Geotechnical Stability Analysis and Dam Breach Update
Abandoned Clinton Creek Asbestos Mine Site
Yukon



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EXECUTIVE SUMMARY

The subject project site is an abandoned asbestos mine located near Dawson City, Yukon Territory. The waste rock pile and tailings pile at the mine site have experienced slope failures in the past and blocked two water channels, Clinton Creek and Wolverine Creek, respectively. Both piles have undergone significant movement over the last 40 years, although the rate of movement over the past 10 years has greatly diminished. This report addresses the stability of both piles and evaluates the change in factors of safety for various closure options.

The movement pattern of the waste rock pile at Clinton Creek suggest the pile is creeping across the valley bottom. Among the five shortlisted closure options, one option removes the majority of the waste rock pile, and therefore eliminates the need to meet dam safety requirements. The other four options indicate minimal improvement in the overall stability of the waste rock pile and therefore do not satisfy dam safety requirements.

The existing tailings pile on the valley slope at Wolverine creek is considered marginally stable for cross valley movement. In the stream direction, the existing tailings pile at the valley bottom was reasonably stable. Among the seven closure options, only two will improve the stability of the tailings pile on the valley slope, and marginally satisfy the minimum requirement by the British Columbia Mine Waste Rock Pile Research Committee guideline (BCMWRPRC 1991). Under the design earthquake event (1:475 year return period), large slope movement can still be expected with these two options and a future seismic deformation analyses is warranted. Other options will not change the stability of the tailings pile on the valley slope. In the stream direction, stability analyses indicated that most options meet the BCMWRPRC minimum requirements, except for some localized surface failures near the downstream toe for two of the options.

The tailings at the valley bottom of Wolverine Creek is unlikely to be liquefiable based on its gradation. Three of the closure options reduce the liquefaction potential by lowering the water table in the tailings pile. However, further investigations and assessments are required to evaluate the liquefaction potential of the tailings.

It should be noted that the stability analyses were based on limited available information supplemented with many engineering assumptions. The analyses will need to be revised once further information is available from future geotechnical investigations and monitoring programs.

This report also includes a review of previous dam breach assessments. The purpose of this review was to consider if additional dam breach modeling would be useful to support project risk assessments or the selection and design of remedial works. Additional quantitative dam breach modeling would likely not alter the results of the current qualitative approach to risk assessment.

Numerical modeling techniques have improved significantly since 2002 when dam beach modeling was last performed for Clinton Creek. Improvements include 2-D modeling capabilities, the ability to explicitly simulate mudflows, and graphical output of inundation limits. Additional dam breach modeling of possible interest would be to explicitly simulate the transport and re-deposition of the landslide waste rock (and tailings) material that would wash downstream and be the major direct cause of environmental damage. However, unless consideration was to be given to the use of controlled (induced) breaches to remove the landslide material, there might be little practical value in performing this additional modeling.

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LIMITATIONS OF REPORT

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1.0 INTRODUCTION

Government of Yukon, Assessment and Abandoned Mines (AAM) have retained Tetra Tech EBA Inc. (Tetra Tech EBA) to assist with the summary and evaluation of geotechnical information related to closure of the Clinton Creek asbestos mine near Dawson City, Yukon. This report presents a review of the previous geotechnical analysis along with results of a geotechnical stability analysis undertaken by Tetra Tech EBA. A review of the previous dam breach study was conducted and is also discussed herein.

Tetra Tech EBA has been retained to provide engineering services for the above captioned project in an attempt to better understand the mechanisms driving the slope instability of the waste rock and tailings piles. The general scope of work includes summarizing existing information and data gaps, evaluating the data and monitoring programs, as well as assessing stability, site access mitigations, and fish passage for the various short-listed closure options. This work is being conducted to guide project parties in making decisions regarding future design and implementation of short-listed closure options.

The objectives of the stability analysis for the Clinton Creek waste rock pile include evaluating the stability of the existing waste rock pile and the effects of implementing a variety of closure options. For Wolverine Creek tailings pile, the stability analyses are to better estimate the strength parameters of the tailings and the underlying soils as well as to assess the effects of the proposed closure options. The stability analyses were carried out based on the historical information available along with some assumed parameters, as summarized in reports R124 and R126.

2.0 BACKGROUND INFORMATION

Table 2-1 presents the deliverables which have been submitted to AAM as part of this scope of work, and indicates a reference number for each report which will be used when referring to any of those documents in this report.

Table 2-1: Tetra Tech EBA Report Submissions to AAM

Report No.	Report Name	Status
R124	Existing Geotechnical Subsurface and Monitoring Data Summary Report	Issued for Use February 25, 2016
R125	Preliminary Dam Classification – Mine Waste Structures Memo	Issued for Use March 7, 2016
R126	Data Gap Assumption Report	Issued for Use March 31, 2016
R127	Correlation of Existing Movement Rates and Seasonality Report	Issued for Use March 24, 2016
R128	Geotechnical Stability Analysis Report and Dam Breach Update	Issued for Use March 31, 2016
R129	Geotechnical Monitoring Program Review Memo	Issued for Use March 31, 2016
R130	Geotechnical Investigations Program Scope of Work Memo	Issued for Use March 31, 2016
R131	Site Access Geotechnical Engineering Review and Mitigation Memo	Issued for Use March 31, 2016
R132	Fish Passage Memo	Issued for Use March 31, 2016

3.0 STABILITY ANALYSIS – WASTE ROCK PILE

The slope stability analyses were carried out using a 2-D limit equilibrium computer program ***Slope/W*** (GeoStudio 2012). The Morgenstern-Price method which satisfies both moment and force equilibrium was used for the factor of safety calculation. The location of the sections used in the stability analyses are presented on Figure 1.

For the waste rock pile at Clinton Creek, three sets of slope stability analyses were performed:

- Analysis of a typical section crossing the creek valley (Section B-B' in Figure 1) to back-analyze soil and pore pressure parameters in the underlying colluvium/alluvium layer;
- Stability analysis parallel to stream direction (Section F-F'); and
- Stability analyses for the five short listed closure options to evaluate the change in factor of safety.

3.1 Back Analyses

Back analyses were conducted along Section B-B', which is approximately perpendicular to the valley bottom and where predominant historical movement was observed. In this direction, the slope was considered to be marginally stable (i.e., may become unstable under minor disturbance) with a factor of safety near 1.0. Therefore, some of the soil and groundwater parameters can be refined from the back analyses to further analyze other sections and/or loading scenarios.

The slope profile and soil stratigraphy for Section B-B' (see stability output in Appendix B) were generated from the 2012 topographic survey, the 1949 digital elevation model, and the 2002 UMA report, Section K (R47). Without knowing the depth of colluvium/alluvium at the valley bottom and whether it is frozen or not, it is assumed that the possible failure surface was confined within the upper 8 m of this material. This assumption will result in lower colluvium/alluvium strength from the back analyses and is considered conservative for analyzing other scenarios.

On Section B-B', a phreatic surface at Elev. 405 m (approximately 6.5 m below the normal water level in Hudgeon Lake) was assumed in the waste rock. The assumed phreatic surface was estimated from the average hydraulic gradient through the waste rock pile and was about 3 to 4 m lower than the 1999 standpipe readings (R47). The higher water level recorded in these standpipes was likely due to their proximity to Clinton Creek and Hudgeon Lake while the assumed phreatic surface was considered more representative of areas further away from Clinton Creek and Hudgeon Lake.

Limited material properties are available from existing field and laboratory tests. The key unknowns in the stability analyses are the strength and pore pressure conditions in the colluvium/alluvium. Based on our review of air photos it has been identified that ice-rich permafrost conditions likely existed within the floodplain at the base of the valley prior to mine development, as discussed in Tetra Tech EBA's report R126. As the waste rock failure occurred over 40 years ago, it is possible that there has been some degradation of the permafrost, although this cannot be stated with certainty due to the limited drilling and instrumentation information available. When analyzing permafrost the conventional approach is to analyze the frozen soil using cohesion and no frictional properties. If the subsurface conditions indicate unfrozen ground, then an effective stress analysis is a more reasonable approach to adopt.

The initial assumption for the phreatic surface (as described above) was to assume an elevation of 405 m. It is possible that due to infiltration of rainwater and snow melt that the porewater pressures could be higher. As a result, additional analyses were undertaken which assumed a phreatic surface elevation of 430 and 455 m.

If an effective stress approach is taken, it is possible that drainage of the thawing ground may be impeded by the low permeability of the colluvium/alluvium (except for the thin layer near the waste rock colluvium contact) and higher than normal pore pressures (as discussed above) may exist within the colluvium/alluvium. These excess porewater pressures have been evaluated by applying a pore pressure coefficient ($\bar{B}$) to the underlying thawing alluvium.

To estimate these parameters, the back analyses were performed using the following two approaches:

- A total stress analysis (assuming the ground is frozen) assigning a constant undrained shear strength to colluvium/alluvium to obtain a factor of safety of 1.0; and

- Effective stress analyses using various combinations of friction angle (residual) and porewater pressure to obtain a factor of safety of 1.0.

The results of the analyses are presented in Appendix B. The soil and pore pressure parameters used in the back analyses are summarized in Table 3-1. The material unit weights used in the analyses were adopted from the data gap report (R126). The internal friction angles for the waste rock and weathered argillite are based on previous laboratory testing (R124).

Table 3-1: Soil and Pore Pressure Parameters Adopted in Back Analyses

Soil Type	Unit Weight (kN/m^3)	Internal Friction Angle (°)	Pore Pressure Coefficient $\bar{B}$	Piezometric Level (m)	Undrained Shear Strength (kPa)
Waste Rock	19.6	23	-	-	-
Weathered Argillite	21.6	27	-	-	-
Colluvium/Alluvium (total stress approach)	19.1	-	-	-	60
Colluvium/Alluvium (assumed fully consolidated)	19.1	4 6.4 9.4	N/A	405 430 455	N/A
Colluvium/Alluvium (effective stress approach)	19.1	12 15 17.5	0.85 0.92 0.95	-	-

It should be noted that residual friction angles were assigned to waste rock and colluvium/alluvium due to the large movements that have occurred in the past. The friction angle of weathered argillite presented in Table 3-1 was the peak friction angle from the two existing direct shear test results. The residual friction angle of this material is expected to be lower, but may not be much less than that of waste rock based on its relatively low fines content (8 to 15%). In other words, the critical failure surface is expected to be through the waste rock or waste rock/argillite interface and will not be influenced by the strength of the weathered argillite.

The total stress analysis indicated that an undrained shear strength of 60 kPa is required in the colluvium/alluvium to achieve a factor of safety of 1.0 (Figure B.1 in Appendix B). Assuming the colluvium/alluvium has been thawed and undergone some consolidation, this strength corresponds to a cohesive soil that has undergone approximately 20 to 30% consolidation. Assuming the colluvium/alluvium is still frozen, this strength corresponds to frozen soil at a temperature of about -1°C.

Assuming the colluvium/alluvium is thawed and completely consolidated, a phreatic surface elevation of 405 m yielded an unrealistic low friction angle of $\varphi' = 4°$ when back-calculated (Figure B.2 in Appendix B). Increasing the elevation of the phreatic surface by 25 m resulted in a back-calculated friction angle of $\varphi' = 6.4°$ (Figure B.3). Increasing the phreatic surface by another 25 m, (which puts the groundwater table at the surface of the waste pile) resulted in a back-calculated friction angle of $\varphi' = 9.4°$ (Figure B.4).

To analyze the possibility of artesian conditions within the alluvium, a relatively high excess pore pressure $\bar{B} = 0.85$ to 0.95 was assumed in the analysis. These results indicated that a residual friction angle ($\varphi_r' = 12$ to 17.5°) in the colluvium/alluvium is required to achieve a factor of safety of unity (Figures B.5 to B.7 in Appendix B). The alluvium is expected to comprise primarily silt and sand. The friction angles back-analyzed from the analyses are considered to be very low for this type of soil. Also the pore pressures necessary to achieve the $\bar{B}$ values indicated are not considered realistic. Without further investigation and testing, these values cannot be justified with confidence. Therefore, assuming that the colluvium/alluvium underlying the waste rock pile is characterized by relatively warm permafrost, only a total stress approach (Su = 60 kPa) was adopted for further analyses.

3.2 Stability Analyses for Existing Waste Rock Pile

3.2.1 Standards and Criteria

Since the waste rock pile was classified as a "Significant" dam structure (see R125), it is necessary to meet the requirements specified in the following:

- Dam Safety Guidelines (Canadian Dam Association 2013);
- Geotechnical Considerations for Dam Safety (CDA 2007); and
- Application of Dam Safety Guidelines to Mining Dams (CDA 2014).

For the waste rock pile at the "Closure" phase in terms of dam life, the various scenarios and corresponding factors of safety presented in Table 3-2 are considered applicable for stability analyses. Post-Earthquake analysis was not performed as the state of the colluvium/alluvium (i.e., frozen or thawed) and its liquefaction potential could not be well identified at this time. The analyses were carried out for both Section F-F' (parallel with stream direction) and Section B-B' (across stream direction), although analyzing Section B-B' is not required for typical man-made dams.

Table 3-2: Scenarios and Required Minimum Factors of Safety (CDA 2014)

Loading Condition	Minimum Factor of Safety
Steady State Seepage	1.5
Pseudo-Static Seismic (1:2,475 year return period)	1.0

3.2.2 Steady State Seepage

For steady state seepage, the soil parameters obtained from back analyses presented in Table 3-1 were used. The groundwater in the waste rock was assumed to flow at a constant gradient from the normal lake level to the toe of the slope.

3.2.3 Pseudo-Static Seismic Analyses

For seismic analysis using the pseudo-static method, a Peak Ground Acceleration (PGA) of 0.308 g was adopted for the subject site. This value was obtained from the Geological Survey of Canada (GSC) website and was adjusted for the expected site condition (i.e., Site Class D instead of Class C) according to the 2015 National Building Code of Canada.

According to CDA (2007), the Hynes-Griffin and Franklin (1984) method was adopted for the pseudo-static analyses. For this method, a horizontal seismic coefficient (k_h) equal to 50% of the near-surface PGA (i.e., $k_h = 0.154$) together with a 20% strength reduction in all strength properties is recommended. The soil parameters used for pseudo-static seismic analyses are presented in Table 3-3.

Table 3-3: Soil Parameters Used in Pseudo-Static Seismic Stability Analysis

Soil Type	Unit Weight (kN/m^3)	Internal Friction Angle (°)	Undrained Shear Strength (kPa)
Waste Rock	19.6	18.8	-
Colluvium/Alluvium	19.1	-	48

3.2.4 Results

The results of the stability analyses for the existing waste rock pile are summarized in Table 3-4. Individual plots of the stability results are presented in Appendix B.

Table 3-4: Stability Analysis Results for Existing Waste Rock Pile

Loading Condition	Section	Calculated Factor of Safety	Figure
Steady State Seepage	B-B'	1.0	B.1
	F-F'	1.10 (1.21)	B.9
Pseudo-Static Seismic (1:2,475 year return period)	B-B'	0.35	B.8
	F-F'	0.24	B.10

Note: Numbers in brackets are factor of safety for failure surface through dam crest

The analyses indicate that the analyzed cases of the existing waste rock pile do not meet the minimum CDA factor of safety requirements in either direction for all scenarios. It should be noted that actual factors of safety for the waste rock pile in the stream direction (Section F-F') may be much higher than the calculated values due to its long and narrow shape. Both sides of the waste rock pile are confined by the two valley slopes and a significant 3-D effect (i.e., 15 to 50% increase in factor of safety) can be expected (Stark 2003). In a 2-D stability analyses, the resistance on both ends of the slip surface (in the direction perpendicular to the slip surface) is ignored. This may be adequate for a wide slope but conservative for a narrow slope. The 3-D effect in slope stability analyses takes into consideration the resistance on both ends of the slip surface. Such 3-D effect can only be properly evaluated once more information is available on subsurface conditions using a 3-D stability analysis program.

The low factors of safety calculated from the pseudo-static seismic analyses indicate that the waste rock pile would either fail or undergo very large deformation (in the order of tens of metres) during the design earthquake event. For such low factors of safety, seismic deformation analysis using Newmark's type approach is not expected to yield any credible result.

3.3 Stability Analyses for Closure Options

For the Clinton Creek waste rock pile, five closure options were short listed as shown in Table 3-5. Detailed descriptions of the closure options are presented in the lifecycle cost analysis by WorleyParsons (R56) and the technical options assessment by AECOM (R97).

Table 3-5: Clinton Creek Closure Options Summary

Closure Option	Clinton Creek Channel	Clinton Creek Waste Rock Pile	Hudgeon Lake
C3	Armour channel	No remediation	No remediation
D3	Armour channel	Excavate upper waste rock pile	No remediation
E3	Armour and Lower channel	Excavate upper waste rock pile	Lower lake level
F	Restore channel to valley floor	Remove waste rock to expose valley floor	Fully drain lake
I2	Armour channel in new alignment over waste rock	Excavate upper waste rock pile	No remediation

Stability analyses for the short-listed closure options were carried out in a similar manner as described in Section 3.2. These analyses all assumed an ice-rich frozen layer in the upper alluvium with a cohesion of 60 kPa. The results for the various stability analyses along Section B-B' are presented in Appendix C and are summarized in Table 3-6. Other than Options E3 and F, the remaining options have negligible influences on the overall stability of the waste rock pile. Option E3 improves the factor of safety from 1.0 to 1.13 under steady state seepage (may take a few years to reach); however, it is still well below the required minimum of 1.5. The reduced factor of safety shown for Option F is due to the combination of the residual friction angle, low shear strength mobilized in the frozen alluvium, and the steep slope face (2.5H:1V) of the waste rock. Such a cut slope is considered reasonable if the waste rock is underlain by competent subgrade. With the presence of relatively weak colluvium/alluvium, a slope angle of 6H:1V is required to obtain a factor of safety of 1.0.

Table 3-6: Summary of Factors of Safety (Section B-B') for Closure Options

Remediation Options	Steady State Seepage		Pseudo-Static Seismic	
	Factor of Safety	Figure	Factor of Safety	Figure
Existing Condition	1.0	B.1	0.35	B.8
Option C3	0.96	C.1	0.27	C.2
Option D3	1.01	C.3	0.31	C.4
Option E3	1.13	C.5	0.32	C.6
Option F	0.71	C.7	0.29	C.8
Option I2	1.0	C.9	0.37	C.10

Analyzing a section parallel to the stream direction (Section F-F'), only Option E3 has a marginal increase on the waste rock pile stability (see Table 3-7). Options C3, D3, I2 do not alter the geometry and groundwater conditions along Section F-F'; therefore, do not change the stability of the existing waste rock pile. By draining Hudgeon Lake completely, Option F disqualifies the waste rock pile as a "significant dam"; therefore, the stability in this direction is no longer a concern. Furthermore Section F-F' runs along the valley bottom and the lower slope of the re-shaped waste rock pile and does not represent the general profile of Option F in this direction. Analyzing another section further south which is more representative of Option F is not warranted at this stage since the proposed 2.5H:1V cut slope for Option F will likely need to be flattened as discussed above, which may extend the crest of the cut slope to the existing valley slope.

Table 3-7: Summary of Factors of Safety (Section F-F') for Closure Options

Remediation Options	Steady State Seepage		Pseudo-Static Seismic	
	Factor of Safety	Figure	Factor of Safety	Figure
Existing Condition / Options C3, D3, I2	1.10	B.9	0.24	B.10
Option E3	1.12	C.11	0.25	C.12

3.4 Liquefaction Susceptibility

The liquefaction susceptibility of soils is typically assessed using a simplified method which requires penetration test results or shear wave velocities to determine the resistance to liquefaction. In the absence of such information, the liquefaction susceptibility of saturated deposits was qualitatively assessed based on gradation tests, Atterberg Limits, age, and soil descriptions. This approach is suitable as a screening level assessment but not intended to replace the state-of-practice liquefaction triggering assessment.

The following factors were considered in the qualitative liquefaction assessment for the colluvium/alluvium:

- **Saturation** – The granular soils must be saturated to be susceptible to liquefaction. If the upper colluvium/alluvium is thawed, it is likely to be saturated. A few borehole logs indicate the colluvium/alluvium is frozen, which will increase the liquefaction resistance. However, the existing information is insufficient to rule out the presence of thawed/unfrozen colluvium/alluvium.

- **Gradation** – Figure 2 shows the gradation boundaries of most liquefiable and potentially liquefiable soils (after Tsuchida 1970). The gradation curves obtained from tests conducted on six colluvium/alluvium samples at the site are also plotted for comparison (red lines). The plot indicates that four out of the six samples have gradations that fall within the boundaries of potentially liquefiable soils, even though the finer portion may be slightly outside. Based on this, it is not possible to eliminate the liquefaction potential for this material.

- **Relative Density (or Penetration Resistance)** – The liquefaction resistance of a granular soil is related to its relative density, which is typically estimated from penetration tests. Irrespective of the intensity of the seismic event, a soil with Standard Penetration Test (SPT) blow counts, $(N_1)_{60,CS} > 30$ or normalized corrected Cone Penetration Test (CPT) tip resistance, $q_{C,1N,CS} > 170$ is generally considered unlikely to undergo liquefaction or significant strength reduction (Idriss and Boulanger 2008). Note that, with relatively moderate levels of seismicity, a lower penetration resistance may be sufficient to resist liquefaction at this site. The consistency of colluvium/alluvium material is described as "compact" in one testhole log, which indicates that the penetration resistance is smaller than those limits given for non-liquefiable soils.

- **Fines Content** – In comparison to a clean granular soil, the resistance to liquefaction increases with increasing fines content. It is also known from cyclic tests performed on clayey soils that fines with higher plasticity further increases the liquefaction resistance, and fines with Plasticity Index (PI) greater than 15 are generally considered non-liquefiable (Gratchev et al. 2006). On average, the measured fines content in colluvium/alluvium is about 45% but four out of the six soil samples indicated fines content less than 30%. Atterberg Limits tests performed on the finer fraction showed a relatively low PI of 8.6%. Although some increase in liquefaction resistance is expected due to relatively high fines content, without knowing the cyclic resistance of an equivalent clean sand, it is not possible to eliminate the risk of liquefaction on the grounds of fines content and its characteristics.

- **Age** – The susceptibility of sand-like soils to cyclic liquefaction depends on the geologic age of the soil deposit (Andrus et al. 2009). Holocene and Pleistocene epochs formed during the Quaternary period (less than 1.6 million years ago) are found to be moderately to very highly susceptible to liquefaction. Therefore, the recent colluvium/alluvium deposits could be at high risk of liquefaction, depending on the in situ relative density.

The above qualitative assessment indicates that there is insufficient reliable data to eliminate the possibility of liquefaction of the colluvium/alluvium deposits. Therefore, in this preliminary study, this soil unit should be considered as susceptible to liquefaction until further studies and explorations are conducted to determine the groundwater regime, in situ density, and liquefaction resistance of these materials.

The waste rock is unlikely to be susceptible to liquefaction due to its gradation and the relatively low groundwater level.

4.0 STABILITY ANALYSES – TAILINGS PILE

Based on the contour drawing (R56) and a normal water level of about Elev. 408 m (R120), the volume of water retained by the north lobe of the tailings pile was estimated to be about 80,000 m^3, which exceeds the limit for CDA dam definition (over 2.5 m high and over 30,000 m^3 fluid). As a result, the existing tailings pile should be treated as a dam. However, all closure options except for Option C disqualify the tailings pile as a dam by either permanently lowering the creek level or displacing the ponded water with waste rock fill. Therefore, when analyzing the tailings stability for the closure options, it is rational to categorize the tailings pile as a mine waste pile and follow the waste pile guidelines (BCMWRPRC 1991). For the tailings pile, the guideline recommends:

- A minimum factor of safety of 1.3 under long term operation (steady state seepage);and
- A minimum factor of safety of 1.0 under 1:475 year return period earthquake event using the pseudo-static method.

For the stabilization of the tailings pile at Wolverine Creek, two sets of slope stability analyses were performed:

- Back analyses of two sections crossing the creek valley (Section D-D’ and E-E’ on Figure 1) to refine soil and pore pressure parameters in the colluvium and tailings layers; and
- Stability analyses for the existing conditions and closure options to evaluate the change in factor of safety after implementation of possible closure options.

4.1 Back Analysis

Back-analyses were conducted along Section D-D’ (south lobe) and Section E-E’ (north lobe) where predominant historical movement has been observed. The slope was considered to be marginally stable with a factor of safety near 1.0. Similar to Clinton Creek, the slope profile and soil stratigraphy for Wolverine Creek were generated from the 2012 topographic survey, the 1949 digital elevation model, and the 2002 UMA report (R47).

Considering the uncertainty with the failure mechanics, soil strength, and porewater pressures, etc., the back analyses were performed in the following sequence:

- Back-analyze the internal friction angle of the tailings from a steep segment of the north lobe (Section E-E’). The slope angle over this steep section is considered to be the angle of repose of the tailings (Figure D.1 in Appendix D); and

- Back-analyze the residual friction angle of the colluvium by modeling the observed lower slope zone of movement for both south lobe and north lobe (Figures D.2 and D.3 in Appendix D).

In the back-analyses, the underlying argillite and weathered argillite was assumed to be impenetrable and not influencing the factor of safety calculation. No porewater pressure was assigned to any soil strata assuming that the permanent groundwater level on the slope is below the colluvium and any water from infiltration will be dissipated quickly. Such assumption yields lower soil strength parameters and is considered conservative for assessing the closure options.

The results of the back-analyses are presented in Appendix D. The back-calculated factor of safety for the north lobe (Figure D.3) is slightly higher than that for the south lobe (Figure D.2), which is in agreement with the different observed movement rates. The soil parameters used in the back-analyses are summarized in the following Table 4-1. Soil unit weights are adopted from the data gap report (R126).

Table 4-1: Soil Parameters Calculated from Back Analyses

Soil Type	Unit Weight (kN/m^3)	Internal Friction Angle (°)	Cohesion (kPa)
Tailings	18.1	26	0
Colluvium	19.1	17.5	0

Based on the existing borehole data, topsoil and organics were present at the interface of the tailings and the underlying colluvium. The back calculated friction angle of 17.5° in the colluvium could either be the residual friction angle in this material or the friction between the tailings and the topsoil. In other words, the potential failure surface could be at the tailings and topsoil interface or even the active layer (annual freeze/thaw layer above the permafrost) interface rather than through the colluvium.

4.2 Analyses of Closure Options

Seven closure options were considered in WorleyParsons life cycle cost analysis (R56) for the Wolverine Creek tailings pile, including a no-action option (Option A). A summary of these options is presented in Table 4-2. Detailed descriptions and drawings for each option can be found in the life cycle cost analysis report (R56).

Table 4-2: Wolverine Creek Tailings Pile Closure Options Summary

Closure Option	Description
A	No remediation.
B	Rock drain along toe of tailings lobe(s).
C	Armoured channel through cover to existing rock channel; provide cover over tailings pile base.
D	Rock drain along toe of tailings lobe(s), armoured channel through cover to existing rock channel; provide cover over tailings pile base.
E	Rock drain along toe of tailings lobe(s), armoured channel through cover to existing rock channel, stabilize tailings and provide cover and armouring.
D2	Same as Option D but lowering the proposed waste rock cover over the tailings pile base from 422 m to 415 m, eliminating the rock drain, backfilling Wolverine Creek upstream with waste rock to 412 m to avoid a dam classification.
E2	Same as Option E but eliminating the rock drain, backfilling Wolverine Creek upstream with tailings (and waste rock cover) to 419 m to avoid a dam classification.

