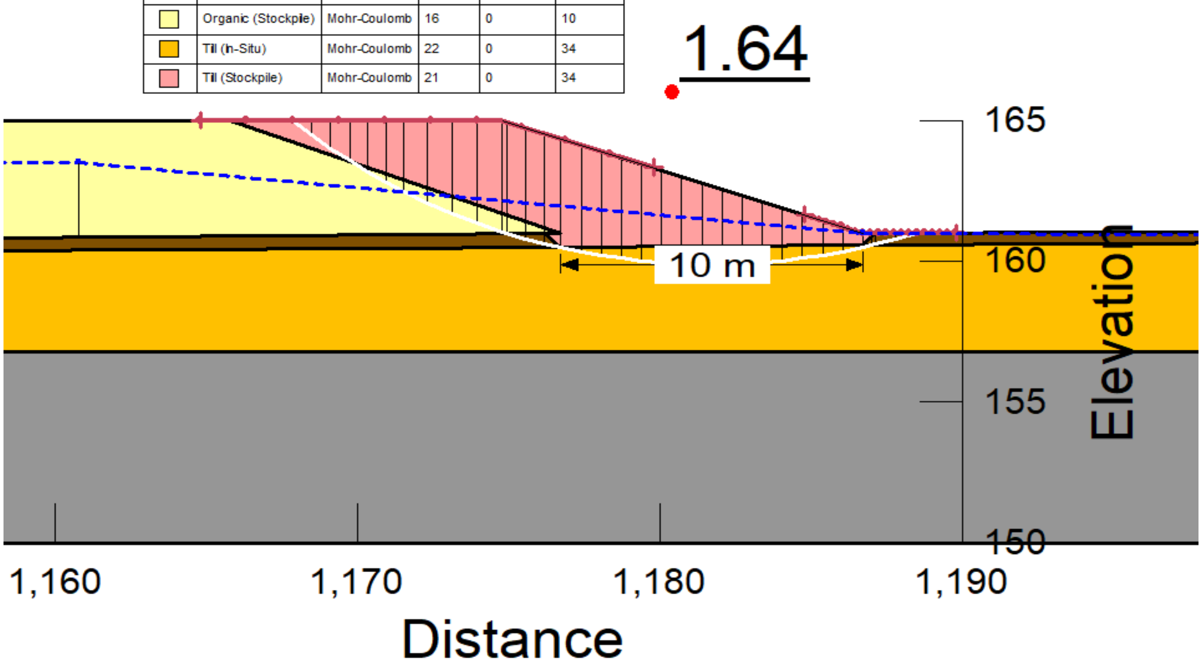
Long Term (steady-state)

Color	Name	Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)
	Bedrock	Bedrock (Impenetrable)			
	Organic (In-Situ)	Mohr-Coulomb	18	0	10
	Organic (Stockpile)	Mohr-Coulomb	16	0	10
	Till (In-Situ)	Mohr-Coulomb	22	0	34
	Till (Stockpile)	Mohr-Coulomb	21	0	34



Pseudo-Static

Color	Name	Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)
	Bedrock	Bedrock (Impenetrable)			
	Organic (In-Situ)	Mohr-Coulomb	18	0	10
	Organic (Stockpile)	Mohr-Coulomb	16	0	10
	Till (In-Situ)	Mohr-Coulomb	22	0	34
	Till (Stockpile)	Mohr-Coulomb	21	0	34



GOLDER	Scale: N.T.S	Stability Analysis - Effective Stress Organic Stockpile North Slope (Crest El. 165 m)
	Date: Mar-21	
	Design: MM	
File Name: AppC_Stability Analyses Figures.x	Check: NN	ATLANTIC GOLD - BEAVER MINE / Figure #: C-8
Project No. 20142100 Version 1	Review: DCJ	

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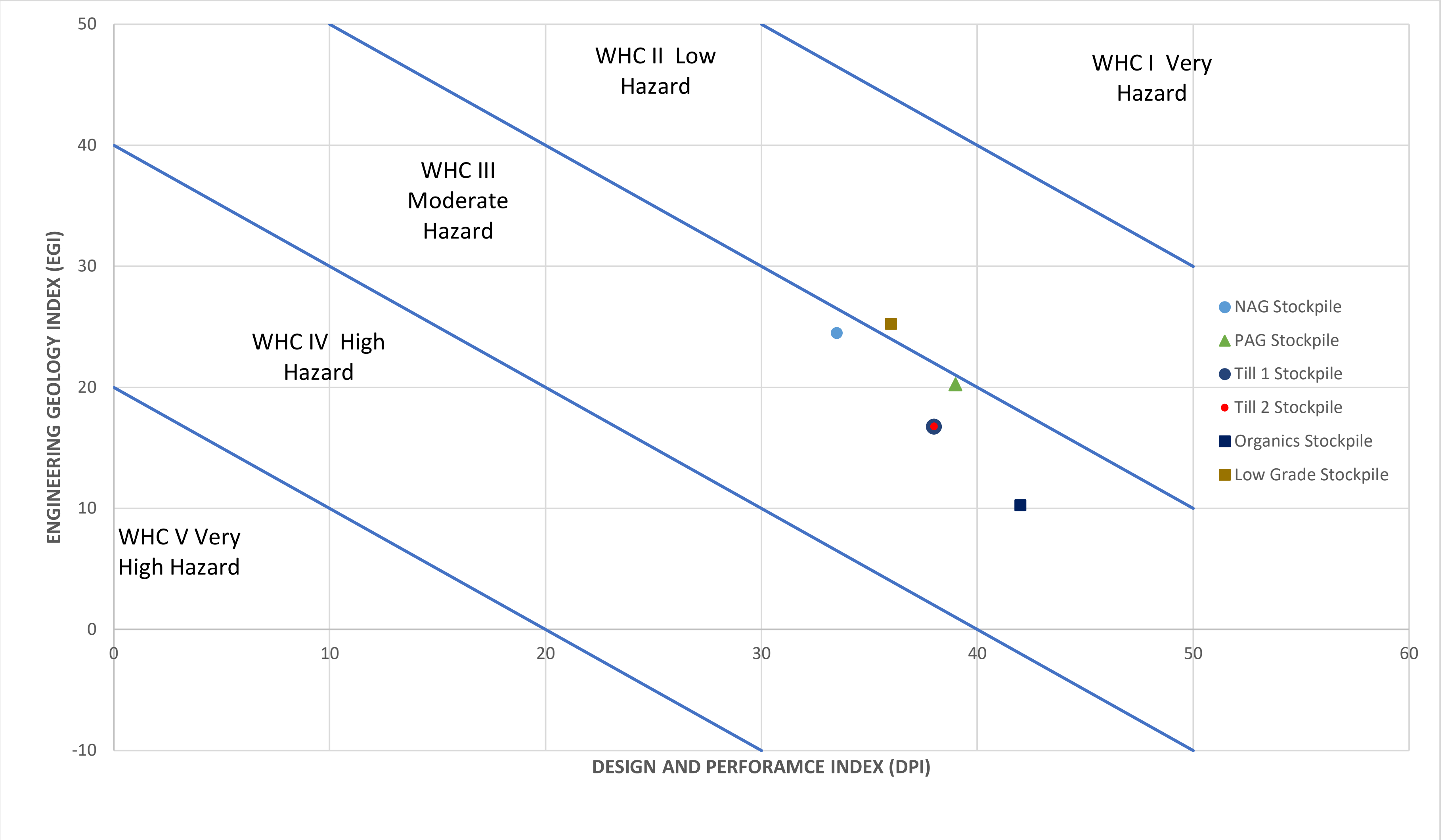
APPENDIX D

Stockpile Hazard Classification



GOLDER

STOCKPILE HAZARD CLASSIFICATION

PROJECT NO:	20142100	DATE:	MAR 2021
BY:	BG	Review:	DCJ

Atlantic Gold Beaver Dam Mine, Nova Scotia

Figure D-1

Engineering Geology Index (EGI)