Among the seven closure options presented, only Options E and E2 are considered to have a positive influence on the overall stability of the tailings pile on the valley slope (Sections D-D' and E-E'). Option A which assumes no remediation will not change the stability of the existing conditions. Other options which introduce remedial measures below Elev. 422 m will not improve the overall stability as the back analyses indicated that the failure surface would likely exit above this elevation. Slope movement observed below this elevation may be a consequence of upper slope movement, which imposes driving forces to soils below Elev. 422 m. However, this cannot be properly simulated in the limit equilibrium analyses.

Parallel to stream direction (Section G-G'), all closure options except for Option A will have an influence on the stability of the downstream slope at the valley bottom. The upstream slope is of no significance to the overall performance of the tailings pile and was not analyzed.

Soil parameters used in the analyses were adopted from the back analysis presented in Table 4-1. The compacted waste rock and compacted tailings specified in some closure options are expected to be track-packed and slightly higher densities and friction angles were assigned to these materials as shown in Table 4-3. In analyzing the stability parallel to the stream direction, the soil at the valley bottom was assumed to be predominantly colluvium approximately 8 m thick. In pseudo-static analyses, a horizontal seismic coefficient $k_h = 0.06$ was adopted for the tailings pile, which is 50% of the PGA (0.12g) for the 1:475 year return period earthquake.

Table 4-3: Additional Soil Parameters Used for Wolverine Creek

Soil Type	Unit Weight (kN/m^3)	Friction Angle (°)	Cohesion (kPa)
Compacted Tailings	19.0	32	0
Compacted Waste Rock	20.0	34	0

The results of the stability analyses are presented in Appendix D. For Sections D-D' and E-E', the calculated factors of safety are summarized in Table 4-4. For Section G-G', the calculated factors of safety are summarized in Table 4-5.

Table 4-4: Stability Summary for Option E/E2 (Section D-D' and E-E')

Loading Condition	Section	Steady State Seepage		Pseudo-Static Seismic (1:475 year)	
		Factor of Safety	Figure	Factor of Safety	Figure
Existing Condition	North Lobe (Section E-E')	1.05	D.3	0.70	D.4
	South Lobe (Section D-D')	1.01	D.2	0.67	D.5
Option E/E2	North Lobe (Section E-E')	1.30	D.6	0.85	D.7
	South Lobe (Section D-D')	1.31	D.8	0.85	D.9

Table 4-5: Closure Option Summary (Section G-G')

Loading Condition	Steady State Seepage		Pseudo-Static Seismic (1:475 year)	
	Factor of Safety	Figure	Factor of Safety	Figure
Existing Condition/Option A	1.83	D.10	1.03	D.11
Option B	2.00	D.12	1.12	D.13
Option C	1.25 (1.35)	D.14	0.71 (0.75)	D.15
Option D	1.93	D.16	1.10	D.17
Option E	1.95	D.18	1.10	D.19
Option D2	1.56	D.20	0.82 (0.92)	D.21
Option E2	1.20 (1.32)	D.22	0.69 (0.73)	D.23

Note: numbers in brackets are for failure surface through downstream crest

Along the valley slope (Section D-D' and E-E'), analyses show that closure options E/E2 improve the factor of safety from about 1.0 to 1.30 under steady state seepage (long-term condition), which meet the required minimum of 1.3 specified in the BCMWRPRC guideline (1991). Using the pseudo-static method, the calculated factors of safety under design earthquake event is below unity, which may indicate significant slope movement. Again, seismic deformation analysis will be required in the future studies.

In the stream direction, the calculated factors of safety for a sizeable slope failure (i.e., failure surface through the downstream crest) for all closure options meet the requirements of BCMWRPRC (1991) guidelines under the steady state seepage condition. The slightly lower than required factors of safety for Options C and E2 corresponded to localized slip surfaces at the downstream toe, which may not impact the overall performance of the tailings pile. Again, a few lower than unity factors of safety using the pseudo-static method may indicate some significant slope movement (metres to tens of metres) and warrant future seismic deformation analysis.

4.3 Liquefaction Potential of Tailings

Gradations of the seven tailings samples from existing investigations are plotted on Figure 3 (red lines) against the Tsuchida (1970) liquefiable boundaries. The plot indicates that all tailings samples are considerably coarser than the upper limit of the potentially liquefiable soil. Based on this, the tailings are unlikely to be liquefiable. In addition, closure Options B, D, and E further reduce the liquefaction potential by lowering the water table in the tailings pile using a sizeable embedded rock drain.

It should be noted that Tsuchida's work was conducted in the late 1960's when liquefaction was not well studied, and no similar approaches have been undertaken ever since. Therefore, simply assessing the liquefaction potential using the gradation limit only may not be sufficient. As a result, additional investigations are recommended to evaluate the liquefaction potential of the tailings.

5.0 DISCUSSION

5.1 Existing Waste Rock Pile

The analyses indicate that the stability of the existing waste rock pile does not meet the CDA requirement in both directions (either cross valley or parallel to the creek). As mentioned in Section 3.2.4, the actual factor of safety in the stream direction (Section F-F') may be much higher (i.e., 1.4 or higher) due to 3-D effect. In the direction across the valley (Section B-B'), the slope is confined by both valley walls and large continuous slope movement is unlikely due to the kinematic restrains.

5.2 Closure Options

Based on the analyses, Closure Options C3, D3, and I2 have very little influence on the overall stability of the waste rock pile. Option E3 has some improvement on the stability in the B-B' direction; however, is still insufficient to meet the CDA criteria.

Option F, which disqualifies the waste rock pile as a "significant dam", eliminates the stability issue parallel to the stream direction (F-F'). Across the stream direction (B-B'), the presence of weak colluvium/alluvium foundation may require a rather flat (6H:1V or flatter) slope face in the waste rock to keep it stable.

5.3 Liquefaction and Seismic Stability

The waste rock is unlikely to be susceptible to liquefaction due to its gradation and the relatively low groundwater level (estimated to be greater than 20 m below existing grade). With limited information available at the current stage, we cannot rule out the liquefaction potential of the colluvium/alluvium. Further investigations and analyses will be required to determine the liquefaction potential of the colluvium/alluvium and the post-earthquake strength, etc.

The pseudo-static stability analyses carried out resulted in low factors of safety, which is partially because the slopes are marginally stable under normal condition. The results indicate that during the design earthquake event the slope will either completely fail or undergo significant deformation (tens of metres). A detailed seismic deformation analysis is not warranted at this time.

5.4 Wolverine Creek

The existing tailings pile at the valley bottom should be classified as a dam since the volume of water retained by the by the north lobe exceeds the limit for CDA dam definition (2.5 m high and 30,000 m^3 volume). However, all closure options except for Option C disqualify the tailings pile as a dam by either permanently lowering the creek level or displacing the ponded water with waste rock fill. Therefore, the tailings piles were deemed a mine waste pile and stability analyses for the closure options were carried out in accordance with the BCMWRPRC guidelines (1991).

Based on the observed continuous surficial movement, the existing slope of the tailings pile at Wolverine creek is considered marginally stable with a factor of safety close to 1.0 along the valley slope (Section D-D' and E-E'). However, the failure mechanism is not clear and the potential failure surface could be either through the colluvium or along the tailings and topsoil interface, or the interface of the active layer. The existing slope is stable in the stream direction with a factor of safety greater than 1.5.

Among the seven closure options, Options E/E2 will improve the stability of the tailings pile on the valley slope, increasing the factor of safety from 1.0 to about 1.30, which meet the required minimum of 1.3 specified in the BCMWRPRC guideline (1991). Under the design earthquake event (1:475 year return period), the calculated factors of safety were less than unity using the pseudo-static method. This is an indication of large slope movement under the earthquake event and warrants a future seismic deformation analyses. Other options will not change the stability of the tailings pile on the valley slope.

In the stream direction, stability analyses showed that most options meet the BCMWRPRC minimum requirements, except for some localized failure surfaces near the downstream toe for Options C and E2. However, it should be recognized that the any remediation measures undertaken at the valley bottom will be affected by the tailings pile on the valley slope if it becomes unstable.

The tailings at the valley bottom is unlikely to be liquefiable based solely on gradation. Closure options B, D, and E reduce the liquefaction potential by lowering the water table in the tailings pile. However, further investigations and assessments are recommended to qualitatively evaluate the liquefaction potential of the tailings.

5.5 Limitations and Future Analyses

It should be noted that the stability analyses were based on limited available information supplemented with many engineering assumptions. Major unknowns that affect the stability results include the failure surface, the depth and spatial extent of colluvium/alluvium, the state of colluvium/alluvium (frozen or thawed), its strength, liquefaction potential, and spatial variabilities, etc. Future geotechnical investigation and monitoring will need to focus on identifying the failure surface, the subsurface stratigraphy, the state of the material (frozen/thawed), the liquefaction potential (if applicable), the groundwater conditions, and the soil properties, etc. Detailed recommendations for additional investigation and monitoring are provided under separate covers (R129 and R130).

A 3-D stability analysis will likely be required when more information is obtained from the future investigation and monitoring program unless Option F is selected. To support the stability study, a liquefaction assessment, seepage analysis, and thermal analysis may also be needed.

Based on the results of these analyses, it appears that there will be limited benefit with any of the closure options proposed other than Option F for the Clinton Creek. Even if one of the other options was implemented, it should be taken into consideration that the waste rock pile should be classified as a dam and in accordance with the Dam Safety Guidelines subject to a formal review and annual monitoring.

For Wolverine Creek, stability analyses indicated that Options E and E2 may improve the stability of the tailings piles on the valley slope to meet the BCMWRPRC requirements. Further investigations are recommended to refine the stability analyses.

6.0 DAM BREACH REVIEW

The scope of work for the present assignment included a review of the existing Clinton Creek dam breach study and associated data. The purpose of this review was to determine if additional work is needed to update the previous study to support the present analysis of the progressive movement and overall failures of both the waste rock and tailings piles.

6.1 Relevant Prior Reports

A screening review of the AAM project files identified the following two documents that include assessments of dam breach scenarios:

- AAM Report # R40 – Abandoned Clinton Creek Asbestos Mine Risk Assessment Report, April 2000. Prepared by UMA Engineering Ltd., Winnipeg office, for Indian and Northern Affairs Canada.
- AAM Report # R116 – Clinton Creek Engineering Review and Assessment, Part 1, 31 March 2015. Prepared by WorleyParsons and EcoNomics for Government of Yukon Energy, Mines and Resources – Assessment and Abandoned Mines.

The UMA report prepared in 2000 was prior to the construction of gabion drop structures on Clinton Creek below the Hudgeon Lake outlet. It documented the progressive stream erosion of waste rock landslide materials in the stream channel and noted that this progressive incising had increased the likelihood of the development of a full breach of the waste rock material. Using the FLDWAV model to assess dam breach and downstream inundation, UMA performed several dam breach scenarios for the waste rock landslide blockage of Clinton Creek, and a single dam breach scenario for the tailings landslide blockage of Wolverine Creek.

For Clinton Creek, a full breach scenario would involve the mobilization of landslide deposits that are 25 m deep at the lake outlet, plus about 12 million cubic metres of water now impounded in Hudgeon Lake behind the landslide blockage. With respect to the tailings landslide blockage of Wolverine Creek, UMA evaluated a single dam breach scenario involving the mobilization of tailings deposits approximately 12 m deep and releasing about 1 million cubic metres of impounded water.

The UMA 2000 report index lists 13 drawings including location plans, creek plans and profiles, tailings plans and profiles, assumed breach geometry, downstream land use occupancy, and flood hazard mapping. These drawings were not included in the documents initially received, and it was confirmed during the course of the review that the drawings were also missing from AAM's hard copy library. Copies of the drawings were subsequently located in a different government library, but due to schedule constraints to complete this report, were not reviewed in detail as part of the present assignment.

The WorleyParsons report prepared in 2015 was after the flood event in August 2010 which severely damaged the furthest downstream of four gabion drop structures constructed from 2002 to 2004 on Clinton Creek below the Hudgeon Lake outlet. This same drop structure, identified as DS4, experienced prior significant damage during the 2009 spring runoff event. The report assessed two failure scenarios for Clinton Creek: (1) the complete failure of DS4 alone, and (2) a cascading failure of all stabilization works, resulting in a full breach condition similar to scenarios considered by UMA.

The WorleyParsons report assessed the two Clinton Creek failure scenarios using a risk assessment format obtained from Aboriginal Affairs and Northern Development Canada (AANDC). This format does not involve quantitative evaluations of breach scenarios and inundation mapping. Instead, qualitative consequence classifications were provided based on anticipated environmental impact, legal obligations, costs, community reputation, human health, and other considerations.

6.2 Review Findings

The present review focussed on the UMA 2000 report which provides a quantitative assessment of dam breach scenarios. The objective of the review is to consider whether there is merit in performing additional or updated dam breach assessment work to better understand the impact of continuing failure modes or the design of remedial works.

The impacts of past and future failures of the landslide blockages on Clinton and Wolverine Creeks, although potentially severe, will be constrained by limited human occupation of this remote site. The geographic extent of adverse impacts is expected to be limited to identifiable segments of Clinton, Wolverine, and Forty Mile Creek upstream of the Yukon River. Additional or updated breach analyses and inundation mapping is unlikely to significantly alter or inform qualitative consequence classifications such as presented in the 2015 WorleyParsons report.

The conditions that would result in a worst-case (full breach) scenario for Clinton Creek are more or less unchanged from those that were assumed in the 2000 UMA study. The scenario involves progressive downcutting and erosion of the waste rock material that now occupies the downstream channel, over an unspecified period of time, followed by a rapid massive breach of the remaining deep fill at the Hudgeon Lake outlet.

Reasons in favour of updating the prior dam breach study include:

- Preparation of flood inundation mapping using current modeling techniques that will update the original drawings;
- Validation/verification of original model results; and
- Explicit simulation of sediment/mudflow aspects of a breach, which may be of greater consequence than the clear water simulations that were performed.

The importance of the flood inundation mapping in assessing dam consequence classifications and design criteria for remedial works is unknown. If decisions for remediation efforts are driven mostly by qualitative consequence considerations such as assessed in the WorleyParsons 2015 report, there may be little benefit in developing updated mapping to inform the decision making process.

The UMA 2000 report presents a thorough assessment of dam breach scenarios, but review was hampered by the initial lack of drawings and access to detailed model results. It was noticed that the full breach scenario for Clinton Creek seemed peculiar in that the breach flood wave is substantially attenuated within 2 km downstream of the dam. A preliminary review of these model results was initiated with a 2-dimensional model, Flo-2D, set up with readily available 10-m contour information and the breach hydrograph presented by UMA report. This review was not completed because of gaps in the contour data which could not be resolved within the present scope. Also, the volume of the flood hydrograph which was manually digitized from a plot in the UMA report could not be reconciled with the volume that would be associated with the draining of Hudgeon Lake. Additional review or re-modeling would be required to validate or challenge the UMA results for Clinton Creek.

Numerical modeling techniques have improved significantly since FLDWAV, particularly with respect to 2-D modeling capabilities, user interface, and graphical outputs. Also, existing models such as Flo-2D provide the ability to explicitly simulate mudflows. Past analyses of the dam breach scenarios for Clinton Creek have assessed clear water conditions, in part because of model limitations. A major consequence of a full breach of the Hudgeon Lake outlet to Clinton Creek would be the flow and deposition of waste rock debris into the downstream channel. If the UMA breach analysis is correct, the rapid attenuation of the flood wave within 2 km of the Hudgeon Lake outlet would presumably be accompanied by the re-deposition of the waste rock material at that downstream location. This, in turn, could create a new blockage with a new lake at the downstream location.

The consequences of a dam breach on Clinton Creek cannot be accurately characterized without explicit consideration of the fate of the breached and re-located waste rock material. However, it is a judgement call by others on whether this would provide useful input to the assessment of the dam failure consequence classification or the design of remedial works. Additional breach modeling including mud-flow and sediment re-deposition could be a very useful tool to assess the consequence of sporadic minor failures, such as occurred in August 2010. Modeling could similarly be used to assess scenarios of using controlled (induced) breaches to lower and remove the existing waste rock blockage at the outlet of Hudgeon Lake.

7.0 CLOSURE

We trust this report meets your present requirements. If you have any questions or comments, please contact the undersigned.

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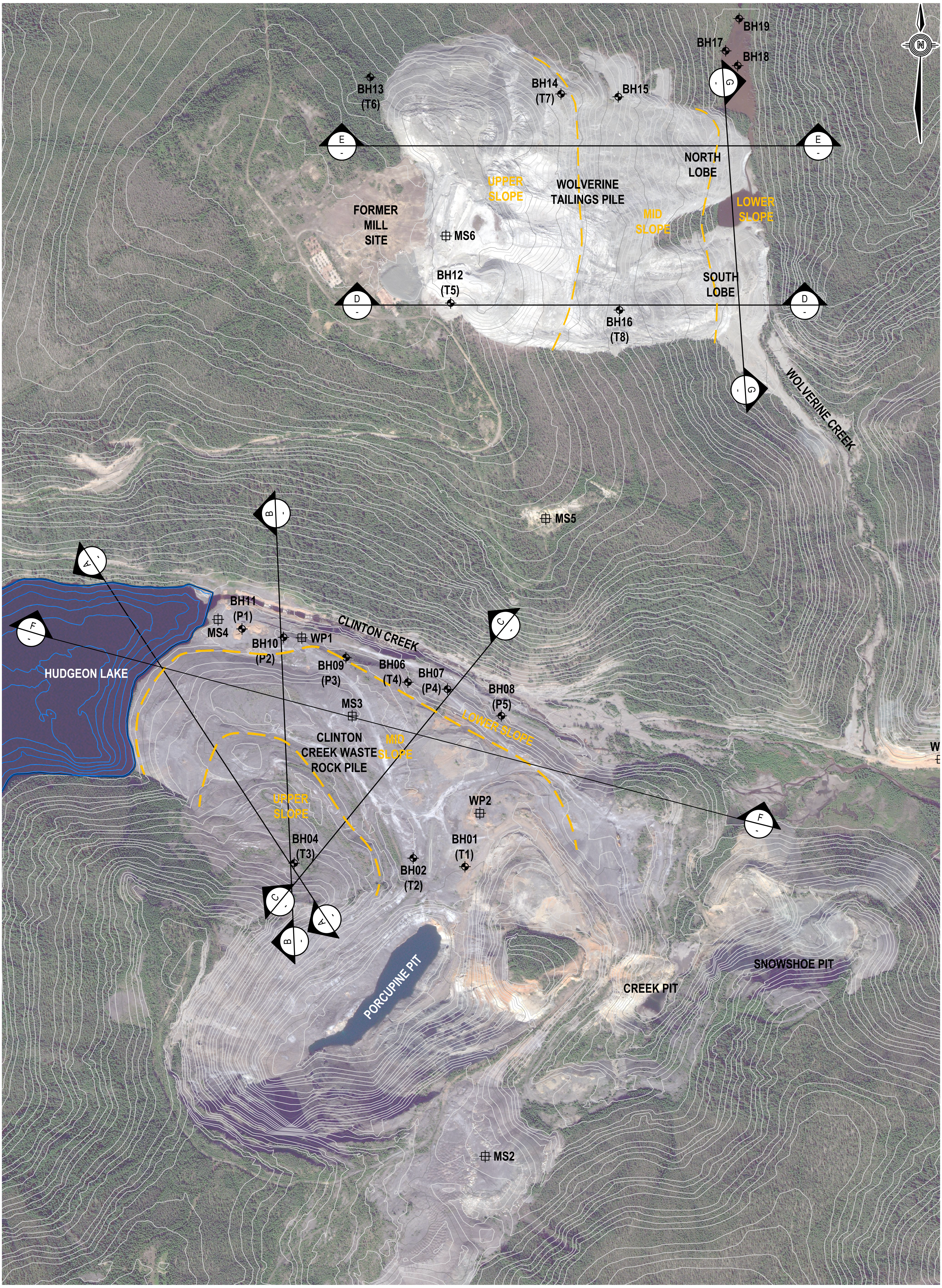
FIGURES

Q:\Kelowna\Drafting\ENG.WARC\ENGWARC03039-01 Abandoned Clinton Creek Mine Geotech Evaluation\Civil 3D\Production Drawings\Stability Analysis Report Figures\ENGWARC03039-01 Fig.1-R0.dwg [FIGURE 1] March 30, 2016 - 12:54:36 pm (BY: HALLEY, RICHARD)



NOTES :
- IMAGERY WAS EXTRACTED FROM GOOGLE EARTH PRO EDITION (DATED 2012)
- ONSITE TOPOGRAPHY CONTOURS PROVIDED BY CLIENT (DATED 2012), AND ARE SET TO INTERVALS OF 10 m

0 250m
Scale: 1:7,500 @ 11"x17"

LEGEND
- BOREHOLE LOCATION
- TESTPIT LOCATION
P - INDICATES HISTORIC PIEZOMETER
T - INDICATES HISTORIC THERMISTOR

MS1 - CHILKOOT (2015) - 6 LOCATIONS
BH01 - GOLDER (1978) - 19 LOCATIONS
CLMS - RRU (1999) - 13 LOCATIONS (NOT SHOWN)
CLWS - RRU (1999) - 7 LOCATIONS (NOT SHOWN)
WP1 - WORLEY (2015) - 3 LOCATIONS

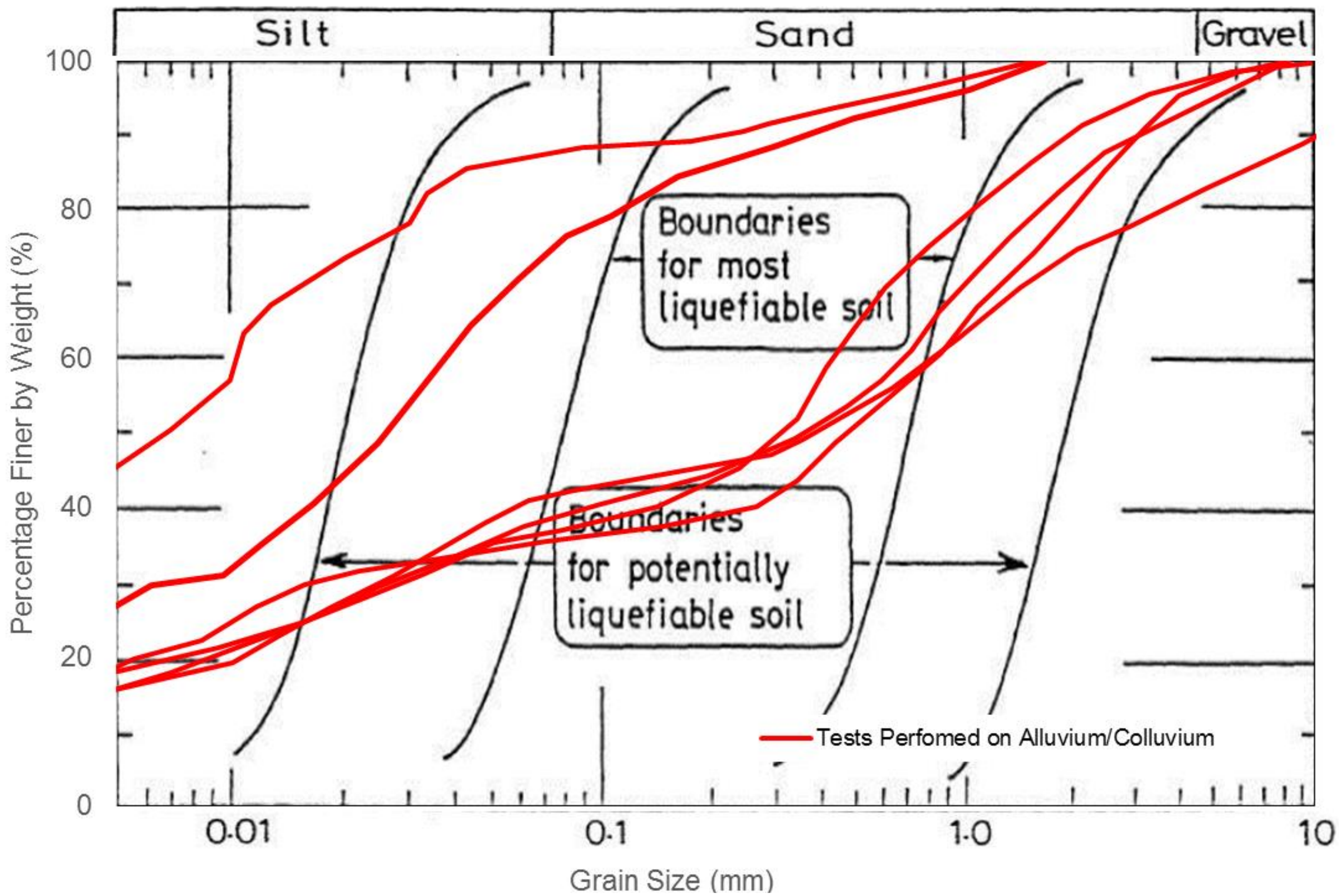
CLIENT
Yukon
Government
Energy, Mines & Resources
Assessment & Abandoned Mines Branch

TETRA TECH EBA

STABILITY ANALYSIS REPORT
ABANDONED CLINTON CREEK MINE, YUKON

SITE PLAN

PROJECT NO.	DWN	CKD	REV	
ENGWARC03039	RERH	SB	0	Figure 1
OFFICE	DATE			
EBA-WHSE	March 30, 2016			



LEGEND

NOTES

STATUS

Yukon Government
Energy, Mines & Resources
Assessment & Abandoned Mines Branch

TETRA TECH EBA

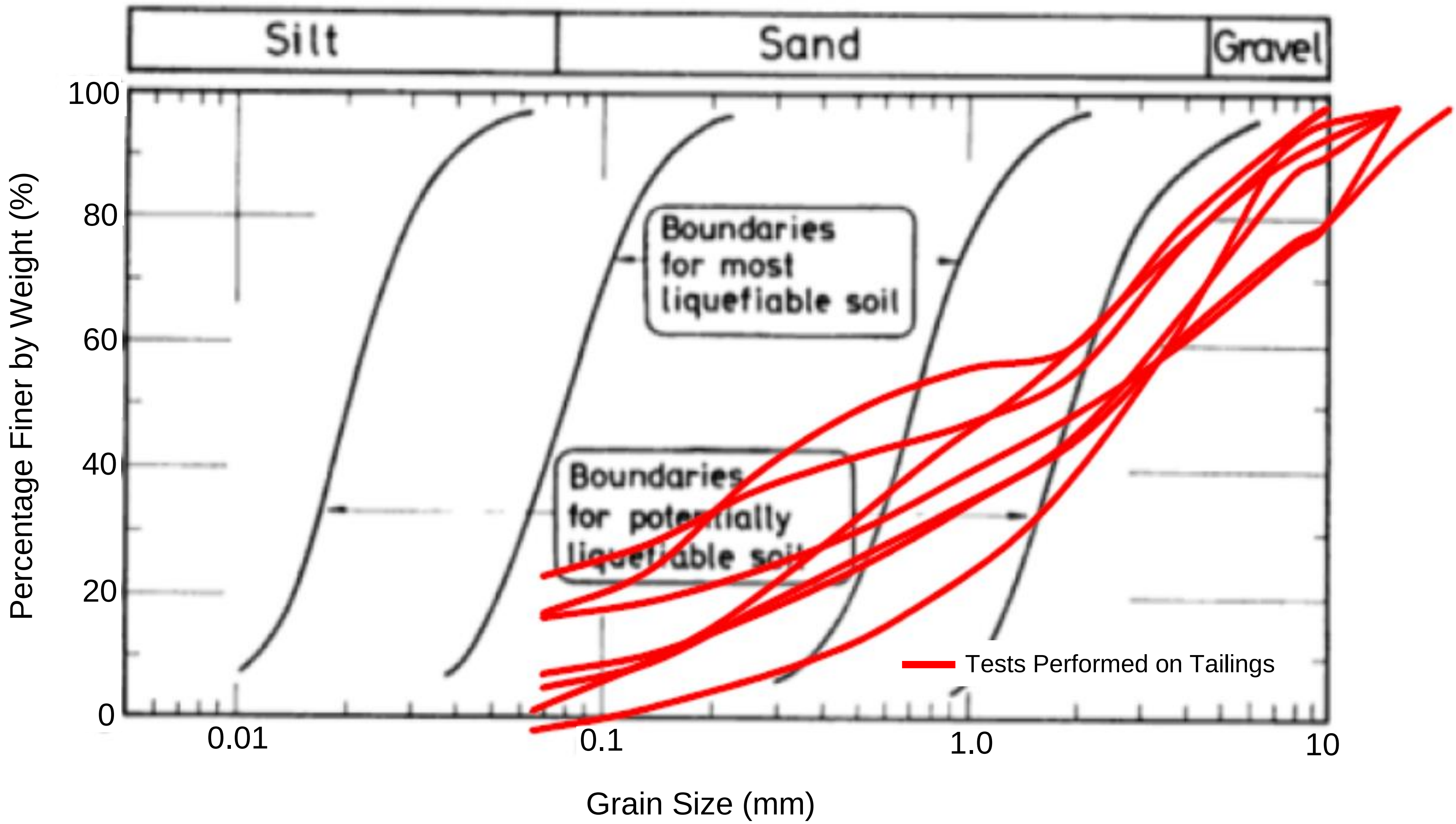
STABILITY ANALYSIS REPORT
ABANDONED CLINTON CREEK MINE, YUKON

Gradations of Liquefiable Soil and Alluvium/Colluvium

PROJECT NO.	DWN	CKD	APVD	REV
704-ENG.WARC03039	LW	AA	XL	0

OFFICE	DATE
EBA-VANC	March 30, 2016

Figure 2



LEGEND

NOTES

STATUS

CLIENT

Yukon Government
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TETRA TECH EBA

STABILITY ANALYSIS REPORT
ABANDONED CLINTON CREEK MINE, YUKON

Gradations of Liquefiable Soil and Tailings

PROJECT NO.	DWN	CKD	APVD	REV
704-ENG.WARC03039	XL	AFR	XL	0

OFFICE	DATE
EBA-VANC	March 30, 2016

Figure 3

APPENDIX A

TETRA TECH EBA'S GENERAL CONDITIONS

GENERAL CONDITIONS

GEOTECHNICAL REPORT – YUKON GOVERNMENT

This report incorporates and is subject to these "General Conditions".