NAG Stockpile

Group	Factor	Value	Index	Rating
Regional Setting	Seismicity	PGA=0.027g (10 % in 50 years)	Very Low	2
	Precipitation	1981-2010 weather normal: 1357.6mm	High	2
Foundation Conditions	Foundation Slope	~5 degrees (from CAD and field observations)	Gentle (5-15)	4
	Foundation Shape	Planer to concave	Planar/Concave	1.5
	Overburden Type	Glacial Till (moderately dense)	Type IV	3
	Overburden Thickness	2-13 m	>5	0
	Undrained Failure Potential	Borehole logs (WC~PL)	Moderate	-5
	Foundation Liquefaction Potential	Well graded, dense, non-liquefiable soils	Negligible	0
	Bedrock	Moderately competent; slightly weathered	Type C	2
	Groundwater	Groundwater less then 3 m below surface	Moderate	1
Material Quality(1)	Gradation	Assumed: Based 50-75 % greater than 75 mm	Coarse Grained	5
	Intact Strength and Durability	Assumed: Based on Type C bedrock	Type 3	4
	Material Liquefaction Potential	Waste rock, well graded	Negligible	0
	Chemical Stability	Non-acid generating rock	Neutral	5
			Total	24.5

Note: 1) Material characteristic for waste rock estimated

Design and Performance Index (DPI)

NAG Stockpile

Group	Factor	Value	Index	Rating
Geometry & Mass	Height	46 meters	Very Low	4
	Slope Angle	18 degrees	Flat	3
	Volume and Mass	34 million tonnes	Medium	1
Stability Analysis	Static Stability	Static FOS = 1.35	1.3-1.5	5
	Dynamic Stability	Pseudo-static FOS = 1.19	>1.15	3
Construction	Construction Method	Ascending placement on gentle slopes	Method V	8
	Loading Rate (1)	114 t/d/m	High	2
Performance	Stability Performance	Assumed: Stable	Good	7.5
			Total	33.5

Note: 1) Mass loading rate assumes bulk density of 2.00 t/m3

Engineering Geology Index (EGI)

LG Stockpile

Group	Factor	Value	Index	Rating
Regional Setting	Seismicity	PGA=0.027g (10 % in 50 years)	Very Low	2
	Precipitation	1981-2010 weather normal: 1357.6mm	High	2
Foundation Conditions	Foundation Slope	2-10 degrees (from CAD and field observations)	Gentle (5-15)	4
	Foundation Shape	Planer to concave	Planar/Concave	1.5
	Overburden Type	Glacial Till (moderately dense)	Type IV	3
	Overburden Thickness	On average 3.5 m. Greater then 3.5 in some areas.	3 to 5 m	1
	Undrained Failure Potential	Borehole logs (WC~PL)	Moderate	-5
	Foundation Liquefaction Potential	Well graded, dense, non-liquefiable soils	Negligible	0
	Bedrock	Fresh to slightly weather (RQD 75-100)	Type C	2
	Groundwater	Ground water (0.5 to 2.9 mbgs)	Moderate/High	0.75
Material Quality(1)	Gradation	Assumed: Based 50-75 % greater than 75 mm	Coarse Grained	5
	Intact Strength and Durability	Assumed: Based on Type C bedrock	Type 3	4
	Material Liquefaction Potential	Waste rock, well graded	Negligible	0
	Chemical Stability	Non-acid generating rock	Neutral	5
			Total	25.25

Note: 1) Material characteristic for waste rock estimated

Design and Performance Index (DPI)

LG Stockpile

Group	Factor	Value	Index	Rating
Geometry & Mass	Height	Approx. height 14-26 m	Very Low	4
	Slope Angle	18 degrees	Flat	3
	Volume and Mass	2.48 million tonnes	Small	1.5
Stability Analysis	Static Stability	Static FOS = 1.94	>1.5	7
	Dynamic Stability	Pseudo-static FOS = 1.73	>1.15	3
Construction	Construction Method	Ascending placement on gentle slopes	Method V	8
	Loading Rate (1)	Assumed	High	2
Performance	Stability Performance	Assumed: Stable	Good	7.5
			Total	36

Note: 1) Mass loading rate assumes bulk density of 2.00 t/m3

Engineering Geology Index (EGI)

PAG Stockpile

Group	Factor	Value	Index	Rating
Regional Setting	Seismicity	PGA=0.027g (10 % in 50 years)	Very Low	2
	Precipitation	1981-2010 weather normal: 1357.6mm	High	2
Foundation Conditions	Foundation Slope	10-15 degrees (from CAD and field observations)	Gentle (5-15)	4
	Foundation Shape	Planer to concave	Planar/Concave	1.5
	Overburden Type	Glacial Till (moderately dense)	Type IV	3
	Overburden Thickness	On average 2 m, in areas >4.0 m	3 to 5 m	1
	Undrained Failure Potential	Borehole logs (WC~PL)	Moderate	-5
	Foundation Liquefaction Potential	Well graded, dense, non-liquefiable soils	Negligible	0
	Bedrock	Fresh to slightly weather (RQD 35-90)	Type C	2
	Groundwater	Groundwater (0.7 to 1.5 mbgs)	Moderate/High	0.75
Material Quality(1)	Gradation	Assumed: Based 50-75 % greater than 75 mm	Coarse Grained	5
	Intact Strength and Durability	Assumed: Based on Type C bedrock	Type 3	4
	Material Liquefaction Potential	Waste rock, well graded	Negligible	0
	Chemical Stability	Potential for generation of ARD	Moderately Reactive	0
			Total	20.25

Note: 1) Material characteristic for waste rock estimated

Design and Performance Index (DPI)

PAG Stockpile

Group	Factor	Value	Index	Rating
Geometry & Mass	Height	Approx. height 14-26 m	Very Low	4
	Slope Angle	18 degrees	Flat	3
	Volume and Mass	2.5 million tonnes	Small	1.5
Stability Analysis	Static Stability	Static FOS = 1.61	>1.5	7
	Dynamic Stability	Pseudo-static FOS = 1.44	>1.15	3
Construction	Construction Method	Ascending placement on gentle slopes	Method V	8
	Loading Rate (1)	18 t/d/m	Low	5
Performance	Stability Performance	Assumed: Stable	Good	7.5
			Total	39

Note: 1) Mass loading rate assumes bulk density of 2.00 t/m3

Engineering Geology Index (EGI)

Till 1 Stockpile

Group	Factor	Value	Index	Rating
Regional Setting	Seismicity	PGA=0.027g (10 % in 50 years)	Very Low	2
	Precipitation	1981-2010 weather normal: 1357.6mm	High	2
Foundation Conditions	Foundation Slope	10-15 degrees (from CAD and field observations)	Gentle (5-15)	4
	Foundation Shape	Planer to concave	Planar/Concave	1.5
	Overburden Type	Glacial Till (moderately dense)	Type IV	3
	Overburden Thickness	Till between 0.5 to 10 m	>5	0
	Undrained Failure Potential	Borehole logs (WC~PL)	Moderate	-5
	Foundation Liquefaction Potential	Well graded, dense, non-liquefiable soils	Negligible	0
	Bedrock	Fresh (RQD 52-89)	Type C	2
	Groundwater	Groundwater b/w 0.2 m to 3.5 m below ground	Moderate/High	0.75
Material Quality	Gradation	Average fines content from lab samples 29-59	Fine Grained/ Mixed Grain	2
	Intact Strength and Durability	Fine/mixed grain size overburden	Type 2	2
	Material Liquefaction Potential	Low liquefaction potential but cannot be discounted	Low	-2.5
	Chemical Stability	Neutral	Neutral	5
			Total	16.75

Design and Performance Index (DPI)

TILL 1 Stockpile

Group	Factor	Value	Index	Rating
Geometry & Mass	Height	Approx. height 10-25 m	Very Low	4
	Slope Angle	18 degrees	Flat	3
	Volume and Mass	0.69 million tonnes	Very Small	2
Stability Analysis	Static Stability	Static FOS = 1.74	>1.5	7
	Dynamic Stability	Pseudo-static FOS = 1.58	>1.15	3
Construction	Construction Method	Ascending placement on gentle slopes	Method V	8
	Loading Rate (1)	55 t/d/m	Moderate	3.5
Performance	Stability Performance	Assumed: Stable	Good	7.5
			Total	38

Note: 1) Mass loading rate assumes bulk density of 2.00 t/m3

Engineering Geology Index (EGI)