1.0 USE OF REPORT AND OWNERSHIP

This geotechnical report pertains to a specific site, a specific development and a specific scope of work. It is not applicable to any other sites nor should it be relied upon for types of development other than that to which it refers. Any variation from the site or development would necessitate a supplementary geotechnical assessment.

This report and the recommendations contained in it are intended for the sole use of Tetra Tech EBA's Client, the Yukon Government. Tetra Tech EBA does not accept any responsibility for the accuracy of any of the data, the analyses or the recommendations contained or referenced in the report when the report is used or relied upon by any party other than Tetra Tech EBA's Client unless otherwise authorized in writing by Tetra Tech EBA. Any unauthorized use of the report is at the sole risk of the user.

This report is subject to copyright and shall not be reproduced either wholly or in part without the prior, written permission of the Yukon Government, the Client, or Tetra Tech EBA. It is acknowledged that the Yukon Government, the Client, may reproduce the report freely for internal usage.

2.0 ALTERNATE REPORT FORMAT

Where Tetra Tech EBA submits both electronic file and hard copy versions of reports, drawings and other project-related documents and deliverables (collectively termed Tetra Tech EBA's instruments of professional service), only the signed and/or sealed versions shall be considered final and legally binding. The original signed and/or sealed version archived by Tetra Tech EBA shall be deemed to be the original for the Project.

Both electronic file and hard copy versions of Tetra Tech EBA's instruments of professional service shall not, under any circumstances, no matter who owns or uses them, be altered by any party except Tetra Tech EBA. Tetra Tech EBA's instruments of professional service will be used only and exactly as submitted by Tetra Tech EBA.

Electronic files submitted by Tetra Tech EBA have been prepared and submitted using specific software and hardware systems. Tetra Tech EBA makes no representation about the compatibility of these files with the Client's current or future software and hardware systems.

3.0 ENVIRONMENTAL AND REGULATORY ISSUES

Unless stipulated in the report, Tetra Tech EBA has not been retained to investigate, address or consider and has not investigated, addressed or considered any environmental or regulatory issues associated with development on the subject site.

4.0 NATURE AND EXACTNESS OF SOIL AND ROCK DESCRIPTIONS

Classification and identification of soils and rocks are based upon commonly accepted systems and methods employed in professional geotechnical practice. This report contains descriptions of the systems and methods used. Where deviations from the system or method prevail, they are specifically mentioned.

Classification and identification of geological units are judgmental in nature as to both type and condition. Tetra Tech EBA does not warrant conditions represented herein as exact, but infers accuracy only to the extent that is common in practice.

Where subsurface conditions encountered during development are different from those described in this report, qualified geotechnical personnel should revisit the site and review recommendations in light of the actual conditions encountered.

5.0 LOGS OF TESTHOLES

The testhole logs are a compilation of conditions and classification of soils and rocks as obtained from field observations and laboratory testing of selected samples. Soil and rock zones have been interpreted. Change from one geological zone to the other, indicated on the logs as a distinct line, can be, in fact, transitional. The extent of transition is interpretive. Any circumstance which requires precise definition of soil or rock zone transition elevations may require further investigation and review.

6.0 STRATIGRAPHIC AND GEOLOGICAL INFORMATION

The stratigraphic and geological information indicated on drawings contained in this report are inferred from logs of test holes and/or soil/rock exposures. Stratigraphy is known only at the locations of the test hole or exposure. Actual geology and stratigraphy between test holes and/or exposures may vary from that shown on these drawings. Natural variations in geological conditions are inherent and are a function of the historic environment. Tetra Tech EBA does not represent the conditions illustrated as exact but recognizes that variations will exist. Where knowledge of more precise locations of geological units is necessary, additional investigation and review may be necessary.

7.0 PROTECTION OF EXPOSED GROUND

Excavation and construction operations expose geological materials to climatic elements (freeze/thaw, wet/dry) and/or mechanical disturbance which can cause severe deterioration. Unless otherwise specifically indicated in this report, the walls and floors of excavations must be protected from the elements, particularly moisture, desiccation, frost action and construction traffic.

8.0 SUPPORT OF ADJACENT GROUND AND STRUCTURES

Unless otherwise specifically advised, support of ground and structures adjacent to the anticipated construction and preservation of adjacent ground and structures from the adverse impact of construction activity is required.

9.0 INFLUENCE OF CONSTRUCTION ACTIVITY

There is a direct correlation between construction activity and structural performance of adjacent buildings and other installations. The influence of all anticipated construction activities should be considered by the contractor, owner, architect and prime engineer in consultation with a geotechnical engineer when the final design and construction techniques are known.

10.0 OBSERVATIONS DURING CONSTRUCTION

Because of the nature of geological deposits, the judgmental nature of geotechnical engineering, as well as the potential of adverse circumstances arising from construction activity, observations during site preparation, excavation and construction should be carried out by a geotechnical engineer. These observations may then serve as the basis for confirmation and/or alteration of geotechnical recommendations or design guidelines presented herein.

11.0 DRAINAGE SYSTEMS

Where temporary or permanent drainage systems are installed within or around a structure, the systems which will be installed must protect the structure from loss of ground due to internal erosion and must be designed so as to assure continued performance of the drains. Specific design detail of such systems should be developed or reviewed by the geotechnical engineer. Unless otherwise specified, it is a condition of this report that effective temporary and permanent drainage systems are required and that they must be considered in relation to project purpose and function.

12.0 BEARING CAPACITY

Design bearing capacities, loads and allowable stresses quoted in this report relate to a specific soil or rock type and condition. Construction activity and environmental circumstances can materially change the condition of soil or rock. The elevation at which a soil or rock type occurs is variable. It is a requirement of this report that structural elements be founded in and/or upon geological materials of the type and in the condition assumed. Sufficient observations should be made by qualified geotechnical personnel during construction to assure that the soil and/or rock conditions assumed in this report in fact exist at the site.

13.0 SAMPLES

Tetra Tech EBA will retain all soil and rock samples for 30 days after this report is issued. Further storage or transfer of samples can be made at the Client’s expense upon written request, otherwise samples will be discarded.

14.0 INFORMATION PROVIDED TO TETRA TECH EBA BY OTHERS

During the performance of the work and the preparation of the report, Tetra Tech EBA may rely on information provided by persons other than the Client. While Tetra Tech EBA endeavours to verify the accuracy of such information when instructed to do so by the Client, Tetra Tech EBA accepts no responsibility for the accuracy or the reliability of such information which may affect the report.

APPENDIX B

CLINTON CREEK STABILITY – BACK ANALYSIS OF EXISTING CONDITIONS

MATERIAL INFORMATION

Loading Condition	Color	Material	ϕ' (°)	c' (kPa)	γ (kN/m³)	Piezometric Condition
Steady State Parameters		Colluvium/Alluvium	0	60	19.1	Undrained
		Colluvium/Alluvium (2)	12	0	19.1	Piezometric Line 2 $\bar{B}$ = 0.85
		Colluvium/Alluvium (3)	15	0	19.1	Piezometric Line 2 $\bar{B}$ = 0.92
		Colluvium/Alluvium (4)	17.5	0	19.1	Piezometric Line 2 $\bar{B}$ = 0.95
		Colluvium/Alluvium (5)	4	0	19.1	Piezometric Line 1
		Colluvium/Alluvium (6)	6.4	0	19.1	Piezometric Line 1 (at El. 430 m)
		Colluvium/Alluvium (7)	9.4	0	19.1	Piezometric Line 1 (at El. 455 m)
		Waste Rock	23	0	19.6	Piezometric Line 1
		Weathered Argillite	27	0	21.6	Piezometric Line 1
Seismic Event Parameters		Colluvium/Alluvium S	0	48	19.1	Undrained
		Waste Rock S	18.8	0	19.6	Piezometric Line 1
		Weathered Argillite S	22.2	0	21.6	Piezometric Line 1
Unchanged Parameters		Frozen Alluvium/Argillite	Impenetrable			
		Argillite	Impenetrable			

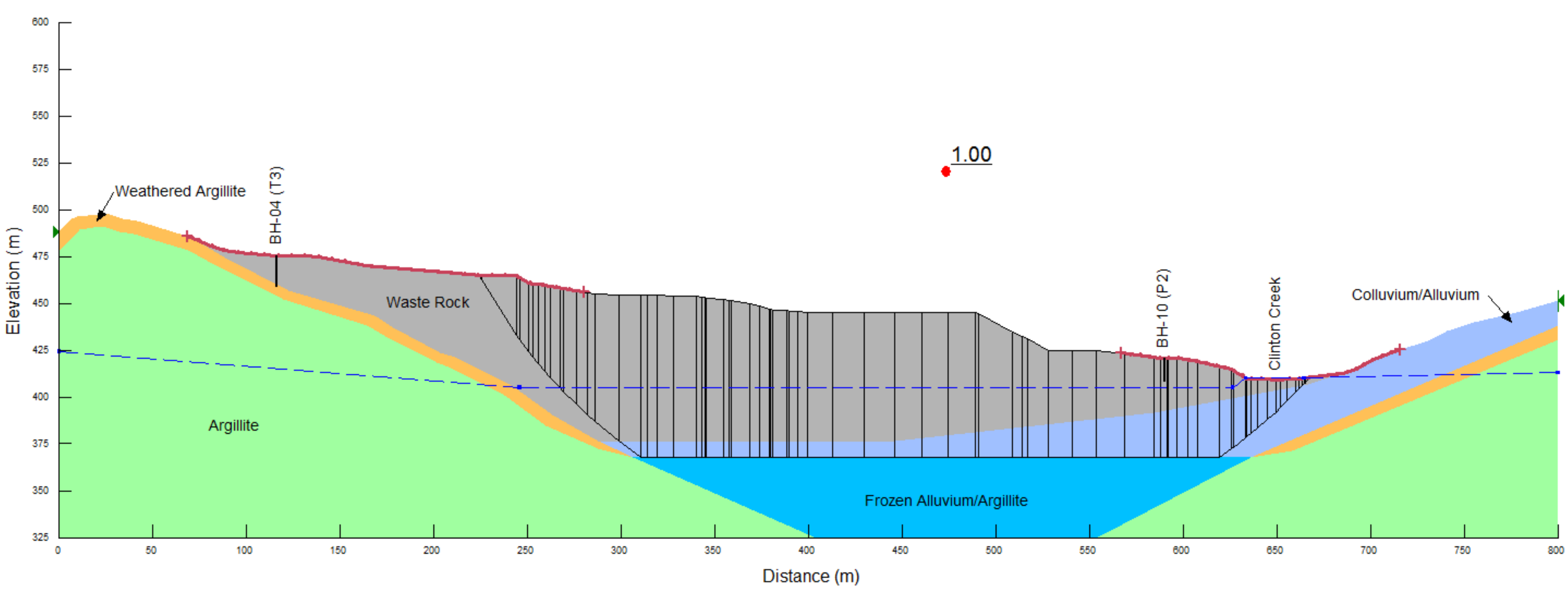
STABILITY ANALYSIS RESULTS – EXISTING CONDITIONS

Figure	Section	Model	Loading Condition	Phreatic Surface Level (masl)	Horizontal Acceleration (g)	FOS
B.1	Section B-B’	Back Analysis – Total Stress	Steady State Seepage	405	0	1.00
B.2	Section B-B’	Back Analysis – Effective Stress (Phi 4°)	Steady State Seepage	405	0	1.01
B.3	Section B-B’	Back Analysis – Effective Stress	Increased PWP (25 m)	430	0	1.00
B.4	Section B-B’	Back Analysis – Effective Stress	Increased PWP (50 m)	455	0	1.00
B.5	Section B-B’	Back Analysis – Effective Stress (Phi 12°)	Steady State Seepage	405	0	1.00
B.6	Section B-B’	Back Analysis – Effective Stress (Phi 15°)	Steady State Seepage	405	0	1.00
B.7	Section B-B’	Back Analysis – Effective Stress (Phi 17.5°)	Steady State Seepage	405	0	1.00
B.8	Section B-B’	Existing Conditions	Pseudo-Static Seismic	405	0.154	0.35
B.9	Section F-F’	Existing Conditions	Steady State Seepage	411 to 370	0	1.10
B.10	Section F-F’	Existing Conditions	Pseudo-Static Seismic	411 to 370	0.154	0.24

Clinton Creek Waste Rock Dump - Section B-B'

Steady State Seepage Conditions

Name: Waste Rock Unit Weight: 19.6 kN/m³ Cohesion': 0 kPa Phi': 23 ° Piezometric Line: 1
Name: Colluvium/Alluvium Unit Weight: 19.1 kN/m³ Cohesion': 60 kPa Piezometric Line: 1
Name: Argillite Piezometric Line: 1
Name: Weathered Argillite Unit Weight: 21.6 kN/m³ Cohesion': 0 kPa Phi': 27 ° Piezometric Line: 1
Name: Frozen Alluvium/Argillite Piezometric Line: 1



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STABILITY ANALYSIS REPORT
ABANDONED CLINTON CREEK MINE, YUKON

Section B-B'
Back Analysis – Total Stress
Steady State Seepage Conditions

PROJECT NO.	DWN	CKD	APVD	REV
704-ENG.WARC03039	CD	XL	AFR	0

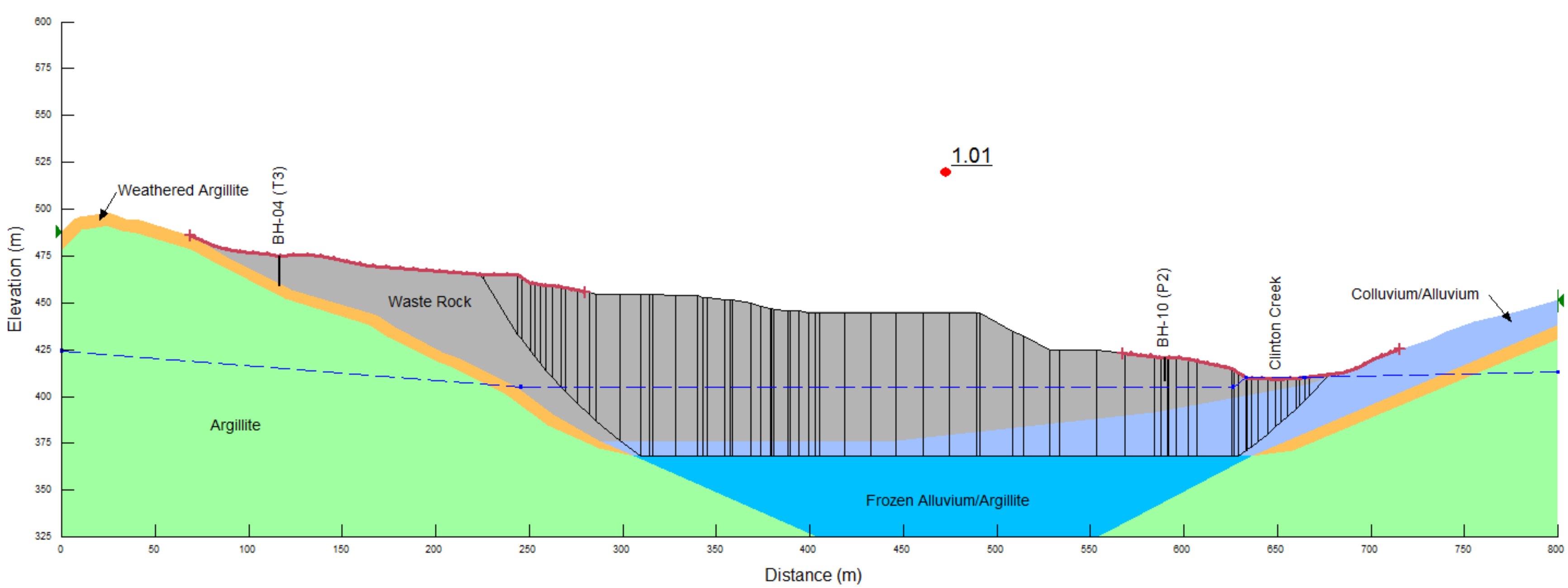
OFFICE	DATE
EBA-EDM	March 31, 2016

Figure B.1

Clinton Creek Waste Rock Dump - Section B-B'

Steady State Seepage Conditions - Effective Stress - No Bbar

Name: Waste Rock Unit Weight: 19.6 kN/m³ Cohesion': 0 kPa Phi': 23 ° Piezometric Line: 1
Name: Argillite Piezometric Line: 1
Name: Weathered Argillite Unit Weight: 21.6 kN/m³ Cohesion': 0 kPa Phi': 27 ° Piezometric Line: 1
Name: Frozen Alluvium/Argillite Piezometric Line: 1
Name: Colluvium/Alluvium (5) Unit Weight: 19.1 kN/m³ Cohesion': 0 kPa Phi': 4 ° Piezometric Line: 1



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ABANDONED CLINTON CREEK MINE, YUKON

Section B-B'
Back Analysis – Effective Stress (Phi 4°)
Steady State Seepage Conditions

PROJECT NO.	DWN	CKD	APVD	REV
704-ENG.WARC03039	CD	XL	AFR	0
OFFICE	**DATE**			
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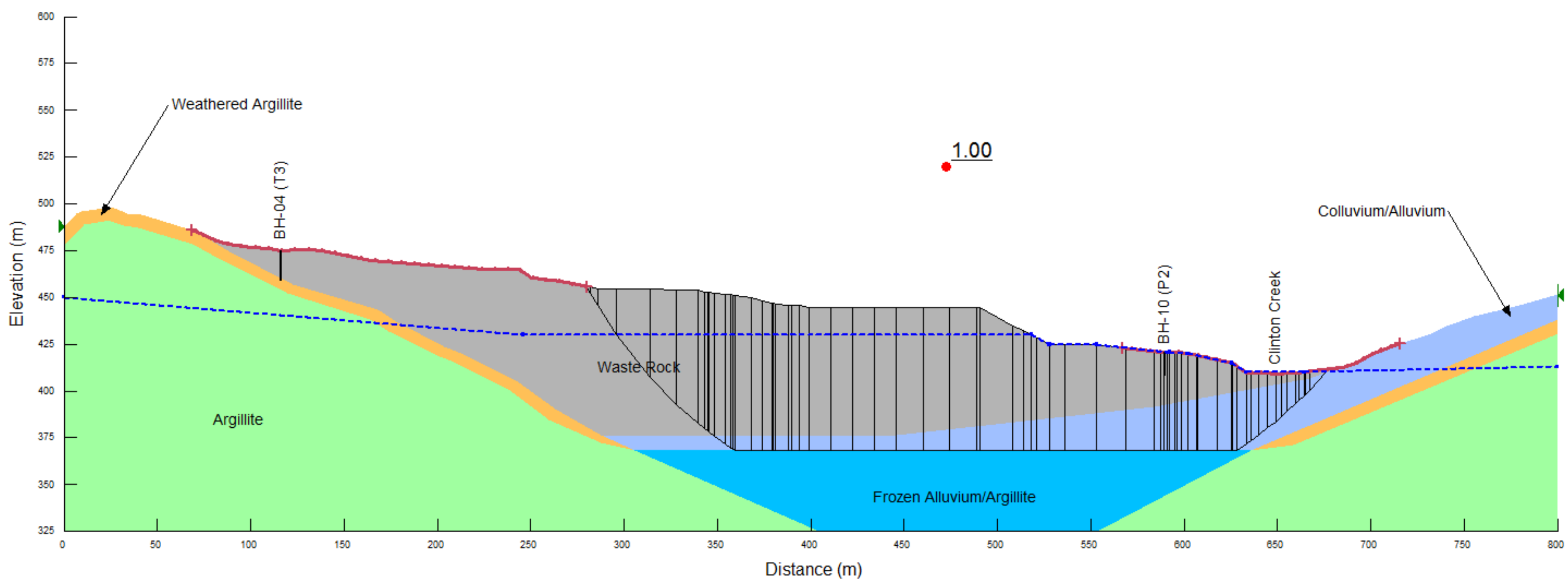
Figure B.2

Clinton Creek Waste Rock Dump - Section B-B'

Steady State Seepage Conditions (25m)

Name: Waste Rock Unit Weight: 19.6 kN/m³ Cohesion': 0 kPa Phi': 23 ° Piezometric Line: 1
Name: Argillite Piezometric Line: 1
Name: Weathered Argillite Unit Weight: 21.6 kN/m³ Cohesion': 0 kPa Phi': 27 ° Piezometric Line: 1
Name: Frozen Alluvium/Argillite Piezometric Line: 1
Name: Colluvium/Alluvium (6) Unit Weight: 19.1 kN/m³ Cohesion': 0 kPa Phi': 6.4 ° Piezometric Line: 1



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Section B-B'
Back Analysis – Effective Stress
Increased PWP Conditions (25 m Increase)

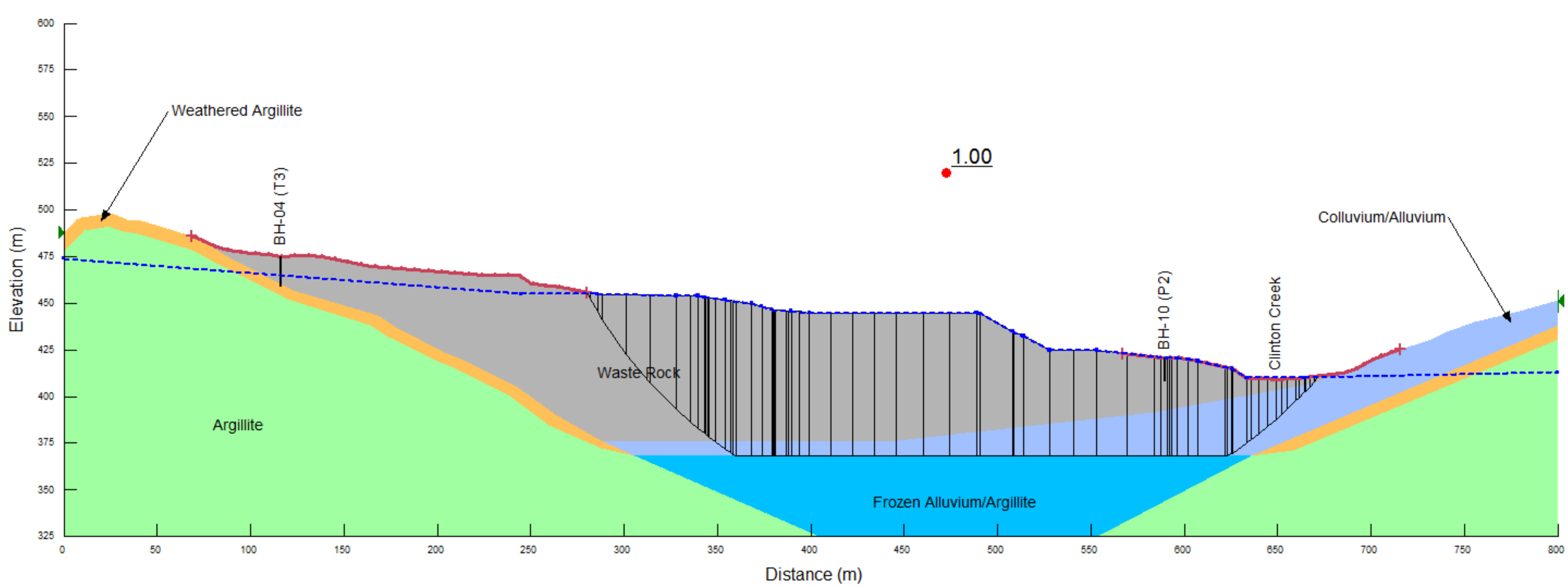
PROJECT NO.	DWN	CKD	APVD	REV
704-ENG.WARC03039	CD	XL	AFR	0
OFFICE	DATE			
EBA-EDM	March 31, 2016			

Figure B.3

Clinton Creek Waste Rock Dump - Section B-B'

Steady State Seepage Conditions (50m)