Till 2 Stockpile

Group	Factor	Value	Index	Rating
Regional Setting	Seismicity	PGA=0.027g (10 % in 50 years)	Very Low	2
	Precipitation	1981-2010 weather normal: 1357.6mm	High	2
Foundation Conditions	Foundation Slope	4-10 degrees (from CAD and field observations)	Gentle (5-15)	4
	Foundation Shape	Planer to concave	Planar/Concave	1.5
	Overburden Type	Glacial Till (moderately dense)	Type IV	3
	Overburden Thickness	O/B thickness 7 to 9 m	>5	0
	Undrained Failure Potential	Borehole logs (WC~PL)	Moderate	-5
	Foundation Liquefaction Potential	Well graded, dense, non-liquefiable soils	Negligible	0
	Bedrock	Fresh (RQD 62-100)	Type C	2
	Groundwater	Groundwater b/w 1.8 m to 3.0 m below ground	Moderate/High	0.75
Material Quality	Gradation	Average fines content from lab samples 29-59	Fine Grained/ Mixed Grain	2
	Intact Strength and Durability	Fine/mixed grain size overburden	Type 2	2
	Material Liquefaction Potential	Low liquefaction potential but cannot be discounted	Low	-2.5
	Chemical Stability	Neutral	Neutral	5
			Total	16.75

Design and Performance Index (DPI)

TILL 2 Stockpile

Group	Factor	Value	Index	Rating
Geometry & Mass	Height	Approx. stockpile height 3-10 m	Very Low	4
	Slope Angle	18 degrees	Flat	3
	Volume and Mass	1.97 million tonnes	Very Small	2
Stability Analysis	Static Stability	Static FOS = 1.80	>1.5	7
	Dynamic Stability	Pseudo-static FOS = 1.62	>1.15	3
Construction	Construction Method	Ascending placement on gentle slopes	Method V	8
	Loading Rate (1)	46 t/d/m	Moderate	3.5
Performance	Stability Performance	Assumed: Stable	Good	7.5
			Total	38

Note: 1) Mass loading rate assumes bulk density of 2.00 t/m3

Engineering Geology Index (EGI)

Organics Stockpile

Group	Factor	Value	Index	Rating
Regional Setting	Seismicity	PGA=0.027g (10 % in 50 years)	Very Low	2
	Precipitation	1981-2010 weather normal: 1357.6mm	High	2
Foundation Conditions	Foundation Slope	3-6 degrees (from CAD and field observations)	Gentle (5-15)	4
	Foundation Shape	Planer to concave	Planar/Concave	1.5
	Overburden Type	Glacial Till (moderately dense)	Type IV	3
	Overburden Thickness	O/B thickness 1 to >4.9 m	>5	0
	Undrained Failure Potential	Borehole logs (WC~PL)	Moderate	-5
	Foundation Liquefaction Potential	Well graded, dense, non-liquefiable soils	Negligible	0
	Bedrock	No boreholes, assume Type C	Type C	2
	Groundwater	Groundwater 0.1 to 3.7 m below ground surface	Moderate/High	0.75
Material Quality	Gradation	Very fined grained organics	Very fined grained	0
	Intact Strength and Durability	Extremely weak	Type I	0
	Material Liquefaction Potential	Moderate or unknown liquefaction potential	Unknown	-5
	Chemical Stability	Assumed neutral	Neutral	5
			Total	10.25

Design and Performance Index (DPI)

ORGANIC Stockpile

Group	Factor	Value	Index	Rating
Geometry & Mass	Height	Approx. height 4 m	Very Low	4
	Slope Angle	8 degrees	Very Flat	4
	Volume and Mass	2.29 million tonnes	Small	1.5
Stability Analysis	Static Stability	With till exterior slope, static FOS = 1.85	>1.5	7
	Dynamic Stability	With till exterior slope, pseudo-static FOS = 1.64	>1.15	3
Construction	Construction Method	Ascending placement on gentle slopes	Method V	8
	Loading Rate (1)	1 t/d/m	Very Low	7
Performance	Stability Performance	Assumed: Stable	Good	7.5
			Total	42

Note: 1) Mass loading rate assumes bulk density of 2.00 t/m3

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