Name: Waste Rock Unit Weight: 19.6 kN/m³ Cohesion': 0 kPa Phi': 23 ° Piezometric Line: 1
Name: Argillite Piezometric Line: 1
Name: Weathered Argillite Unit Weight: 21.6 kN/m³ Cohesion': 0 kPa Phi': 27 ° Piezometric Line: 1
Name: Frozen Alluvium/Argillite Piezometric Line: 1
Name: Colluvium/Alluvium (7) Unit Weight: 19.1 kN/m³ Cohesion': 0 kPa Phi': 9.4 ° Piezometric Line: 1



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Section B-B'
Back Analysis – Effective Stress
Increased PWP Conditions (50 m Increase)

PROJECT NO.	DWN	CKD	APVD	REV
704-ENG.WARC03039	CD	XL	AFR	0

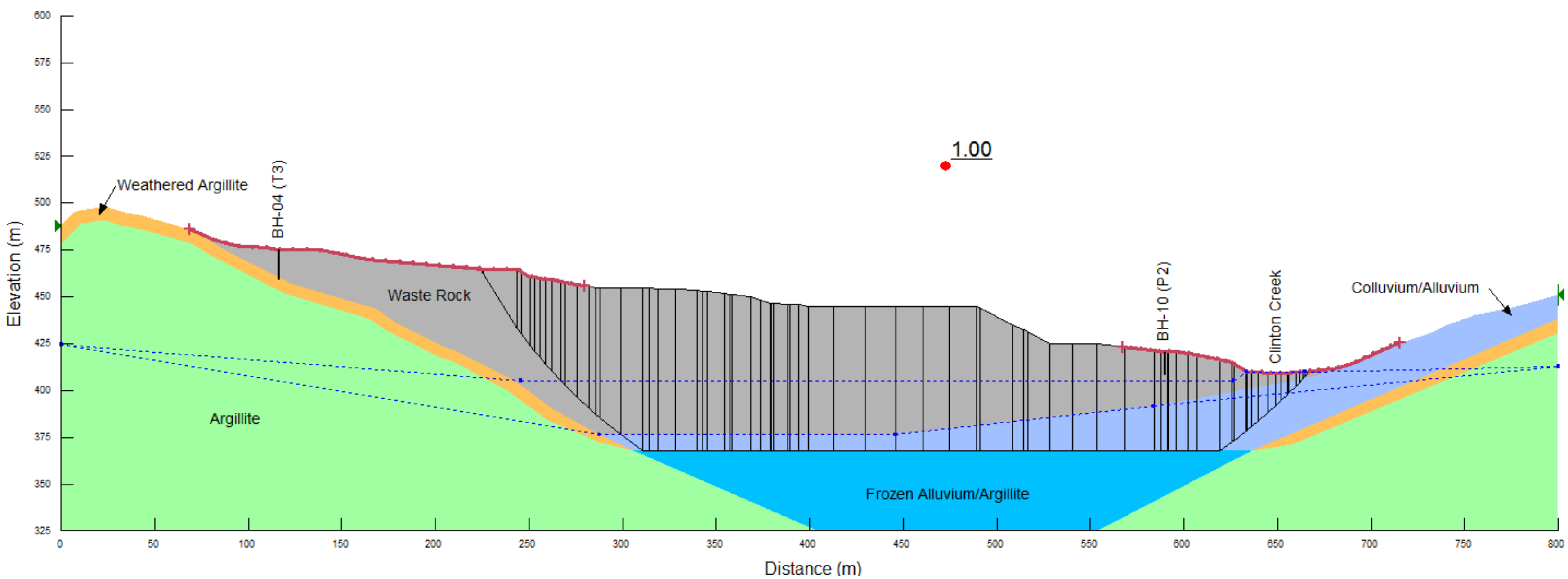
OFFICE	DATE
EBA-EDM	March 31, 2016

Figure B.4

Clinton Creek Waste Rock Dump - Section B-B'

Steady State Seepage Conditions - Effective Stress (Phi12)

Name: Waste Rock Unit Weight: 19.6 kN/m³ Cohesion': 0 kPa Phi': 23 ° Piezometric Line: 1 B-bar: 0 Add Weight: Yes
Name: Argillite Piezometric Line: 1 B-bar: 0 Add Weight: No
Name: Weathered Argillite Unit Weight: 21.6 kN/m³ Cohesion': 0 kPa Phi': 27 ° Piezometric Line: 1 B-bar: 0 Add Weight: No
Name: Frozen Alluvium/Argillite Piezometric Line: 1 B-bar: 0 Add Weight: No
Name: Colluvium/Alluvium (2) Unit Weight: 19.1 kN/m³ Cohesion': 0 kPa Phi': 12 ° Piezometric Line: 2 B-bar: 0.855 Add Weight: No



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Section B-B'
Back Analysis – Effective Stress (Phi 12°)
Steady State Seepage Conditions

PROJECT NO.	DWN	CKD	APVD	REV
704-ENG.WARC03039	CD	XL	AFR	0

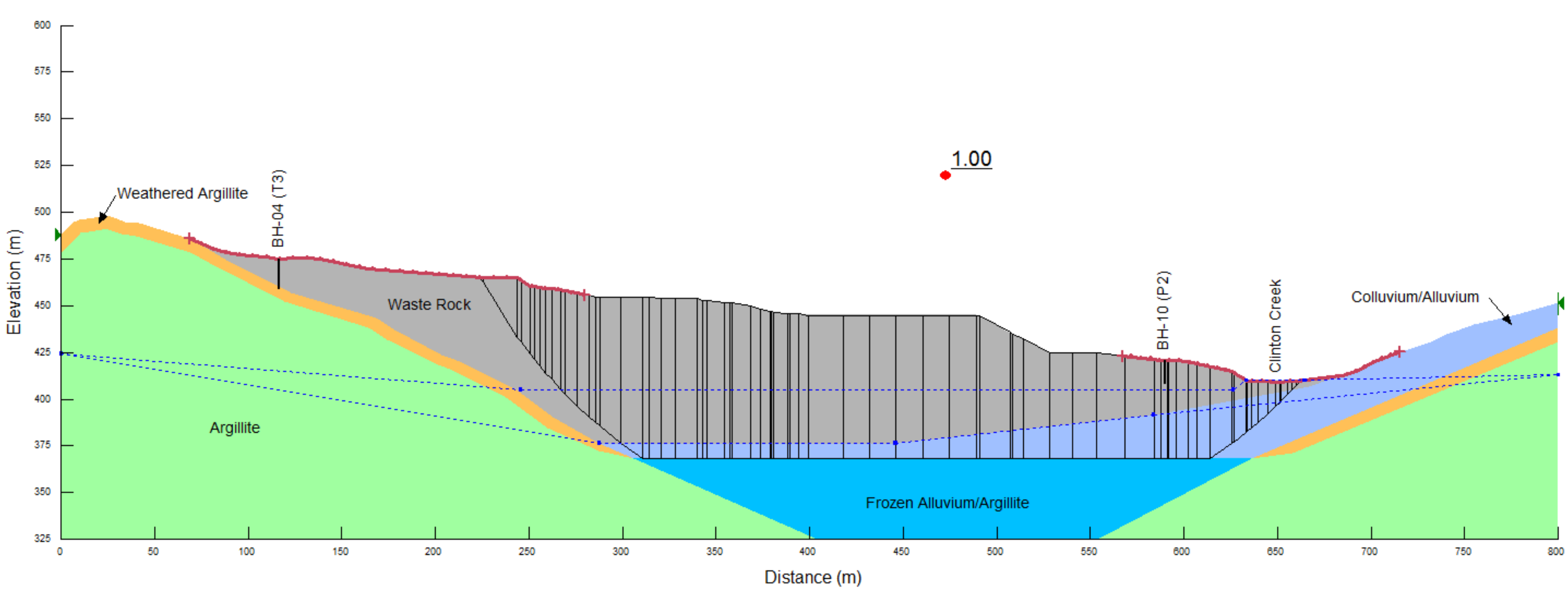
OFFICE	DATE
EBA-EDM	March 31, 2016

Figure B.5

Clinton Creek Waste Rock Dump - Section B-B'

Steady State Seepage Conditions - Effective Stress (Phi15)

Name: Waste Rock Unit Weight: 19.6 kN/m³ Cohesion': 0 kPa Phi': 23 ° Piezometric Line: 1 B-bar: 0 Add Weight: Yes
Name: Argillite Piezometric Line: 1 B-bar: 0 Add Weight: No
Name: Weathered Argillite Unit Weight: 21.6 kN/m³ Cohesion': 0 kPa Phi': 27 ° Piezometric Line: 1 B-bar: 0 Add Weight: No
Name: Frozen Alluvium/Argillite Piezometric Line: 1 B-bar: 0 Add Weight: No
Name: Colluvium/Alluvium (3) Unit Weight: 19.1 kN/m³ Cohesion': 0 kPa Phi': 15 ° Piezometric Line: 2 B-bar: 0.92 Add Weight: No



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Section B-B'
Back Analysis – Effective Stress (Phi 15°)
Steady State Seepage Conditions

PROJECT NO.	DWN	CKD	APVD	REV
704-ENG.WARC03039	CD	XL	AFR	0
OFFICE	DATE			
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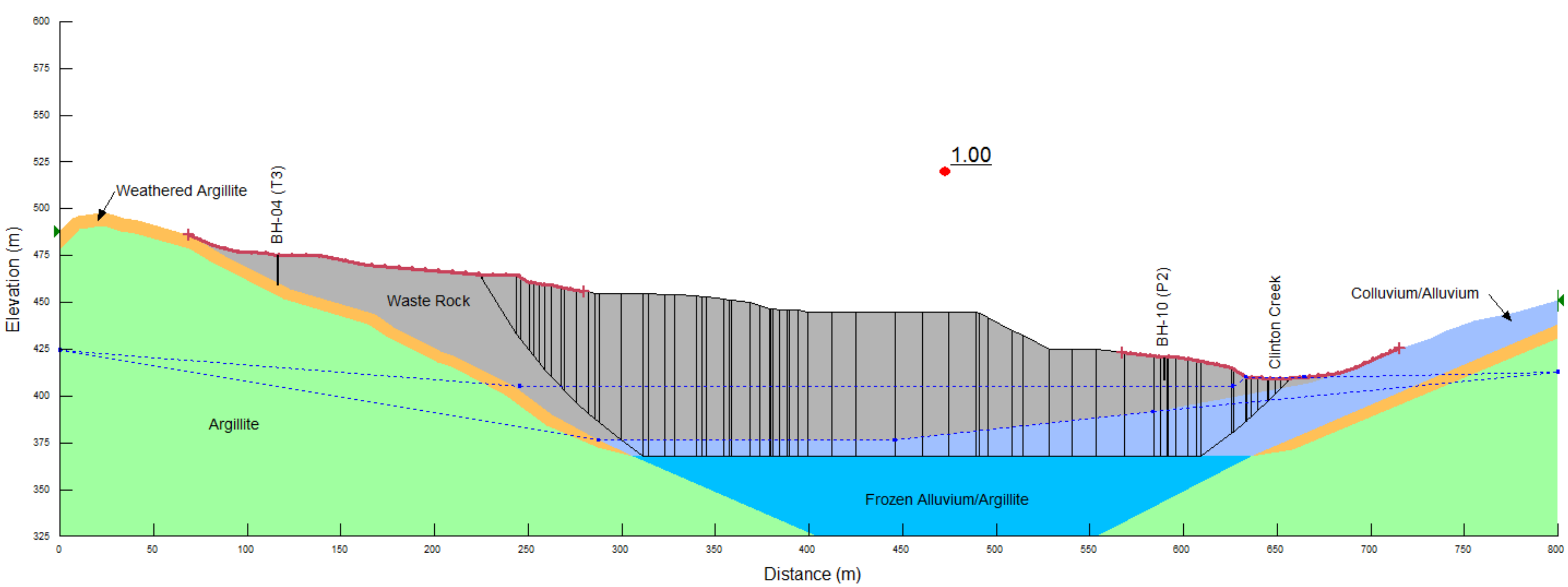
Figure B.6

Clinton Creek Waste Rock Dump - Section B-B'

Steady State Seepage Conditions - Effective Stress (Phi17.5)

Name: Waste Rock Unit Weight: 19.6 kN/m³ Cohesion': 0 kPa Phi': 23 ° Piezometric Line: 1 B-bar: 0 Add Weight: Yes
Name: Argillite Piezometric Line: 1 B-bar: 0 Add Weight: No
Name: Weathered Argillite Unit Weight: 21.6 kN/m³ Cohesion': 0 kPa Phi': 27 ° Piezometric Line: 1 B-bar: 0 Add Weight: No
Name: Frozen Alluvium/Argillite Piezometric Line: 1 B-bar: 0 Add Weight: No
Name: Colluvium/Alluvium (4) Unit Weight: 19.1 kN/m³ Cohesion': 0 kPa Phi': 17.5 ° Piezometric Line: 2 B-bar: 0.955 Add Weight: No



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Section B-B'
Back Analysis – Effective Stress (Phi 17.5°)
Steady State Seepage Conditions

PROJECT NO.	DWN	CKD	APVD	REV
704-ENG.WARC03039	CD	XL	AFR	0

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EBA-EDM	March 31, 2016

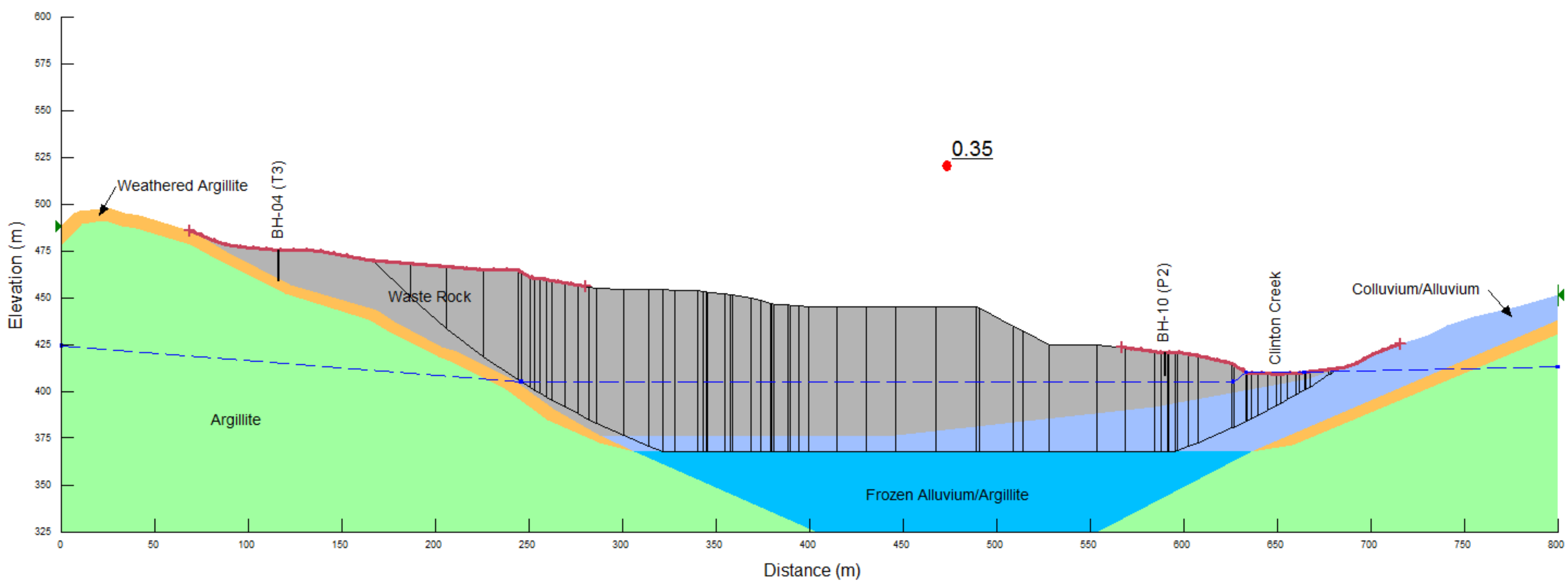
Figure B.7

Clinton Creek Waste Rock Dump - Section B-B'

Pseudo-Static Seismic Conditions

Name: Argillite Piezometric Line: 1
Name: Frozen Alluvium/Argillite Piezometric Line: 1
Name: Colluvium/Alluvium S Unit Weight: 19.1 kN/m³ Cohesion': 48 kPa Piezometric Line: 1
Name: Waste Rock S Unit Weight: 19.6 kN/m³ Cohesion': 0 kPa Phi': 18.8 ° Piezometric Line: 1
Name: Weathered Argillite S Unit Weight: 21.6 kN/m³ Cohesion': 0 kPa Phi': 22.2 ° Piezometric Line: 1

Horz Seismic Coef.: 0.154



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ABANDONED CLINTON CREEK MINE, YUKON

Section B-B'
Existing Conditions
Pseudo-Static Seismic Conditions

PROJECT NO.	DWN	CKD	APVD	REV
704-ENG.WARC03039	CD	XL	AFR	0

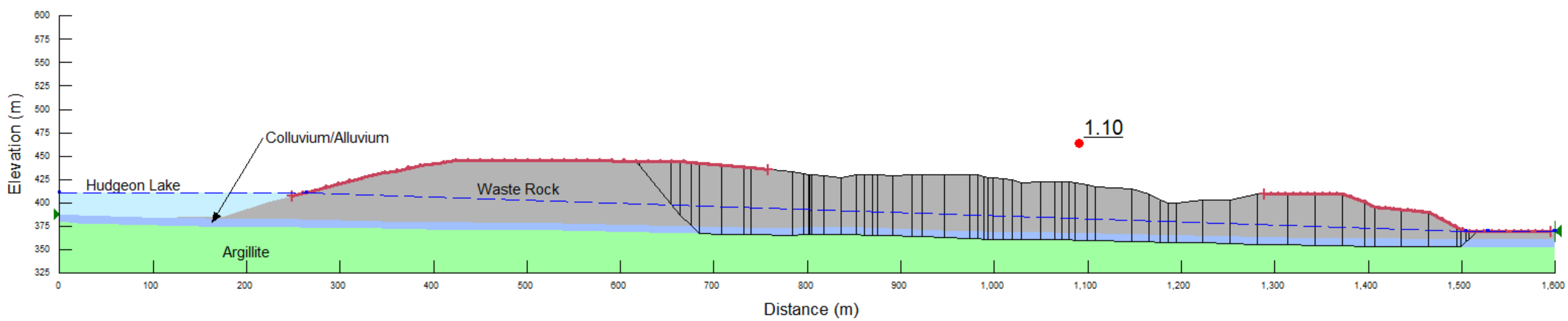
OFFICE	DATE
EBA-EDM	March 31, 2016

Figure B.8

Clinton Creek Waste Rock Dump - Section F-F'

Steady State Seepage Conditions

Name: Frozen Alluvium/Argillite   Piezometric Line: 1
Name: Colluvium/Alluvium 60   Piezometric Line: 1   Unit Weight: 19.1 kN/m³   Cohesion': 60 kPa
Name: Waste Rock   Piezometric Line: 1   Unit Weight: 19.6 kN/m³   Cohesion': 0 kPa   Phi': 23 °



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Section F-F'
Existing Conditions
Steady State Seepage Conditions

PROJECT NO.	DWN	CKD	APVD	REV
704-ENG.WARC03039	CD	XL	AFR	0
OFFICE	DATE			
EBA-EDM	March 31, 2016			

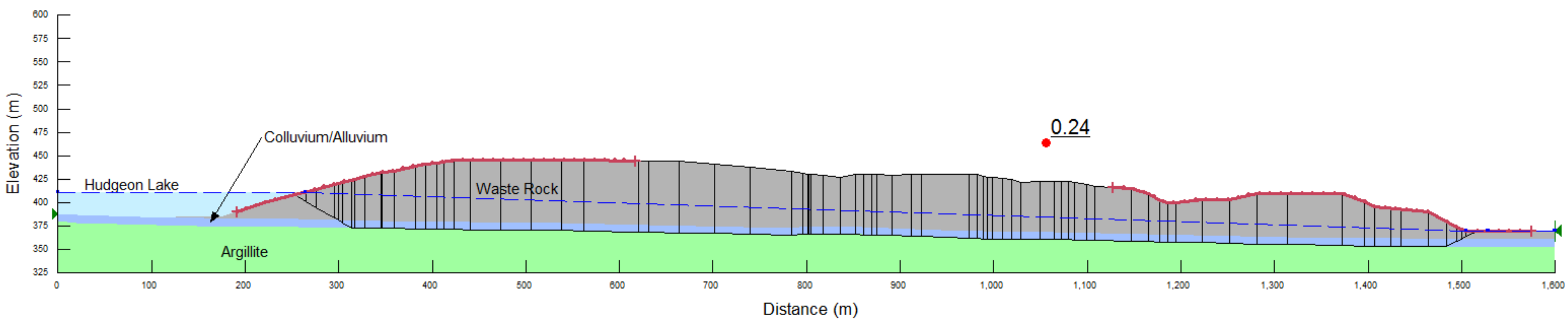
Figure B.9

Clinton Creek Waste Rock Dump - Section F-F'

Pseudo-Static Seismic Conditions

Name: Frozen Alluvium/Argillite   Piezometric Line: 1
Name: Colluvium/Alluvium S   Piezometric Line: 1   Unit Weight: 19.1 kN/m³   Cohesion': 48 kPa
Name: Waste Rock S   Piezometric Line: 1   Unit Weight: 19.6 kN/m³   Cohesion': 0 kPa   Phi': 18.8 °

Horz Seismic Coef.: 0.154



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STABILITY ANALYSIS REPORT
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Section F-F'
Existing Conditions
Pseudo-Static Seismic Conditions

PROJECT NO.	DWN	CKD	APVD	REV
704-ENG.WARC03039	CD	XL	AFR	0

OFFICE	DATE
EBA-EDM	March 31, 2016

Figure B.10

APPENDIX C
CLINTON CREEK STABILITY – ANALYSIS OF CLOSURE OPTIONS

MATERIAL INFORMATION

Loading Condition	Color	Material	ϕ' (°)	c' (kPa)	γ (kN/m^3)	Piezometric Condition
Steady State Seepage Parameters		Colluvium/Alluvium	0	60	19.1	Undrained
		Waste Rock	23	0	19.6	Piezometric Line 1
		Weathered Argillite	27	0	21.6	Piezometric Line 1
Pseudo-Static Seismic Parameters		Colluvium/Alluvium S	0	48	19.1	Undrained
		Waste Rock S	18.8	0	19.6	Piezometric Line 1
		Weathered Argillite S	22.2	0	21.6	Piezometric Line 1
Unchanged Parameters		Frozen Alluvium/Argillite	Impenetrable			
		Argillite	Impenetrable			

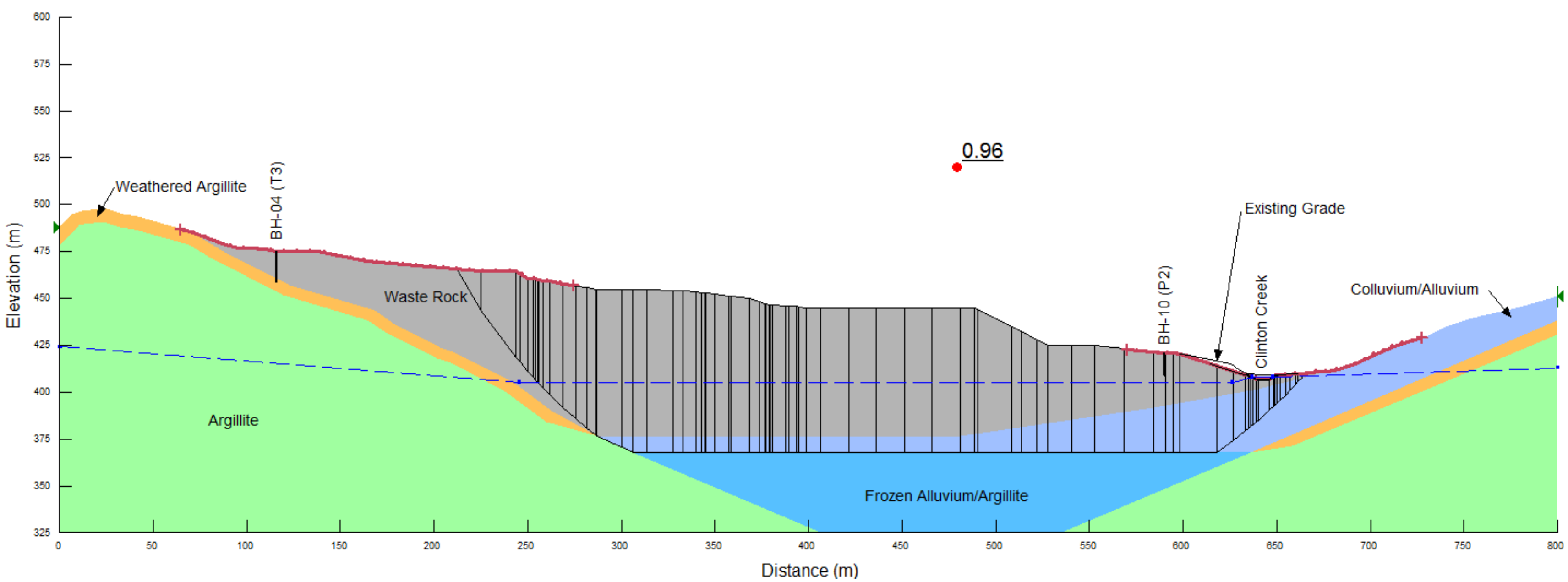
STABILITY ANALYSIS RESULTS – REMEDIATION OPTIONS

Figure	Section	Model	Loading Condition	Phreatic Surface Level (masl.)	Horizontal Acceleration (g)	FOS
C.1	Section B-B'	Option C3	Steady State Seepage	405	0	0.96
C.2	Section B-B'	Option C3	Pseudo-Static Seismic	405	0.154	0.27
C.3	Section B-B'	Option D3	Steady State Seepage	405	0	1.01
C.4	Section B-B'	Option D3	Pseudo-Static Seismic	405	0.154	0.31
C.5	Section B-B'	Option E3	Steady State Seepage	393	0	1.13
C.6	Section B-B'	Option E3	Pseudo-Static Seismic	393	0.154	0.32
C.7	Section B-B'	Option F	Steady State Seepage	Varies	0	0.71
C.8	Section B-B'	Option F	Pseudo-Static Seismic	Varies	0.154	0.29
C.9	Section B-B'	Option I2	Steady State Seepage	405	0	1.00
C.10	Section B-B'	Option I2	Pseudo-Static Seismic	405	0.154	0.37
C.11	Section F-F'	Option E3	Steady State Seepage	398 to 362	0	1.12
C.12	Section F-F'	Option E3	Pseudo-Static Seismic	398 to 362	0.154	0.25

Clinton Creek Waste Rock Dump - Section B-B'

Option C3 - Steady State Seepage Conditions

Name: Waste Rock   Unit Weight: 19.6 kN/m³   Cohesion': 0 kPa   Phi': 23 °   Piezometric Line: 1
Name: Colluvium/Alluvium 60   Unit Weight: 19.1 kN/m³   Cohesion': 60 kPa   Piezometric Line: 1
Name: Argillite   Piezometric Line: 1
Name: Weathered Argillite   Unit Weight: 21.6 kN/m³   Cohesion': 0 kPa   Phi': 27 °   Piezometric Line: 1
Name: Frozen Alluvium/Argillite   Piezometric Line: 1



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ABANDONED CLINTON CREEK MINE, YUKON

Section B-B'
Remediation Option C3
Steady State Seepage Conditions

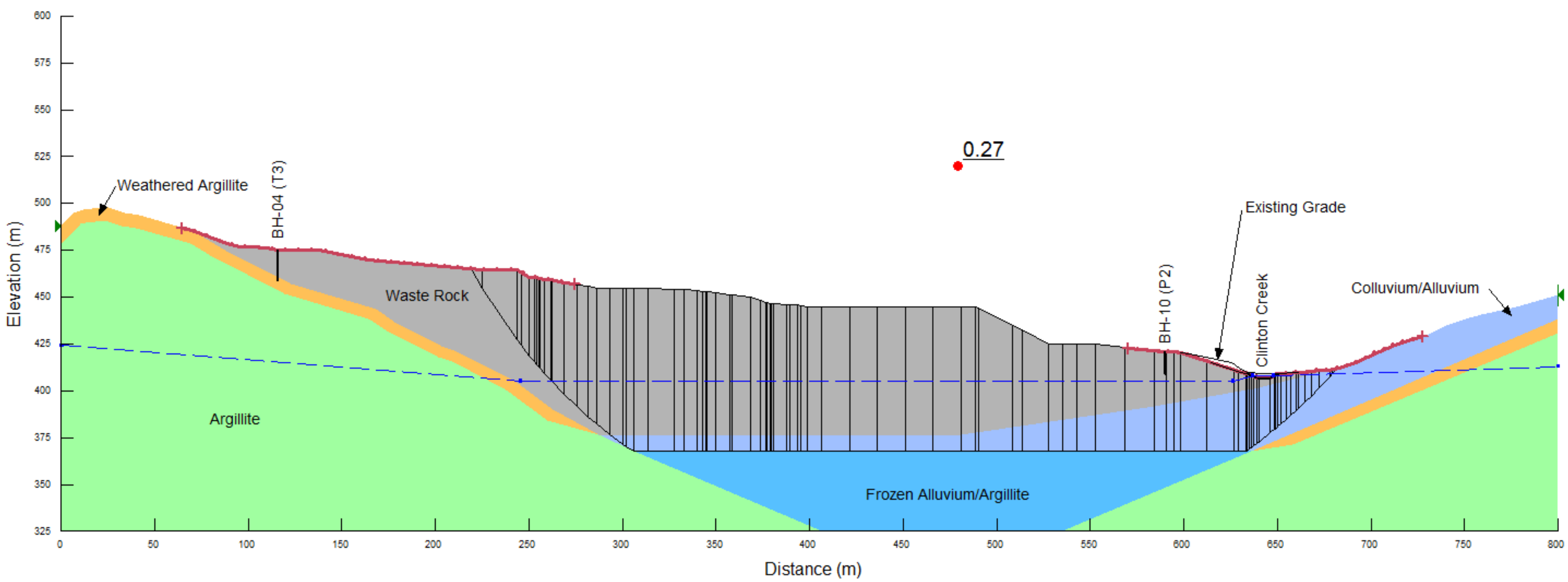
PROJECT NO.	DWN	CKD	APVD	REV	
704-ENG.WARC03039	CD	XL	AFR	0	**Figure C.1**
OFFICE	DATE				
EBA-EDM	March 17, 2016				

Clinton Creek Waste Rock Dump - Section B-B'

Option C3 - Pseudo-Static Seismic Conditions

Name: Argillite Piezometric Line: 1
Name: Frozen Alluvium/Argillite Piezometric Line: 1
Name: Colluvium/Alluvium S Unit Weight: 19.1 kN/m³ Cohesion': 48 kPa Piezometric Line: 1
Name: Waste Rock S Unit Weight: 19.6 kN/m³ Cohesion': 0 kPa Phi': 18.8 ° Piezometric Line: 1
Name: Weathered Argillite S Unit Weight: 21.6 kN/m³ Cohesion': 0 kPa Phi': 22.2 ° Piezometric Line: 1

Horz Seismic Coef.: 0.154



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ABANDONED CLINTON CREEK MINE, YUKON

Section B-B'
Remediation Option C3
Pseudo-Static Seismic Conditions

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704-ENG.WARC03039	CD	XL	AFR	0

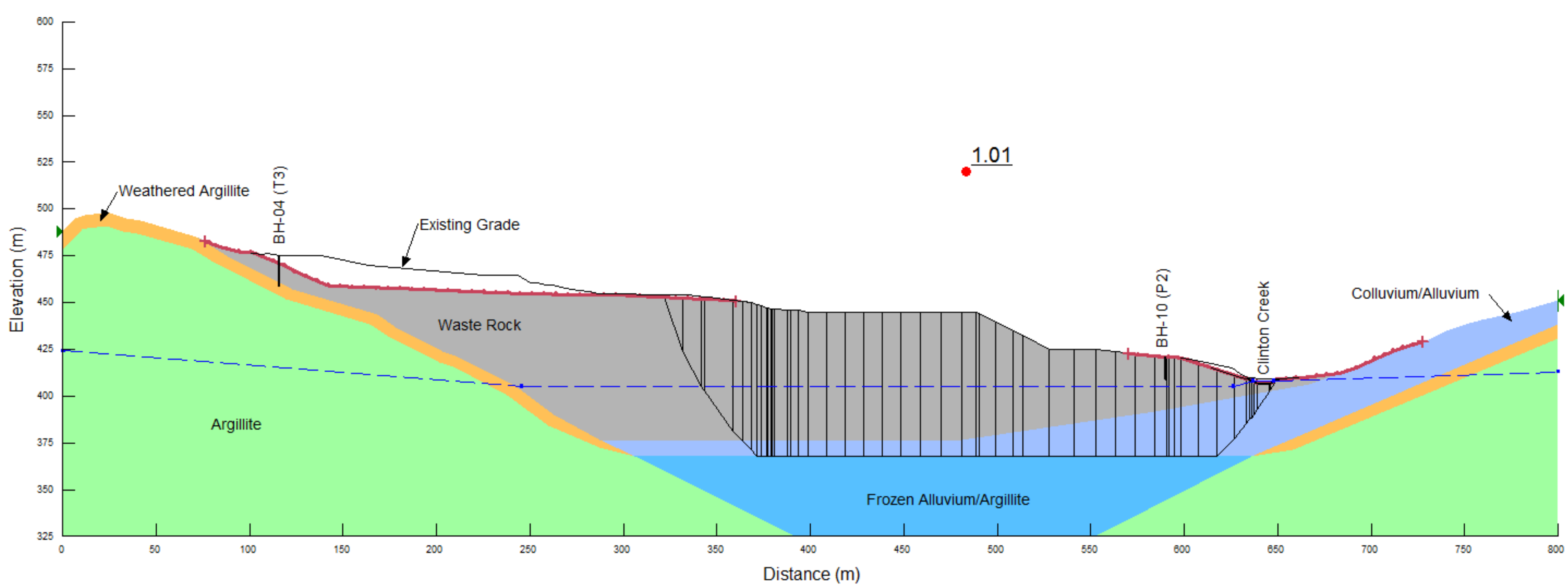
OFFICE	DATE
EBA-EDM	March 17, 2016

Figure C.2

Clinton Creek Waste Rock Dump - Section B-B'

Option D3 - Steady State Seepage Conditions

Name: Waste Rock Unit Weight: 19.6 kN/m³ Cohesion': 0 kPa Phi': 23 ° Piezometric Line: 1
Name: Colluvium/Alluvium Unit Weight: 19.1 kN/m³ Cohesion': 60 kPa Piezometric Line: 1
Name: Argillite Piezometric Line: 1
Name: Weathered Argillite Unit Weight: 21.6 kN/m³ Cohesion': 0 kPa Phi': 27 ° Piezometric Line: 1
Name: Frozen Alluvium/Argillite Piezometric Line: 1



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STABILITY ANALYSIS REPORT
ABANDONED CLINTON CREEK MINE, YUKON

Section B-B'
Remediation Option D3
Steady State Seepage Conditions

PROJECT NO.	DWN	CKD	APVD	REV
704-ENG.WARC03039	CD	XL	AFR	0
OFFICE	DATE			
EBA-EDM	March 17, 2016			

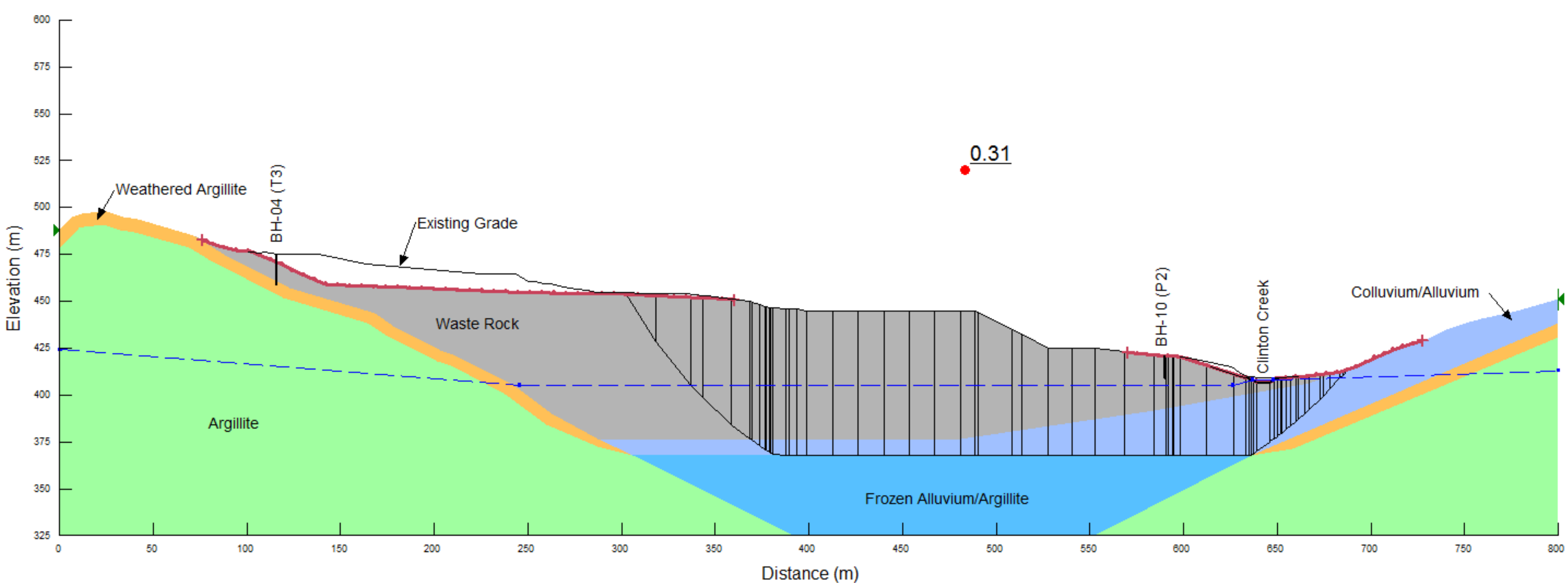
Figure C.3

Clinton Creek Waste Rock Dump - Section B-B'

Option D3 - Pseudo-Static Seismic Conditions

Name: Argillite Piezometric Line: 1
Name: Frozen Alluvium/Argillite Piezometric Line: 1
Name: Weathered Argillite S Unit Weight: 21.6 kN/m³ Cohesion': 0 kPa Phi': 22.2 ° Piezometric Line: 1
Name: Colluvium/Alluvium S Unit Weight: 19.1 kN/m³ Cohesion': 48 kPa Piezometric Line: 1
Name: Waste Rock S Unit Weight: 19.6 kN/m³ Cohesion': 0 kPa Phi': 18.8 ° Piezometric Line: 1

Horz Seismic Coef.: 0.154



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STABILITY ANALYSIS REPORT
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Section B-B'
Remediation Option D3
Pseudo-Static Seismic Conditions

PROJECT NO.	DWN	CKD	APVD	REV
704-ENG.WARC03039	CD	XL	AFR	0

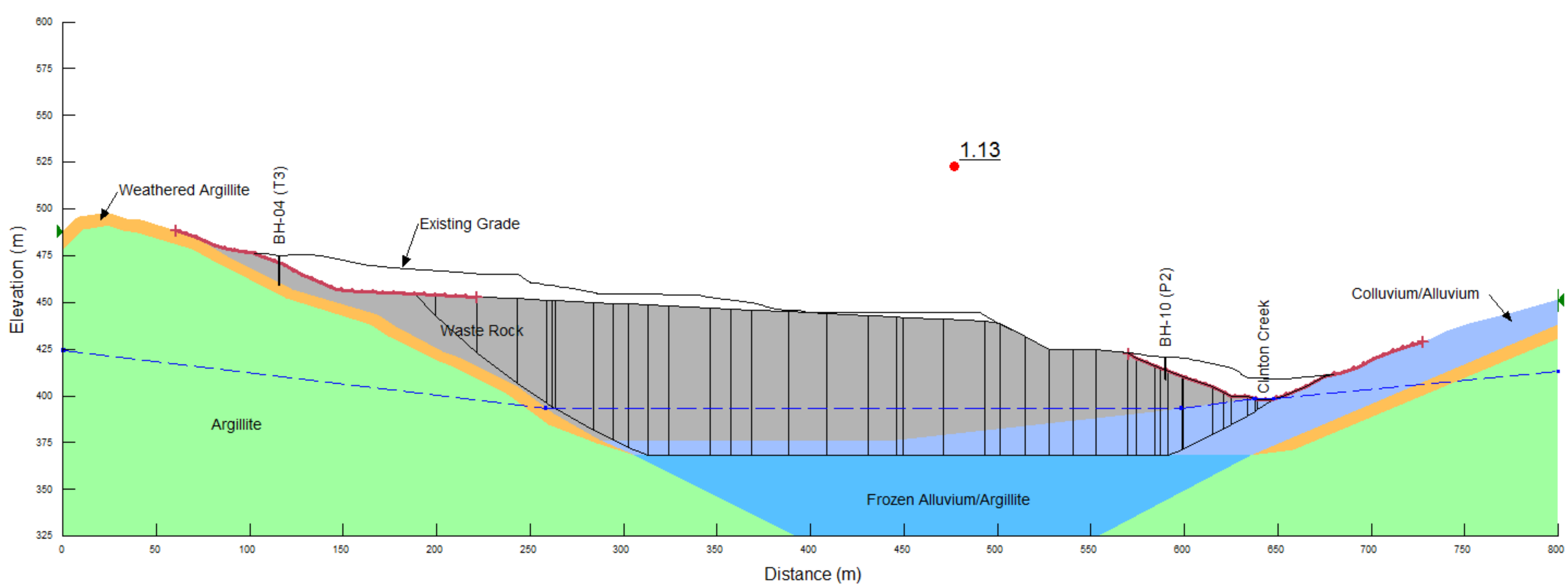
OFFICE	DATE
EBA-EDM	March 17, 2016

Figure C.4

Clinton Creek Waste Rock Dump - Section B-B'

Option E3 - Steady State Seepage Conditions

Name: Waste Rock Unit Weight: 19.6 kN/m³ Cohesion': 0 kPa Phi': 23 ° Piezometric Line: 1
Name: Colluvium/Alluvium Unit Weight: 19.1 kN/m³ Cohesion': 60 kPa Piezometric Line: 1
Name: Argillite Piezometric Line: 1
Name: Weathered Argillite Unit Weight: 21.6 kN/m³ Cohesion': 0 kPa Phi': 27 ° Piezometric Line: 1
Name: Frozen Alluvium/Argillite Piezometric Line: 1



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NOTES

STATUS

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STABILITY ANALYSIS REPORT
ABANDONED CLINTON CREEK MINE, YUKON

Section B-B'
Remediation Option E3
Steady State Seepage Conditions

PROJECT NO.	DWN	CKD	APVD	REV
704-ENG.WARC03039	CD	XL	AFR	0

OFFICE	DATE
EBA-EDM	March 17, 2016

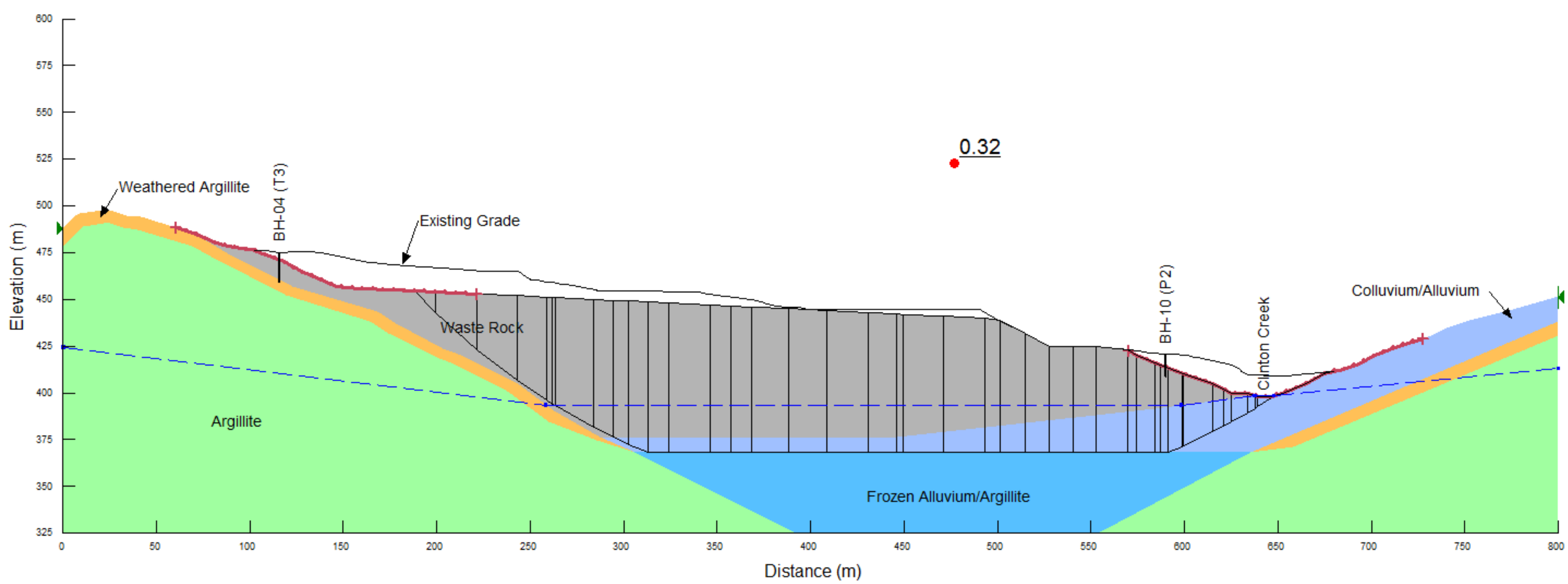
Figure C.5

Clinton Creek Waste Rock Dump - Section B-B'

Option E3 - Pseudo-Static Seismic Conditions

Name: Argillite Piezometric Line: 1
Name: Frozen Alluvium/Argillite Piezometric Line: 1
Name: Colluvium/Alluvium S Unit Weight: 19.1 kN/m³ Cohesion': 48 kPa Piezometric Line: 1
Name: Waste Rock S Unit Weight: 19.6 kN/m³ Cohesion': 0 kPa Phi': 18.8 ° Piezometric Line: 1
Name: Weathered Argillite S Unit Weight: 21.6 kN/m³ Cohesion': 0 kPa Phi': 22.2 ° Piezometric Line: 1

Horz Seismic Coef.: 0.154



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STABILITY ANALYSIS REPORT
ABANDONED CLINTON CREEK MINE, YUKON

Section B-B'
Remediation Option E3
Pseudo-Static Seismic Conditions

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704-ENG.WARC03039	CD	XL	AFR	0

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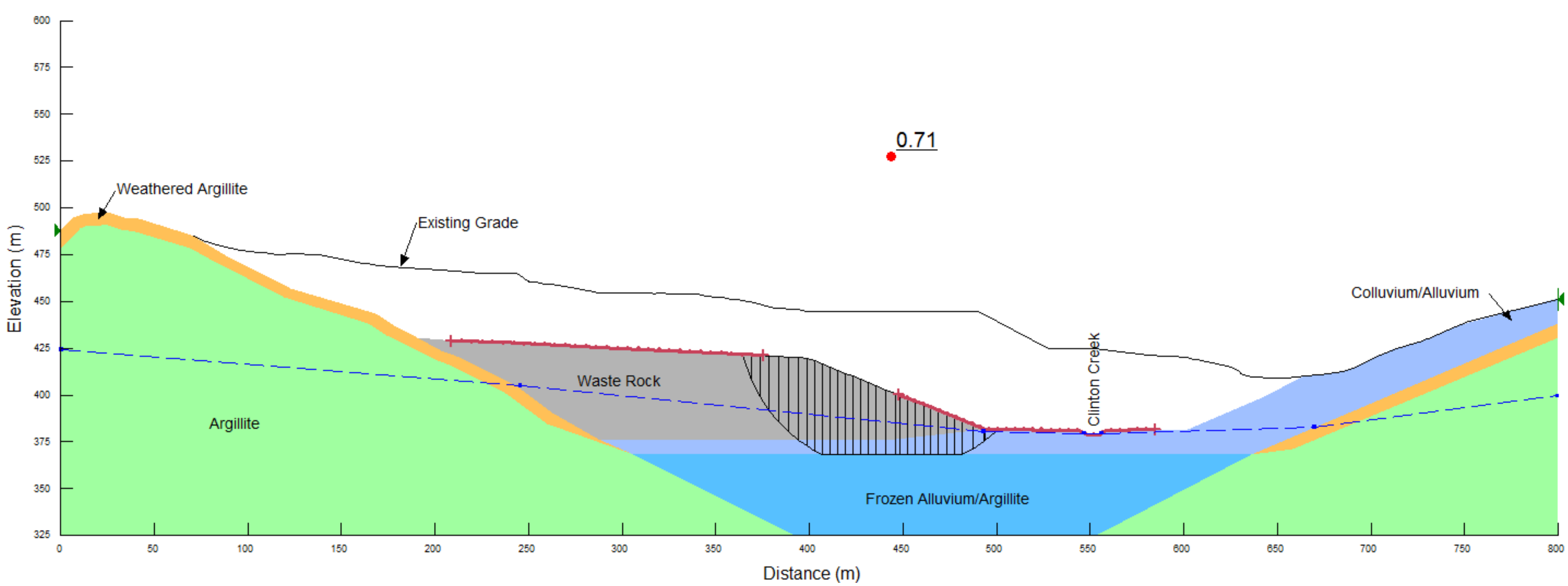
Figure C.6

Clinton Creek Waste Rock Dump - Section B-B'

Option F - Steady State Seepage Conditions

Name: Waste Rock Unit Weight: 19.6 kN/m³ Cohesion': 0 kPa Phi': 23 ° Piezometric Line: 1
Name: Colluvium/Alluvium Unit Weight: 19.1 kN/m³ Cohesion': 60 kPa Piezometric Line: 1
Name: Argillite Piezometric Line: 1
Name: Weathered Argillite Unit Weight: 21.6 kN/m³ Cohesion': 0 kPa Phi': 27 ° Piezometric Line: 1
Name: Frozen Alluvium/Argillite Piezometric Line: 1



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STABILITY ANALYSIS REPORT
ABANDONED CLINTON CREEK MINE, YUKON

Section B-B'
Remediation Option F
Steady State Seepage Conditions

PROJECT NO.	DWN	CKD	APVD	REV
704-ENG.WARC03039	CD	XL	AFR	0

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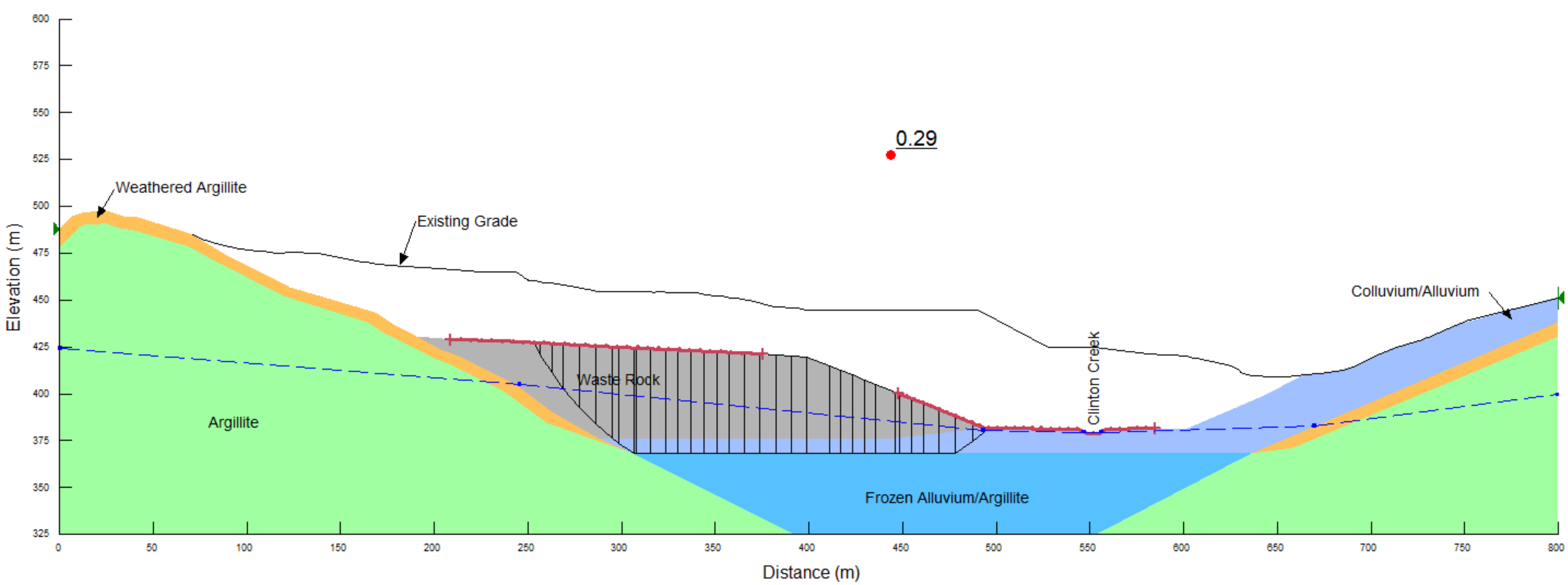
Figure C.7

Clinton Creek Waste Rock Dump - Section B-B'

Option F - Pseudo-Static Seismic Conditions

Name: Argillite   Piezometric Line: 1
Name: Frozen Alluvium/Argillite   Piezometric Line: 1
Name: Colluvium/Alluvium S   Unit Weight: 19.1 kN/m³   Cohesion': 48 kPa   Piezometric Line: 1
Name: Waste Rock S   Unit Weight: 19.6 kN/m³   Cohesion': 0 kPa   Phi': 18.8 °   Piezometric Line: 1
Name: Weathered Argillite S   Unit Weight: 21.6 kN/m³   Cohesion': 0 kPa   Phi': 22.2 °   Piezometric Line: 1

Horz Seismic Coef.: 0.154



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STABILITY ANALYSIS REPORT
ABANDONED CLINTON CREEK MINE, YUKON

Section B-B'
Remediation Option F
Pseudo-Static Seismic Conditions

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704-ENG.WARC03039	CD	XL	AFR	0

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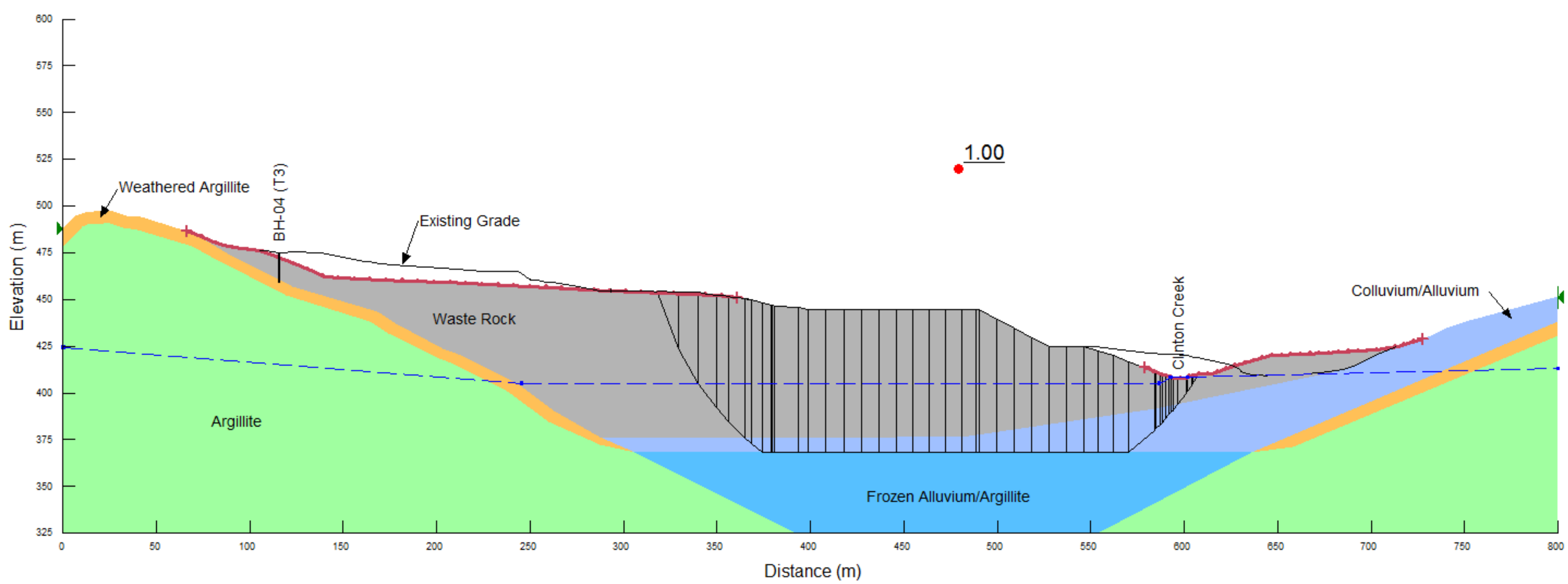
Figure C.8

Clinton Creek Waste Rock Dump - Section B-B'

Option I2 - Steady State Seepage Conditions

Name: Waste Rock Unit Weight: 19.6 kN/m³ Cohesion': 0 kPa Phi': 23 ° Piezometric Line: 1
Name: Colluvium/Alluvium Unit Weight: 19.1 kN/m³ Cohesion': 60 kPa Piezometric Line: 1
Name: Argillite Piezometric Line: 1
Name: Weathered Argillite Unit Weight: 21.6 kN/m³ Cohesion': 0 kPa Phi': 27 ° Piezometric Line: 1
Name: Frozen Alluvium/Argillite Piezometric Line: 1



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STABILITY ANALYSIS REPORT
ABANDONED CLINTON CREEK MINE, YUKON

Section B-B'
Remediation Option I2
Steady State Seepage Conditions

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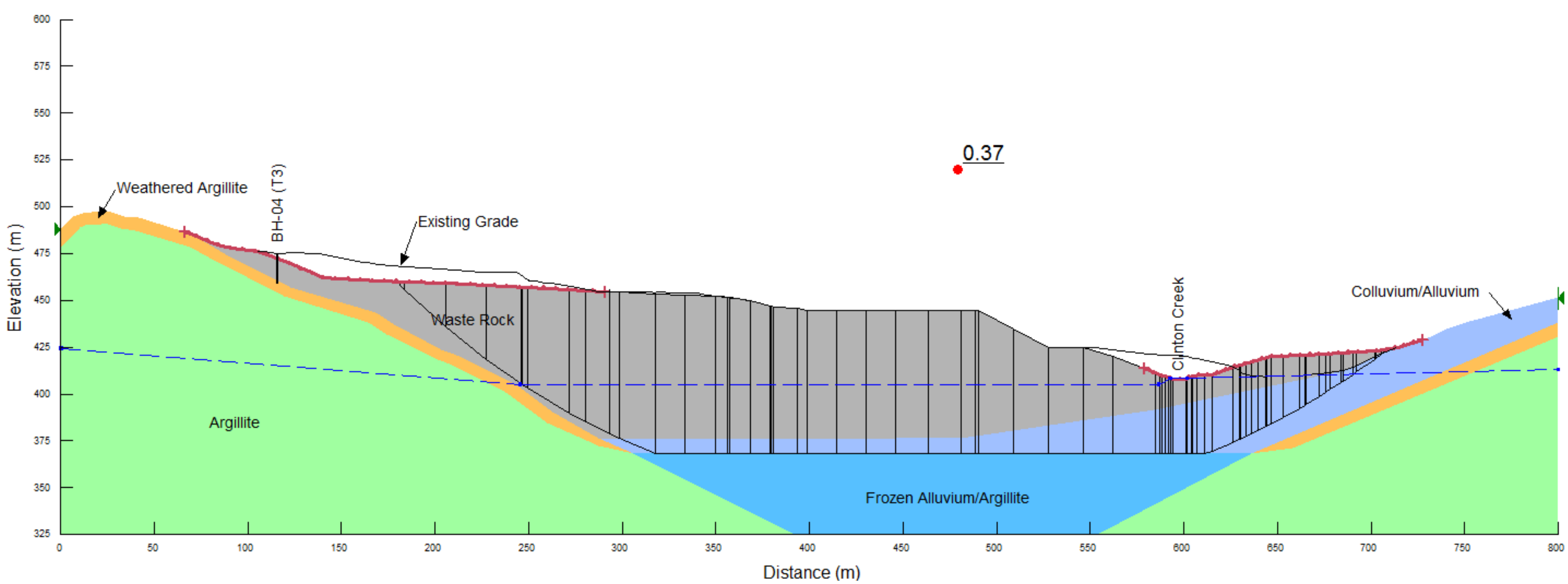
Figure C.9

Clinton Creek Waste Rock Dump - Section B-B'

Option I2 - Pseudo-Static Seismic Conditions

Name: Waste Rock Unit Weight: 19.6 kN/m³ Cohesion': 0 kPa Phi': 23 ° Piezometric Line: 1
Name: Argillite Piezometric Line: 1
Name: Frozen Alluvium/Argillite Piezometric Line: 1
Name: Colluvium/Alluvium S Unit Weight: 19.1 kN/m³ Cohesion': 48 kPa Piezometric Line: 1
Name: Waste Rock S Unit Weight: 19.6 kN/m³ Cohesion': 0 kPa Phi': 18.8 ° Piezometric Line: 1
Name: Weathered Argillite S Unit Weight: 21.6 kN/m³ Cohesion': 0 kPa Phi': 22.2 ° Piezometric Line: 1

Horz Seismic Coef.: 0.154



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STABILITY ANALYSIS REPORT
ABANDONED CLINTON CREEK MINE, YUKON

Section B-B'
Remediation Option I2
Pseudo-Static Seismic Conditions

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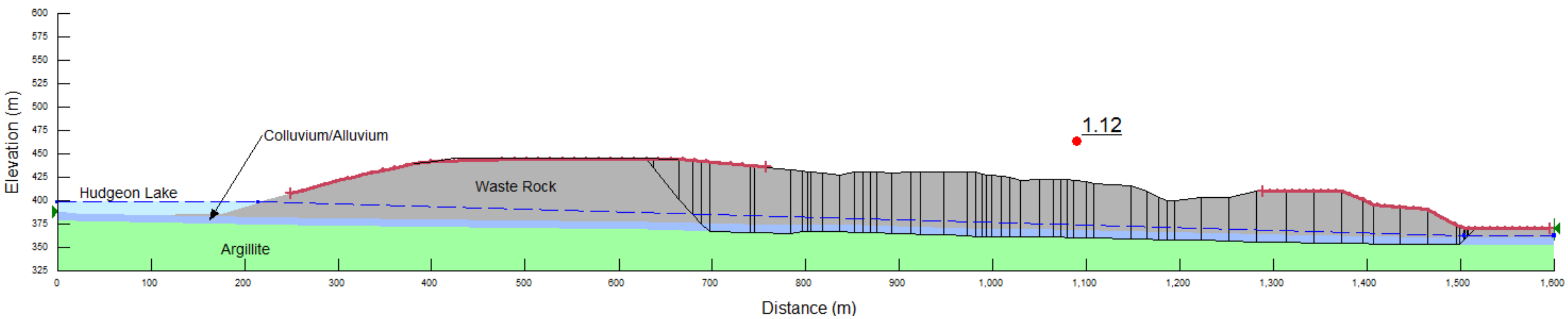
Figure C.10

Clinton Creek Waste Rock Dump - Section F-F'

Option E3 - Steady State Seepage Conditions

Name: Frozen Alluvium/Argillite Piezometric Line: 1
Name: Colluvium/Alluvium 60 Piezometric Line: 1 Unit Weight: 19.1 kN/m³ Cohesion': 60 kPa
Name: Waste Rock Piezometric Line: 1 Unit Weight: 19.6 kN/m³ Cohesion': 0 kPa Phi': 23 °



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STABILITY ANALYSIS REPORT
ABANDONED CLINTON CREEK MINE, YUKON

Section F-F'
Remediation Option E3
Steady State Seepage Conditions

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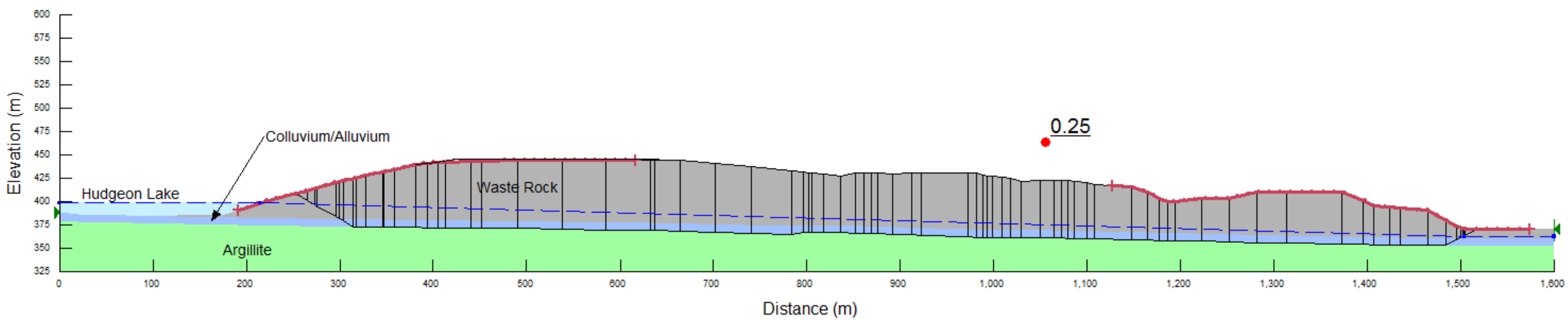
Figure C.11

Clinton Creek Waste Rock Dump - Section F-F'

Option E3 - Pseudo-Static Seismic Conditions

Name: Frozen Alluvium/Argillite Piezometric Line: 1
Name: Colluvium/Alluvium S Piezometric Line: 1 Unit Weight: 19.1 kN/m³ Cohesion': 48 kPa
Name: Waste Rock S Piezometric Line: 1 Unit Weight: 19.6 kN/m³ Cohesion': 0 kPa Phi': 18.8 °

Horz Seismic Coef.: 0.154



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STABILITY ANALYSIS REPORT
ABANDONED CLINTON CREEK MINE, YUKON

Section F-F'
Remediation Option E3
Pseudo-Static Seismic Conditions

PROJECT NO.	DWN	CKD	APVD	REV
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OFFICE	DATE
EBA-EDM	March 17, 2016

Figure C.12

APPENDIX D

WOLVERINE CREEK STABILITY – BACK ANALYSIS AND ANALYSIS OF CLOSURE OPTIONS

MATERIAL INFORMATION

Loading Condition	Color	Material	ϕ' (°)	c' (kPa)	γ (kN/m^3)	Piezometric Condition
Steady State Seepage Parameters		Colluvium	17.5	0	19.1	N/A
		Colluvium (2)	Impenetrable			
		Tailings	26	0	18.1	N/A
		Compacted Tailings	32	0	19.0	N/A
		Compacted Waste Rock	34	0	20.0	N/A
Pseudo-Static Seismic Parameters		Colluvium S	14.1	0	19.1	N/A
		Tailings S	21.3	0	18.1	N/A
		Compacted Tailings S	26.6	0	19.0	N/A
		Compacted Waste Rock S	28.4	0	20.0	N/A
Unchanged Parameters		Weathered Argillite	Impenetrable			
		Argillite	Impenetrable			

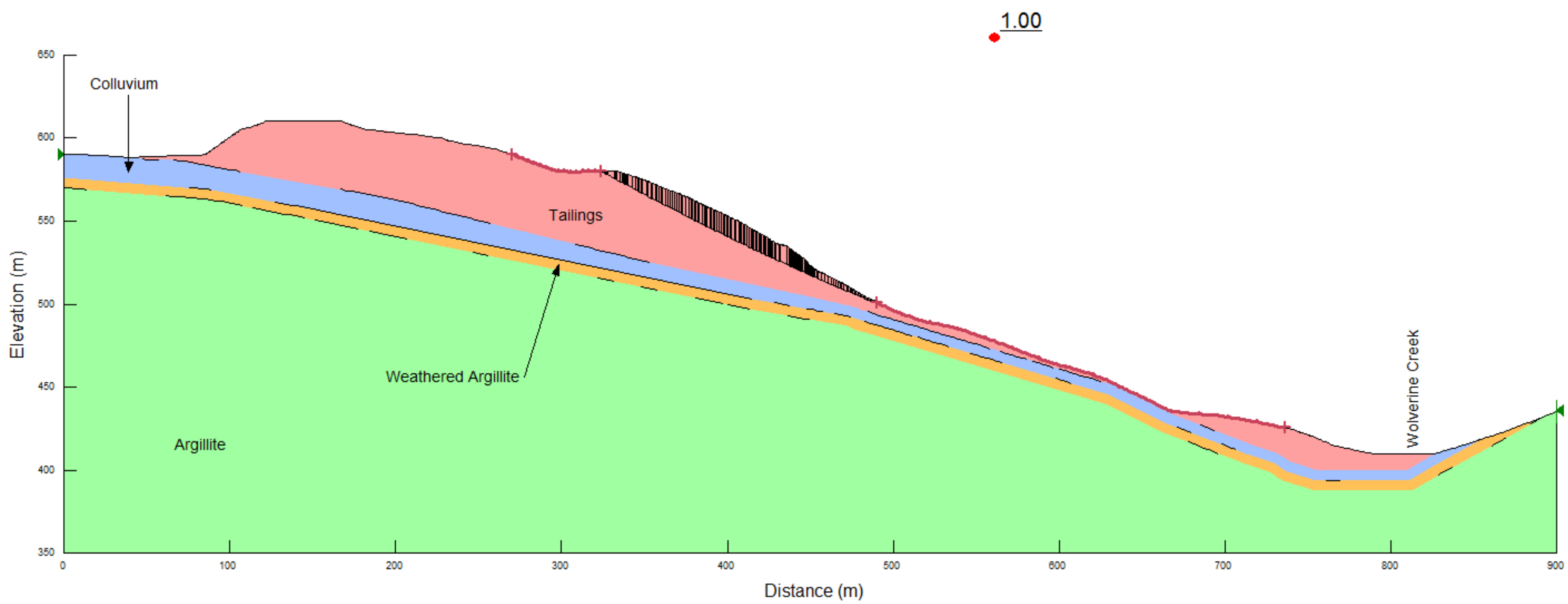
STABILITY ANALYSIS RESULTS – EXISTING COND./REMEDIATION OPTIONS

Figure	Section	Model	Loading Condition	Phreatic Surface Level (masl.)	Horizontal Acceleration (g)	FOS
D.1	Section E-E'	Back Analysis	Steady State Seepage	N/A	0	1.00
D.2	Section D-D'	Back Analysis	Steady State Seepage	N/A	0	1.01
D.3	Section E-E'	Back Analysis	Steady State Seepage	N/A	0	1.05
D.4	Section E-E'	Existing Conditions	Pseudo-Static Seismic	N/A	0.06	0.70
D.5	Section D-D'	Existing Conditions	Pseudo-Static Seismic	N/A	0.06	0.67
D.6	Section E-E'	Option E/E2	Steady State Seepage	N/A	0	1.30
D.7	Section E-E'	Option E/E2	Pseudo-Static Seismic	N/A	0.06	0.85
D.8	Section D-D'	Option E/E2	Steady State Seepage	N/A	0	1.31
D.9	Section D-D'	Option E/E2	Pseudo-Static Seismic	N/A	0.06	0.85
D.10	Section G-G'	Existing Conditions (Option A)	Steady State Seepage	409 (at Intake)	0	1.83
D.11	Section G-G'	Existing Conditions (Option A)	Pseudo-Static Seismic	409 (at Intake)	0.06	1.03
D.12	Section G-G'	Option B	Steady State Seepage	404 (at Intake)	0	2.00
D.13	Section G-G'	Option B	Pseudo-Static Seismic	404 (at Intake)	0.06	1.12
D.14	Section G-G'	Option C	Steady State Seepage	422 (at Intake)	0	1.25
D.15	Section G-G'	Option C	Pseudo-Static Seismic	422 (at Intake)	0.06	0.71
D.16	Section G-G'	Option D	Steady State Seepage	404 (at Intake)	0	1.93
D.17	Section G-G'	Option D	Pseudo-Static Seismic	404 (at Intake)	0.06	1.10
D.18	Section G-G'	Option E	Steady State Seepage	404 (at Intake)	0	1.95
D.19	Section G-G'	Option E	Pseudo-Static Seismic	404 (at Intake)	0.06	1.10
D.20	Section G-G'	Option D2	Steady State Seepage	415 (at Intake)	0	1.56
D.21	Section G-G'	Option D2	Pseudo-Static Seismic	415 (at Intake)	0.06	0.82
D.22	Section G-G'	Option E2	Steady State Seepage	422 (at Intake)	0	1.20
D.23	Section G-G'	Option E2	Pseudo-Static Seismic	422 (at Intake)	0.06	0.69

Wolverine Creek Tailings North Lobe - Section E-E'

Steady State Seepage Conditions

Name: Argillite
Name: Weathered Argillite
Name: Tailings Unit Weight: 18.1 kN/m³ Cohesion': 0 kPa Phi': 26 °
Name: Colluvium (2)



LEGEND	NOTES		STABILITY ANALYSIS REPORT ABANDONED CLINTON CREEK MINE, YUKON
	STATUS	Yukon Government Energy, Mines & Resources Assessment & Abandoned Mines Branch; TETRA TECH EBA	Section E-E' Back Analysis of Tailings Steady State Seepage Conditions

PROJECT NO.	DWN	CKD	APVD	REV	
704-ENG.WARC03039	CD	XL	AFR	0	Figure D.1
OFFICE	DATE				
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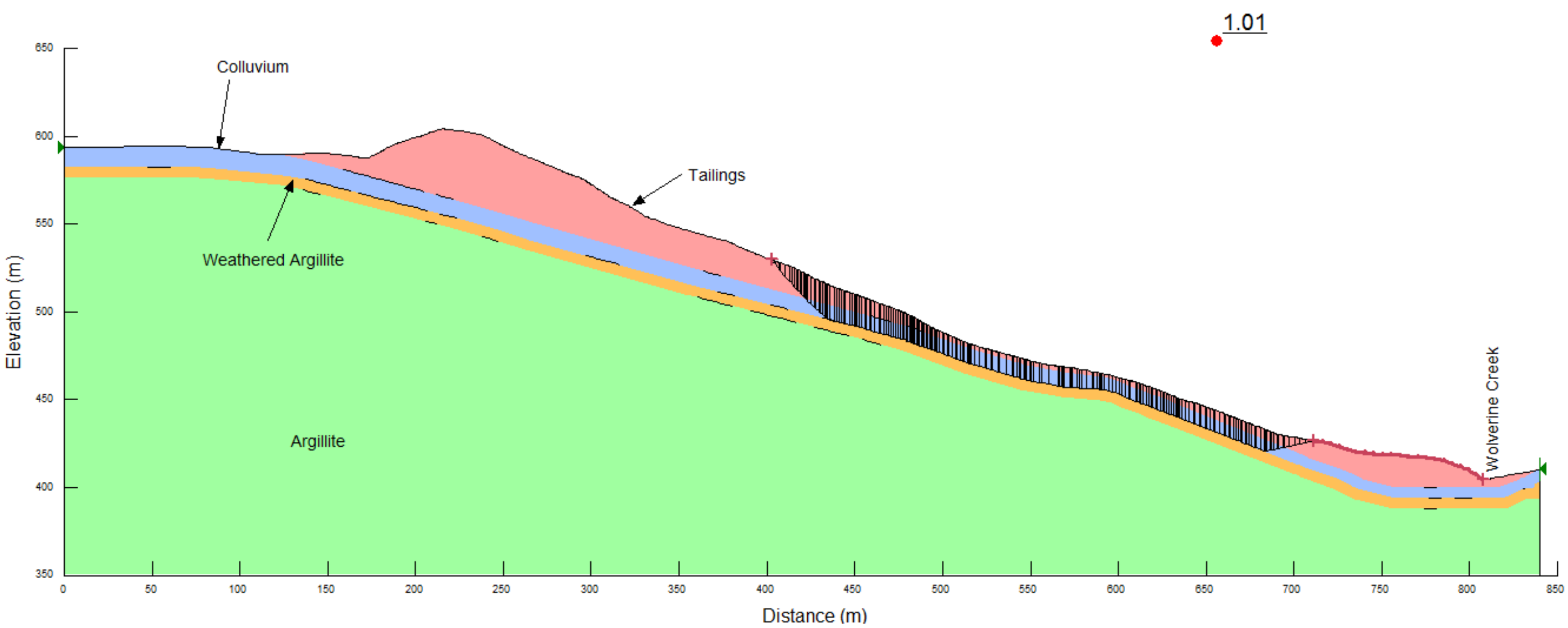
Wolverine Creek Tailings South Lobe - Section D-D'

Steady State Seepage Conditions

Name: Argillite
Name: Weathered Argillite
Name: Tailings Unit Weight: 18.1 kN/m³ Cohesion': 0 kPa Phi': 26 °
Name: Colluvium Unit Weight: 19.1 kN/m³ Cohesion': 0 kPa Phi': 17.5 °



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NOTES

STATUS

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STABILITY ANALYSIS REPORT
ABANDONED CLINTON CREEK MINE, YUKON

Section D-D'
Back Analysis of Colluvium
Steady State Seepage Conditions

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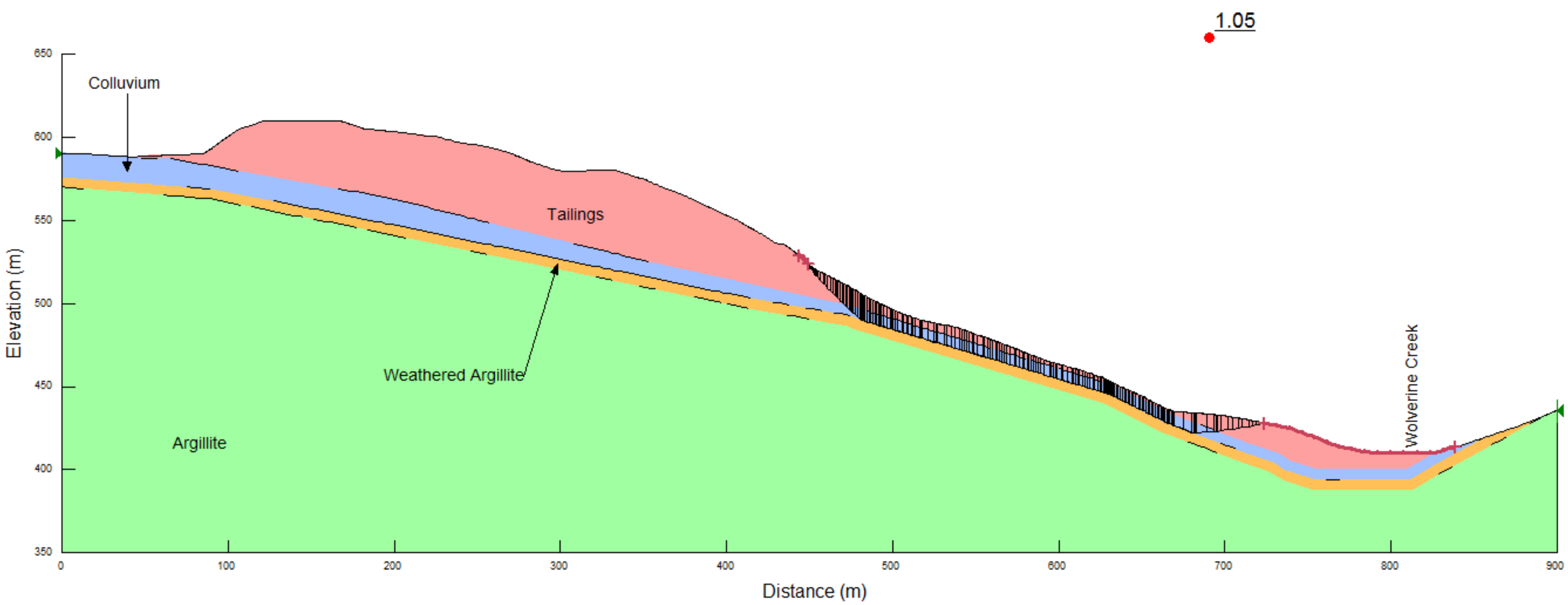
Figure D.2

Wolverine Creek Tailings North Lobe - Section E-E'

Steady State Seepage Conditions

Name: Argillite
Name: Weathered Argillite
Name: Tailings Unit Weight: 18.1 kN/m³ Cohesion': 0 kPa Phi': 26 °
Name: Colluvium Unit Weight: 19.1 kN/m³ Cohesion': 0 kPa Phi': 17.5 °



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ABANDONED CLINTON CREEK MINE, YUKON

Section E-E'
Back Analysis of Colluvium
Steady State Seepage Conditions

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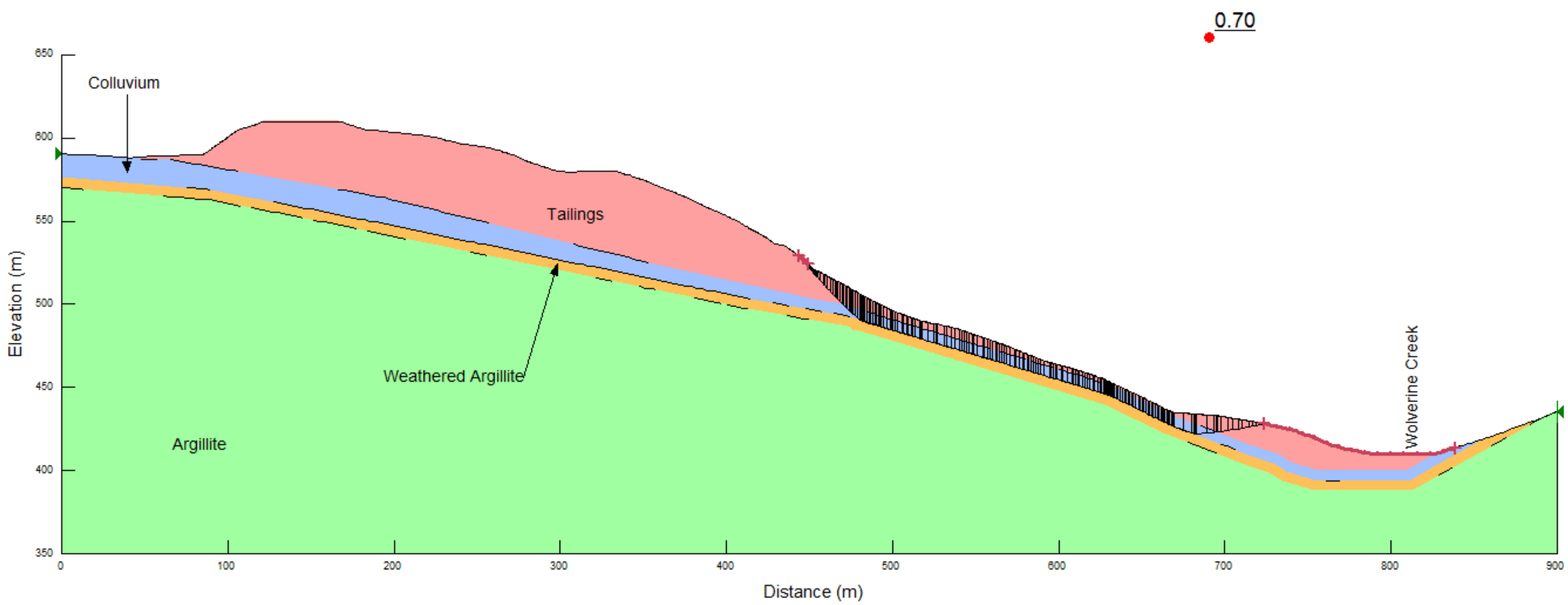
Figure D.3

Wolverine Creek Tailings North Lobe - Section E-E'

Pseudo-Static Seismic Conditions

Name: Argillite
Name: Weathered Argillite
Name: Colluvium S   Unit Weight: 19.1 kN/m³   Cohesion': 0 kPa   Phi': 14.1 °
Name: Tailings S   Unit Weight: 18.1 kN/m³   Cohesion': 0 kPa   Phi': 21.3 °

Horz Seismic Coef.: 0.06



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ABANDONED CLINTON CREEK MINE, YUKON

Section E-E'
Existing Conditions
Pseudo-Static Seismic Conditions

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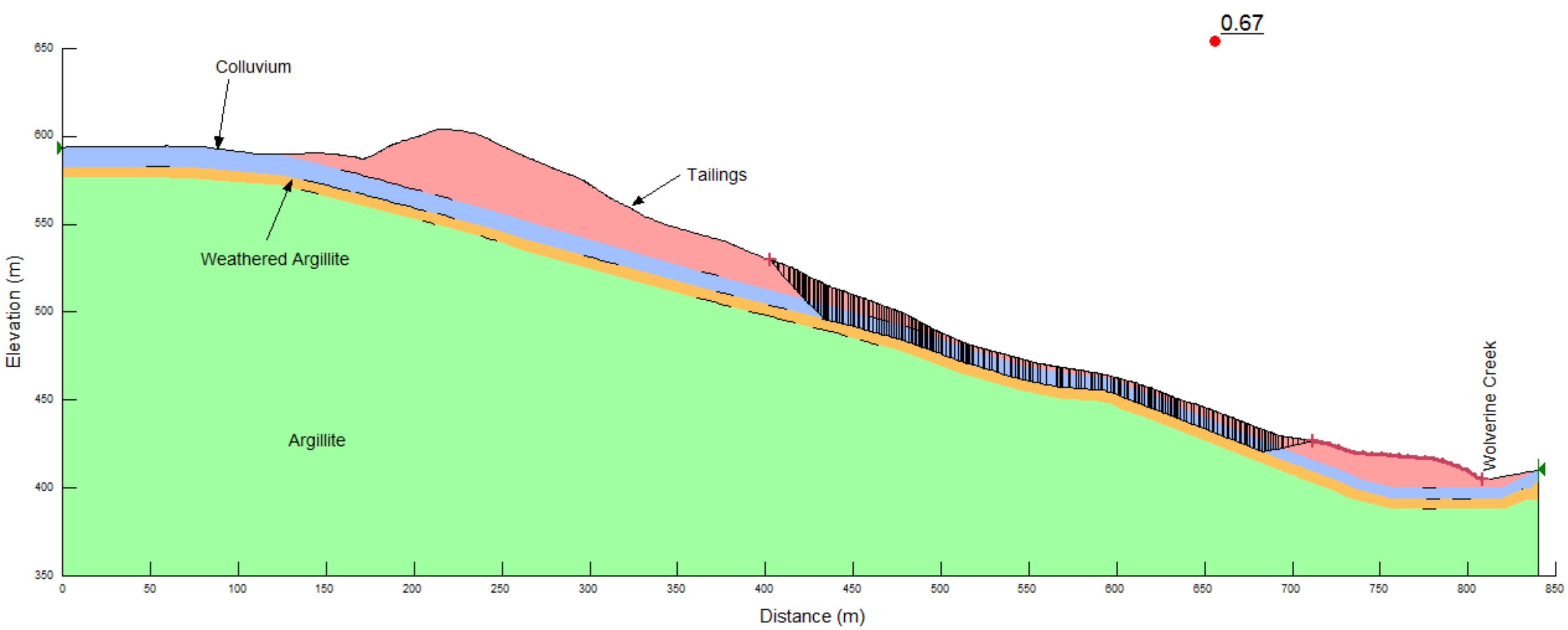
Figure D.4

Wolverine Creek Tailings South Lobe - Section D-D'

Pseudo-Static Seismic Conditions

Name: Argillite
Name: Weathered Argillite
Name: Colluvium S   Unit Weight: 19.1 kN/m³   Cohesion': 0 kPa   Phi': 14.1 °
Name: Tailings S   Unit Weight: 18.1 kN/m³   Cohesion': 0 kPa   Phi': 21.3 °

Horz Seismic Coef.: 0.06



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ABANDONED CLINTON CREEK MINE, YUKON

Section D-D'
Existing Conditions
Pseudo-Static Seismic Conditions

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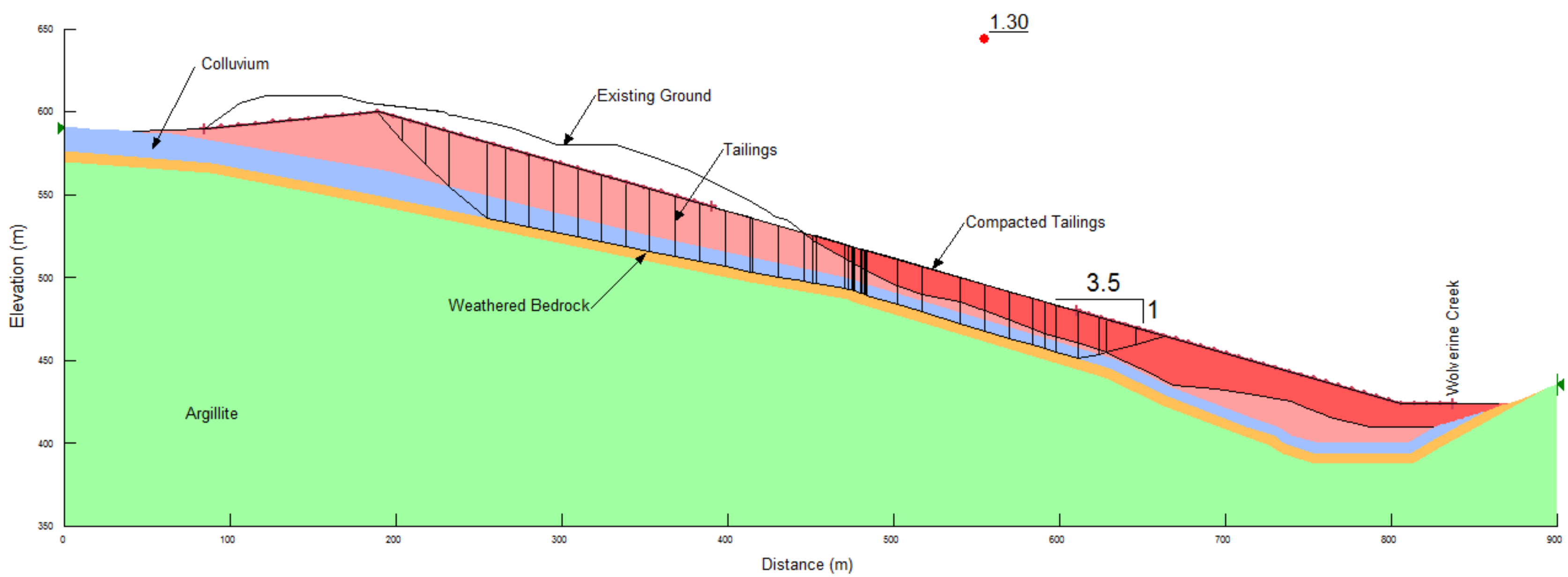
Figure D.5

Wolverine Creek Tailings North Lobe - Section E-E'

Option E/E2 - Steady State Seepage Conditions

Name: Argillite
Name: Weathered Argillite
Name: Colluvium Unit Weight: 19.1 kN/m³ Cohesion': 0 kPa Phi': 17.5 °
Name: Tailings Unit Weight: 18.1 kN/m³ Cohesion': 0 kPa Phi': 26 °
Name: Tailings (Compacted) Unit Weight: 19 kN/m³ Cohesion': 0 kPa Phi': 32 °



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Section E-E'
Remediation Option E/E2
Steady State Seepage Conditions

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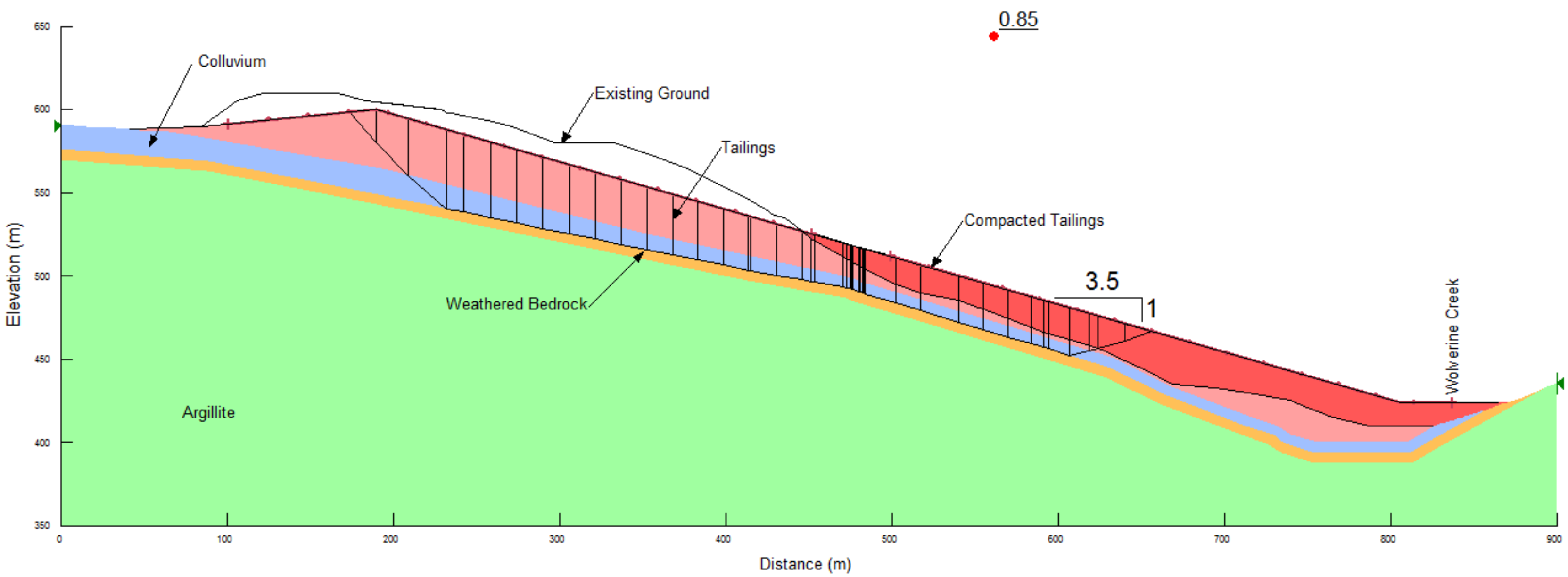
Figure D.6

Wolverine Creek Tailings North Lobe - Section E-E'

Option E/E2 - Pseudo-Static Seismic Conditions

Name: Argillite
Name: Weathered Argillite
Name: Colluvium S   Unit Weight: 19.1 kN/m³   Cohesion': 0 kPa   Phi': 14.2 °
Name: Tailings S   Unit Weight: 18.1 kN/m³   Cohesion': 0 kPa   Phi': 21.3 °
Name: Tailings S (Compacted)   Unit Weight: 19 kN/m³   Cohesion': 0 kPa   Phi': 26.6 °

Horz Seismic Coef.: 0.06



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Remediation Option E/E2
Pseudo-Static Seismic Conditions

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704-ENG.WARC03039	CD	XL	AFR	0

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Figure D.7

Wolverine Creek Tailings South Lobe - Section D-D'

Option E/E2 - Steady State Seepage Conditions

Name: Argillite
Name: Weathered Argillite
Name: Colluvium Unit Weight: 19.1 kN/m³ Cohesion': 0 kPa Phi': 17.5 °
Name: Tailings Unit Weight: 18.1 kN/m³ Cohesion': 0 kPa Phi': 26 °
Name: Tailings (Compacted) Unit Weight: 19 kN/m³ Cohesion': 0 kPa Phi': 32 °

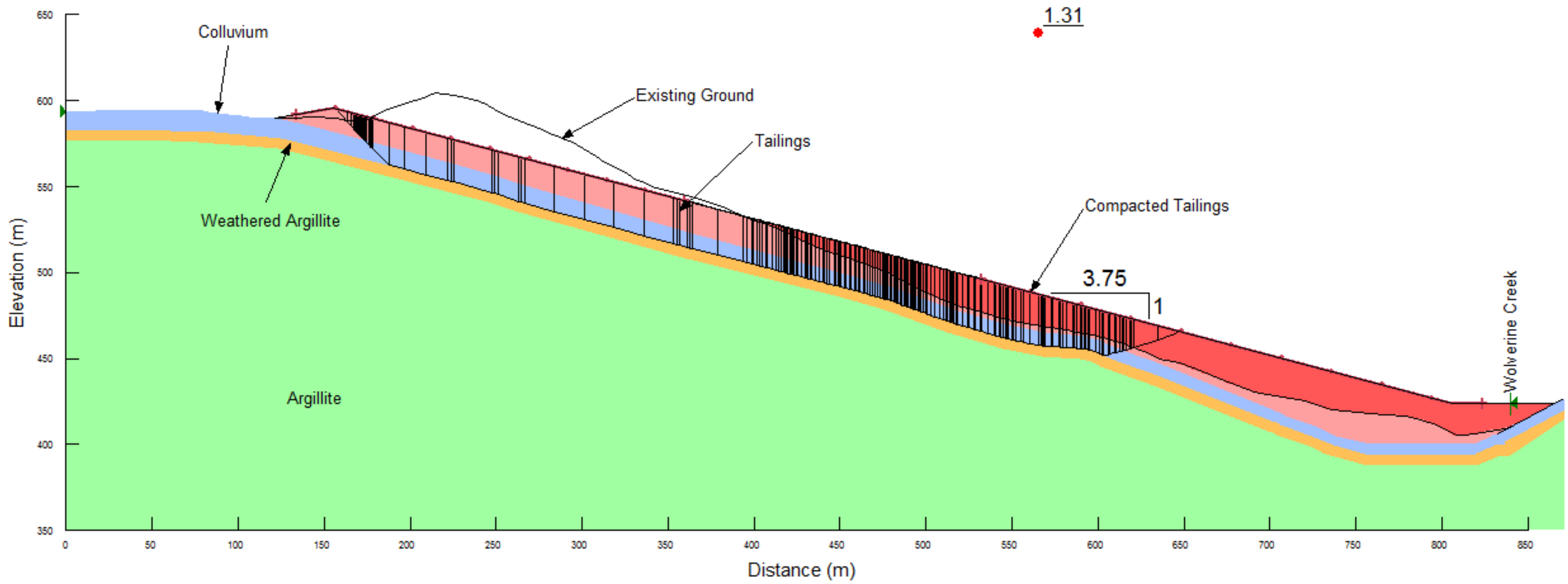

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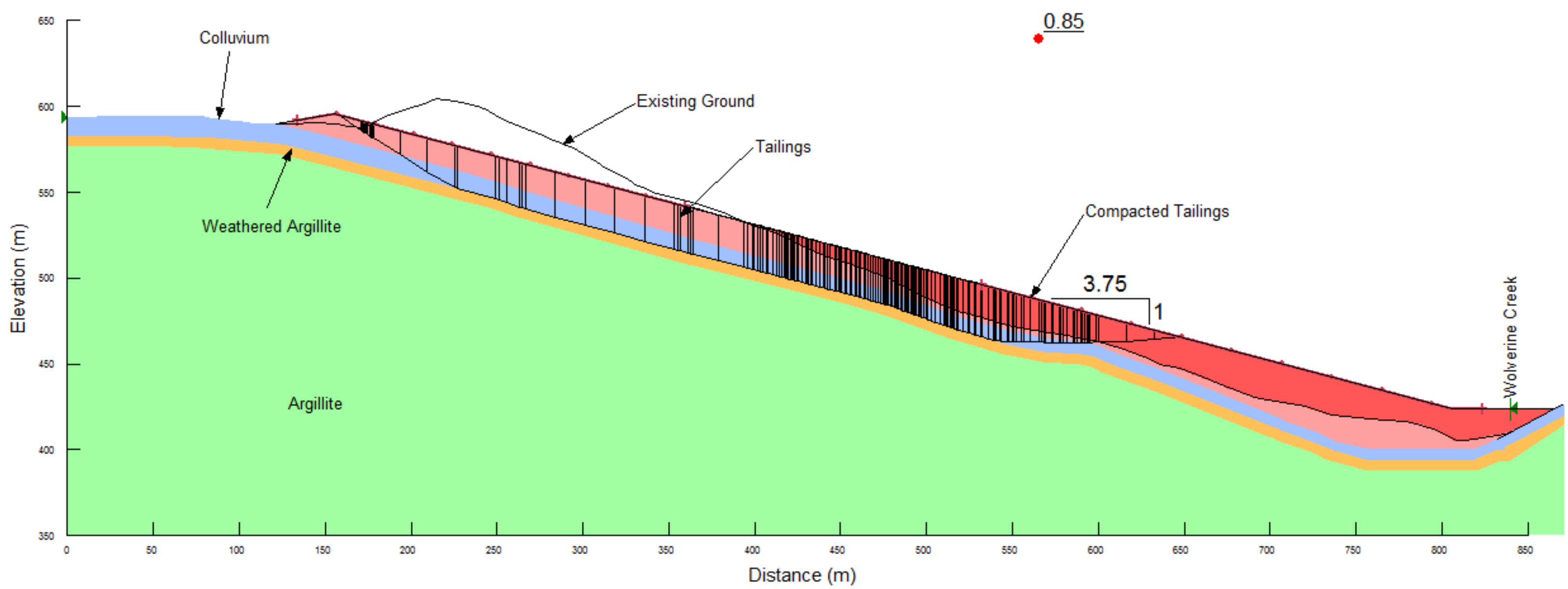
Figure D.8

Wolverine Creek Tailings South Lobe - Section D-D'

Option E/E2 - Pseudo-Static Seismic Conditions

Name: Argillite
Name: Weathered Argillite
Name: Colluvium S   Unit Weight: 19.1 kN/m³   Cohesion': 0 kPa   Phi': 14.2 °
Name: Tailings S   Unit Weight: 18.1 kN/m³   Cohesion': 0 kPa   Phi': 21.3 °
Name: Tailings S (Compacted)   Unit Weight: 19 kN/m³   Cohesion': 0 kPa   Phi': 26.6 °

Horz Seismic Coef.: 0.06



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ABANDONED CLINTON CREEK MINE, YUKON

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Remediation Option E/E2
Pseudo-Static Seismic Conditions

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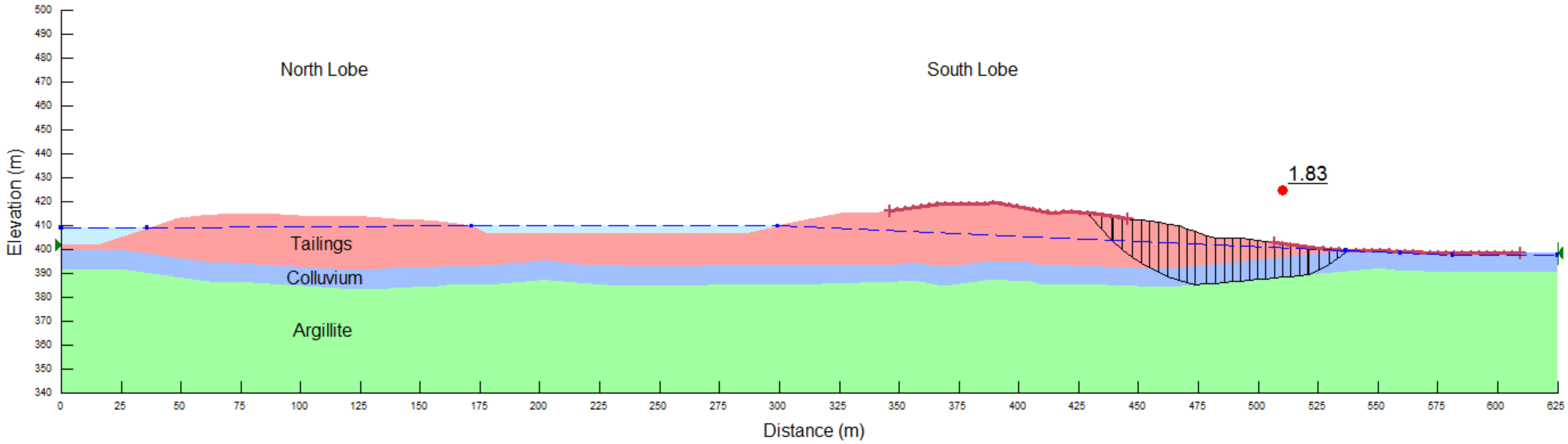
Figure D.9

Wolverine Tailings Pile - Section G-G'

Existing Conditions (Option A) - Steady State Seepage Conditions

Name: Colluvium Piezometric Line: 1 Unit Weight: 19.1 kN/m³ Cohesion': 0 kPa Phi': 17.5 °
Name: Tailings Piezometric Line: 1 Unit Weight: 18.1 kN/m³ Cohesion': 0 kPa Phi': 26 °
Name: Argillite Piezometric Line: 1



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Section G-G'
Existing Conditions (Remediation Option A)
Steady State Seepage Conditions

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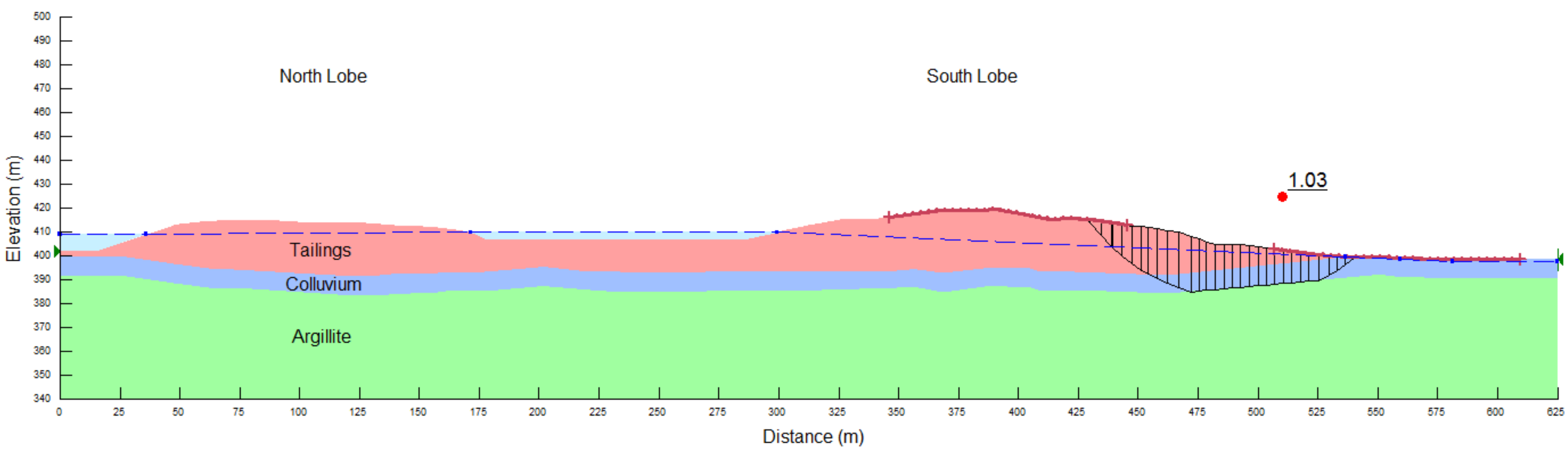
Figure D.10

Wolverine Tailings Pile - Section G-G'

Existing Conditions (Option A) - Pseudo-Static Seismic Conditions

Name: Colluvium S Piezometric Line: 1 Unit Weight: 19.1 kN/m³ Cohesion': 0 kPa Phi': 14.2 °
Name: Tailings S Piezometric Line: 1 Unit Weight: 18.1 kN/m³ Cohesion': 0 kPa Phi': 21.3 °
Name: Argillite Piezometric Line: 1

Horz Seismic Coef.: 0.06



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Section G-G'
Existing Conditions (Remediation Option A)
Pseudo-Static Seismic Conditions

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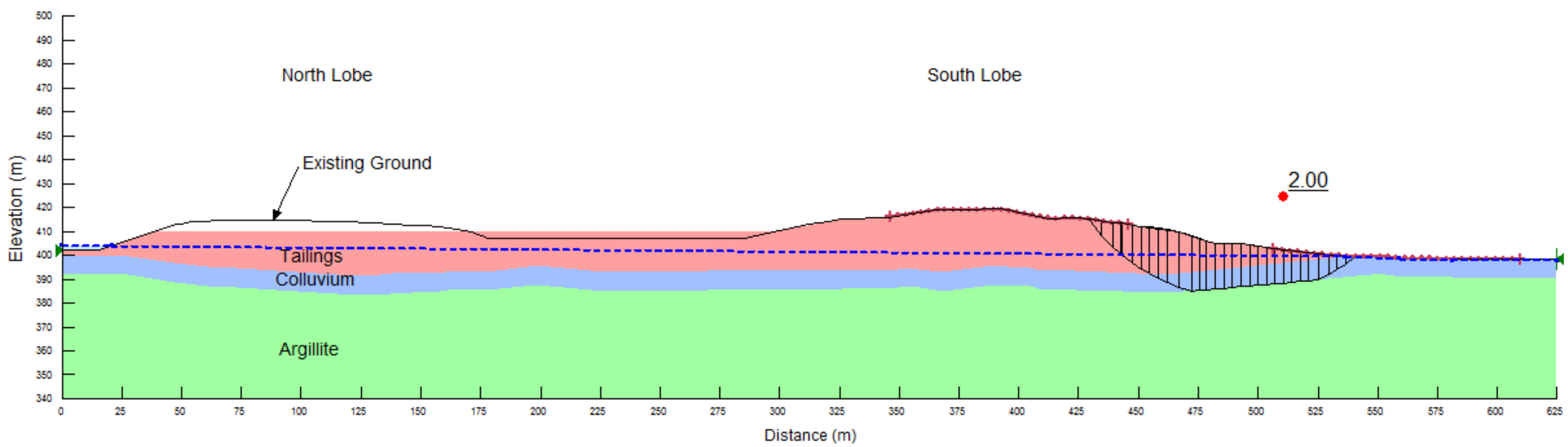
Figure D.11

Wolverine Tailings Pile - Section G-G'

Option B - Steady State Seepage Conditions

Name: Colluvium Piezometric Line: 1 Unit Weight: 19.1 kN/m³ Cohesion': 0 kPa Phi': 17.5 °
Name: Tailings Piezometric Line: 1 Unit Weight: 18.1 kN/m³ Cohesion': 0 kPa Phi': 26 °
Name: Argillite Piezometric Line: 1



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Section G-G'
Remediation Option B
Steady State Seepage Conditions

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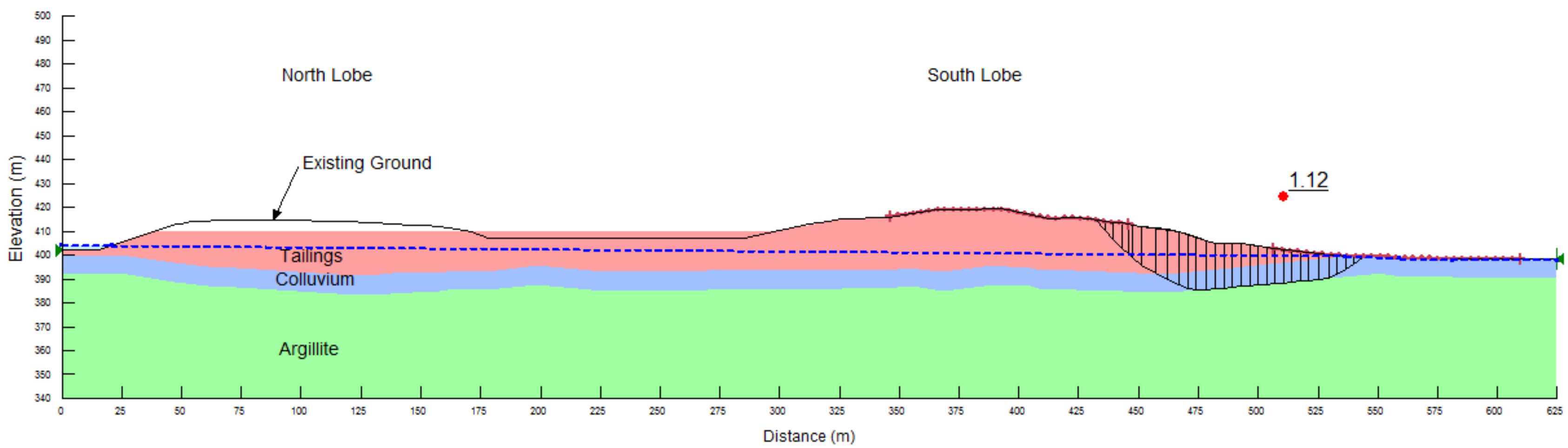
Figure D.12

Wolverine Tailings Pile - Section G-G'

Option B - Pseudo-Static Seismic Conditions

Name: Colluvium S   Piezometric Line: 1   Unit Weight: 19.1 kN/m³   Cohesion': 0 kPa   Phi': 14.2 °
Name: Tailings S   Piezometric Line: 1   Unit Weight: 18.1 kN/m³   Cohesion': 0 kPa   Phi': 21.3 °
Name: Argillite   Piezometric Line: 1

Horz Seismic Coef.: 0.06



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Section G-G'
Remediation Option B
Pseudo-Static Seismic Conditions

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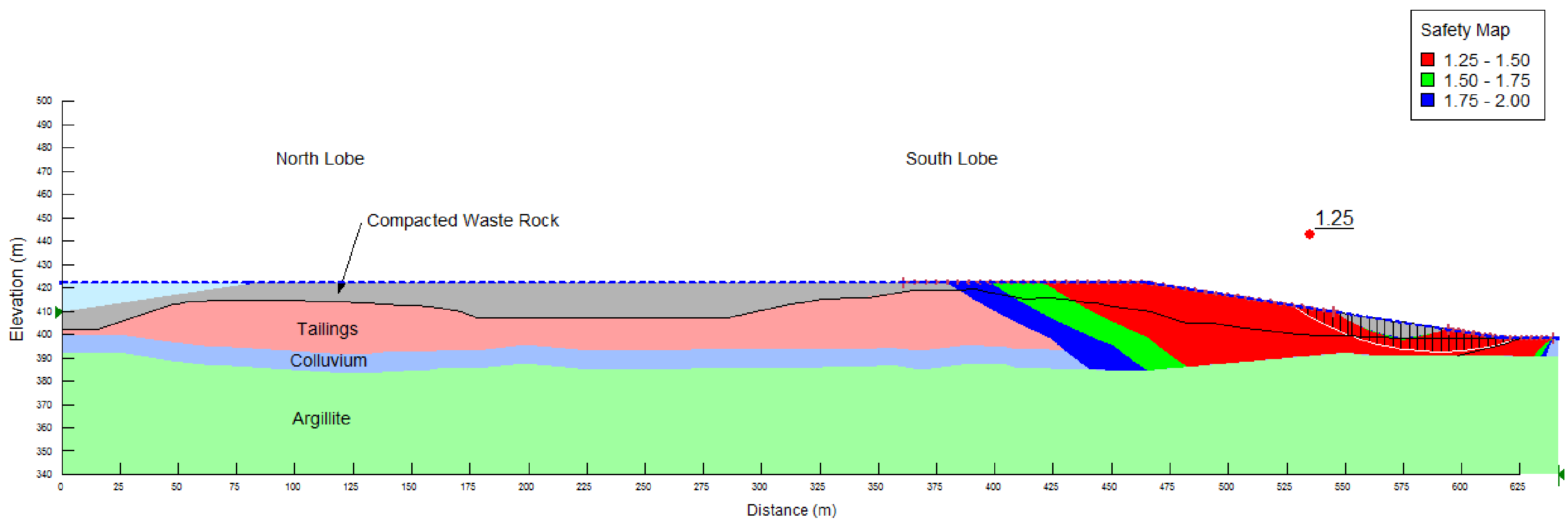
Figure D.13

Wolverine Tailings Pile - Section G-G'

Option C - Steady State Seepage Conditions

Name: Colluvium   Piezometric Line: 1   Unit Weight: 19.1 kN/m³   Cohesion': 0 kPa   Phi': 17.5 °
Name: Tailings   Piezometric Line: 1   Unit Weight: 18.1 kN/m³   Cohesion': 0 kPa   Phi': 26 °
Name: Argillite   Piezometric Line: 1
Name: Waste Rock (Compacted)   Piezometric Line: 1   Unit Weight: 20 kN/m³   Cohesion': 0 kPa   Phi': 34 °



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Section G-G'
Remediation Option C
Steady State Seepage Conditions

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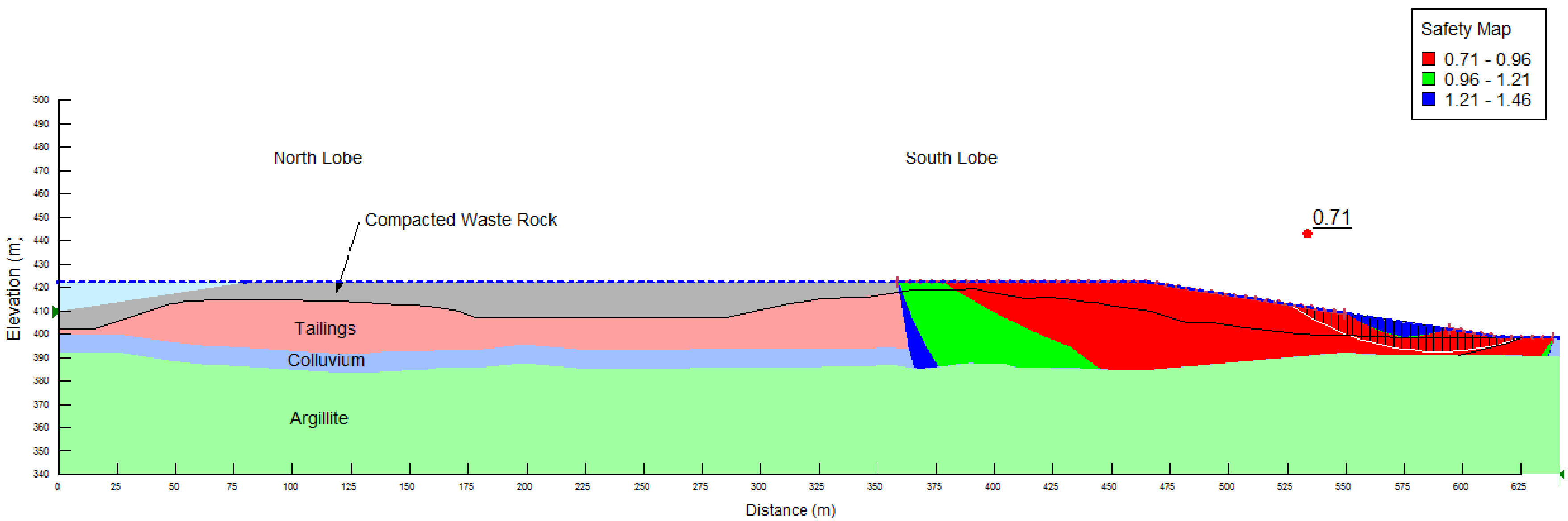
Figure D.14

Wolverine Tailings Pile - Section G-G'

Option C - Pseudo-Static Seismic Conditions

Name: Colluvium S   Piezometric Line: 1   Unit Weight: 19.1 kN/m³   Cohesion': 0 kPa   Phi': 14.2 °
Name: Tailings S   Piezometric Line: 1   Unit Weight: 18.1 kN/m³   Cohesion': 0 kPa   Phi': 21.3 °
Name: Argillite   Piezometric Line: 1
Name: Waste Rock S (Compacted)   Piezometric Line: 1   Unit Weight: 20 kN/m³   Cohesion': 0 kPa   Phi': 28.4 °

Horz Seismic Coef.: 0.06



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Section G-G'
Remediation Option C
Pseudo-Static Seismic Conditions

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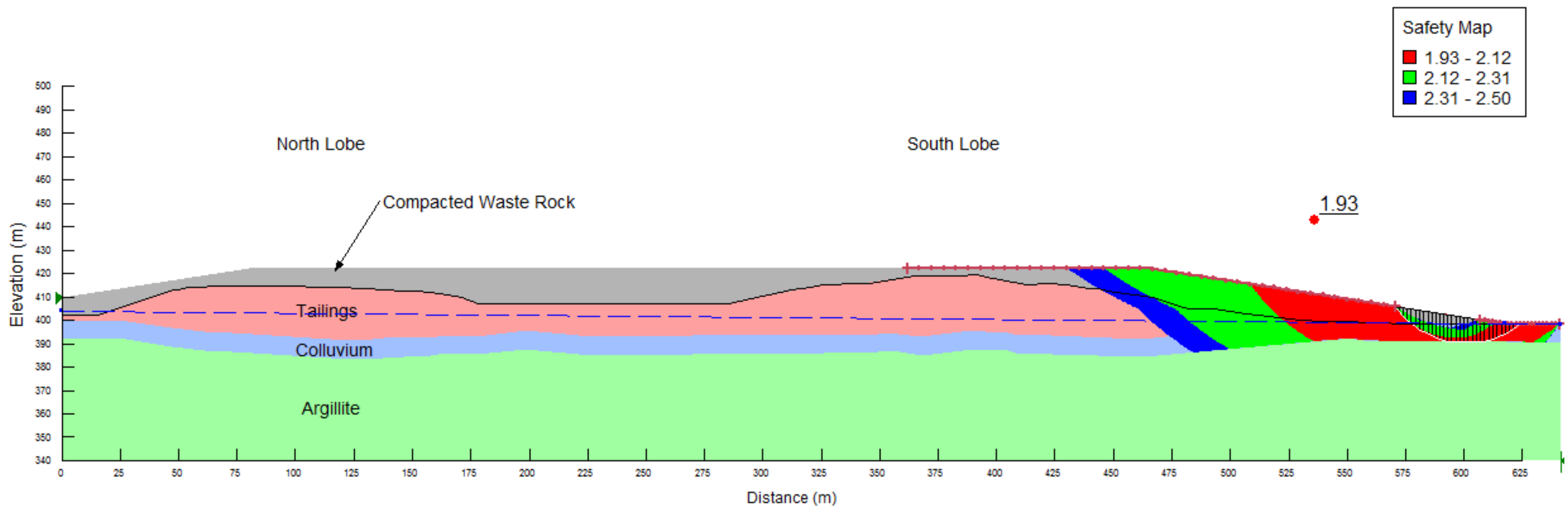
Figure D.15

Wolverine Tailings Pile - Section G-G'

Option D - Steady State Seepage Conditions

Name: Colluvium Piezometric Line: 1 Unit Weight: 19.1 kN/m³ Cohesion': 0 kPa Phi': 17.5 °
Name: Tailings Piezometric Line: 1 Unit Weight: 18.1 kN/m³ Cohesion': 0 kPa Phi': 26 °
Name: Argillite Piezometric Line: 1
Name: Waste Rock (Compacted) Piezometric Line: 1 Unit Weight: 20 kN/m³ Cohesion': 0 kPa Phi': 34 °



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Section G-G'
Remediation Option D
Steady State Seepage Conditions

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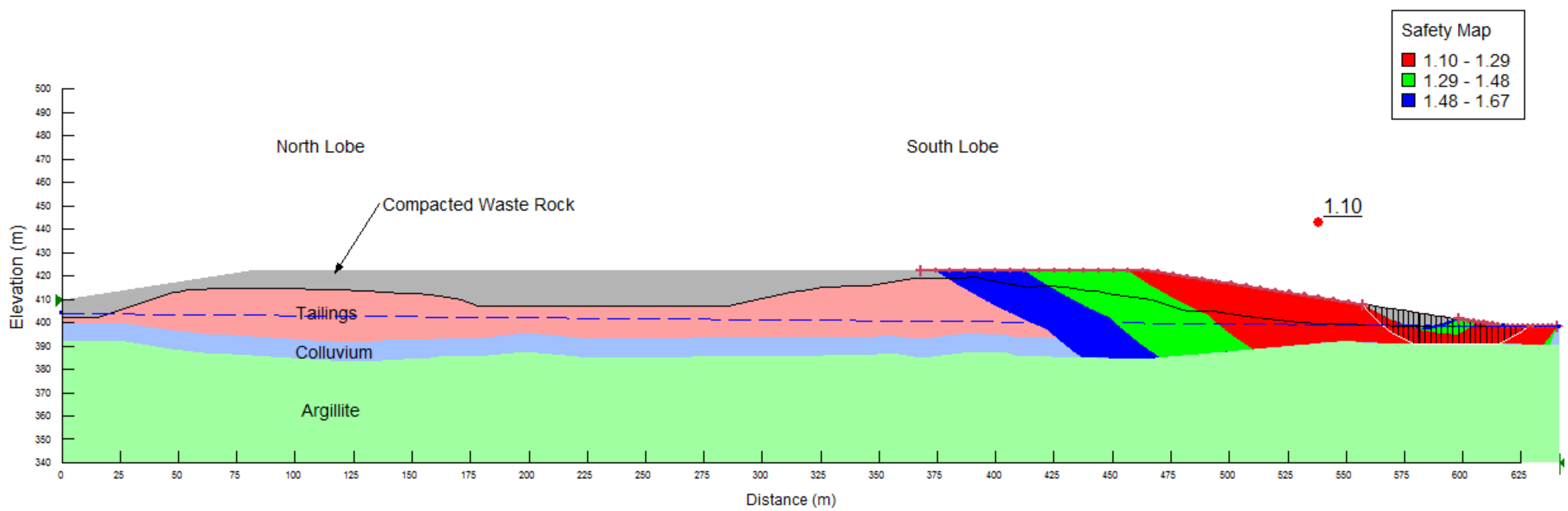
Figure D.16

Wolverine Tailings Pile - Section G-G'

Option D - Pseudo-Static Seismic Conditions

Name: Colluvium S   Piezometric Line: 1   Unit Weight: 19.1 kN/m³   Cohesion': 0 kPa   Phi': 14.2 °
Name: Tailings S   Piezometric Line: 1   Unit Weight: 18.1 kN/m³   Cohesion': 0 kPa   Phi': 21.3 °
Name: Argillite   Piezometric Line: 1
Name: Waste Rock S (Compacted)   Piezometric Line: 1   Unit Weight: 20 kN/m³   Cohesion': 0 kPa   Phi': 28.4 °

Horz Seismic Coef.: 0.06



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Section G-G'
Remediation Option D
Pseudo-Static Seismic Conditions

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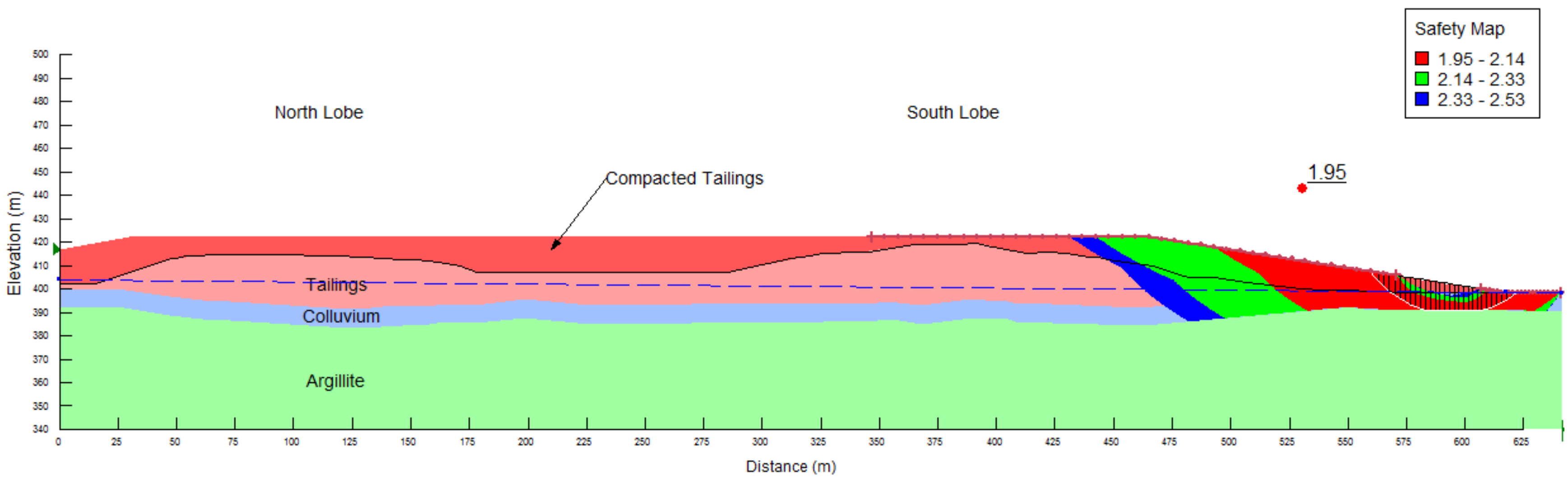
Figure D.17

Wolverine Tailings Pile - Section G-G'

Option E - Steady State Seepage Conditions

Name: Colluvium Piezometric Line: 1 Unit Weight: 19.1 kN/m³ Cohesion': 0 kPa Phi': 17.5 °
Name: Tailings Piezometric Line: 1 Unit Weight: 18.1 kN/m³ Cohesion': 0 kPa Phi': 26 °
Name: Argillite Piezometric Line: 1
Name: Tailings (Compacted) Piezometric Line: 1 Unit Weight: 19 kN/m³ Cohesion': 0 kPa Phi': 32 °



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Section G-G'
Remediation Option E
Steady State Seismic Conditions

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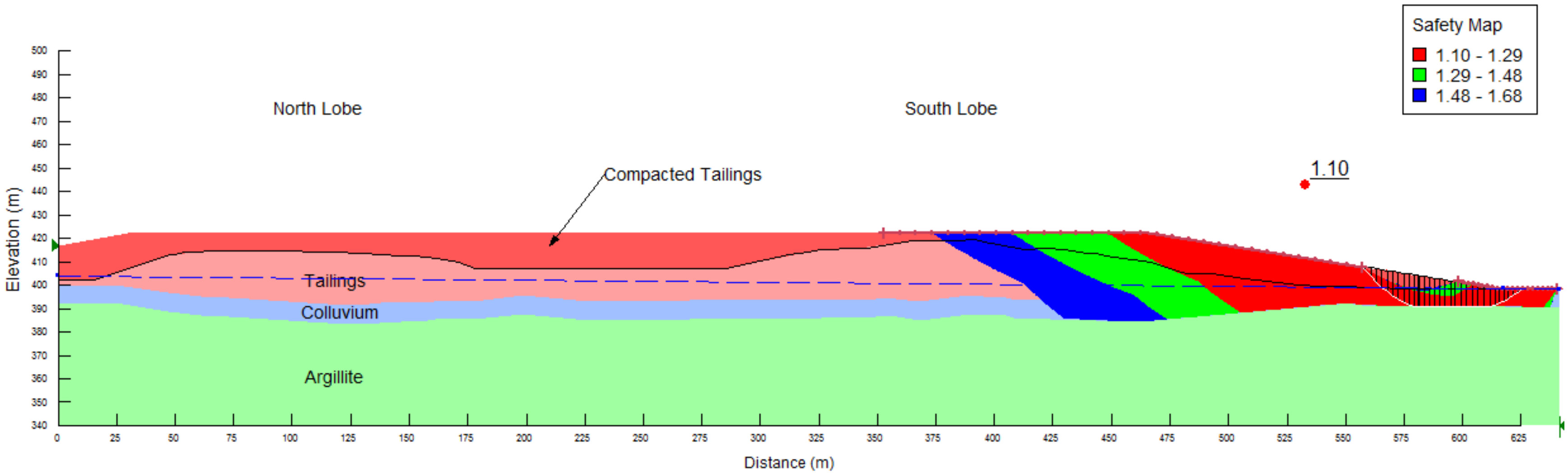
Figure D.18

Wolverine Tailings Pile - Section G-G'

Option E - Pseudo-Static Seismic Conditions

Name: Colluvium S   Piezometric Line: 1   Unit Weight: 19.1 kN/m³   Cohesion': 0 kPa   Phi': 14.2 °
Name: Tailings S   Piezometric Line: 1   Unit Weight: 18.1 kN/m³   Cohesion': 0 kPa   Phi': 21.3 °
Name: Argillite   Piezometric Line: 1
Name: Tailings S (Compacted)   Piezometric Line: 1   Unit Weight: 19 kN/m³   Cohesion': 0 kPa   Phi': 26.6 °

Horz Seismic Coef.: 0.06



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Section G-G'
Remediation Option E
Pseudo Static Seismic Conditions

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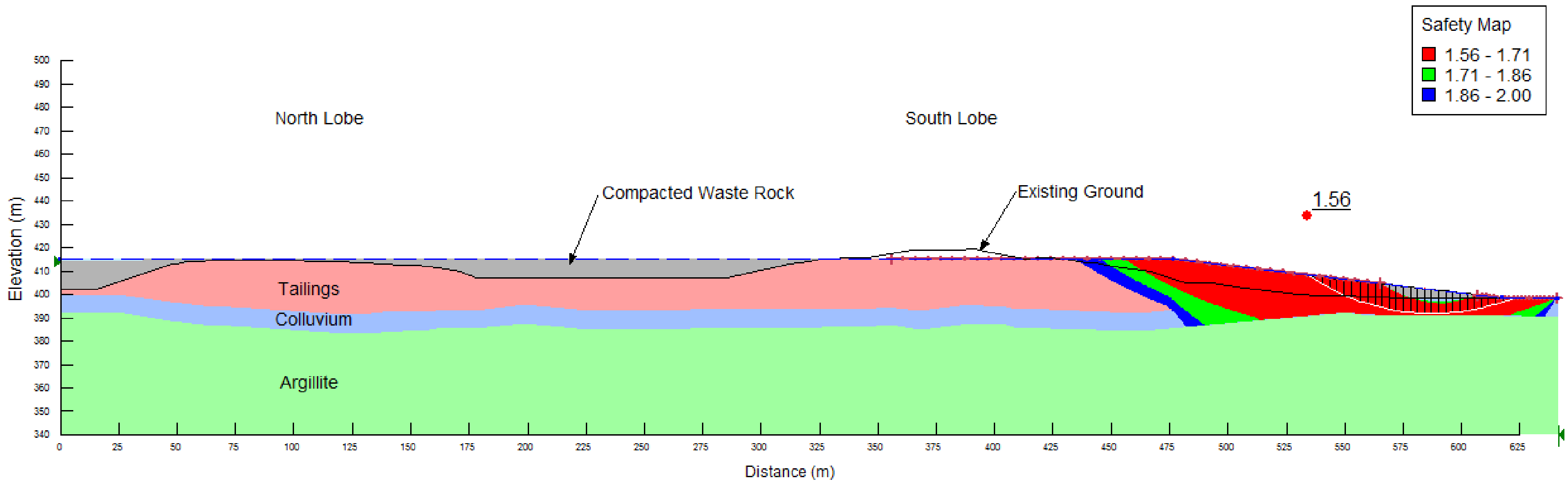
Figure D.19

Wolverine Tailings Pile - Section G-G'

Option D2 - Steady State Seepage Conditions

Name: Colluvium Piezometric Line: 1 Unit Weight: 19.1 kN/m³ Cohesion': 0 kPa Phi': 17.5 °
Name: Tailings Piezometric Line: 1 Unit Weight: 18.1 kN/m³ Cohesion': 0 kPa Phi': 26 °
Name: Argillite Piezometric Line: 1
Name: Waste Rock (Compacted) Piezometric Line: 1 Unit Weight: 20 kN/m³ Cohesion': 0 kPa Phi': 34 °



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Section G-G'
Remediation Option D2
Steady State Seepage Conditions

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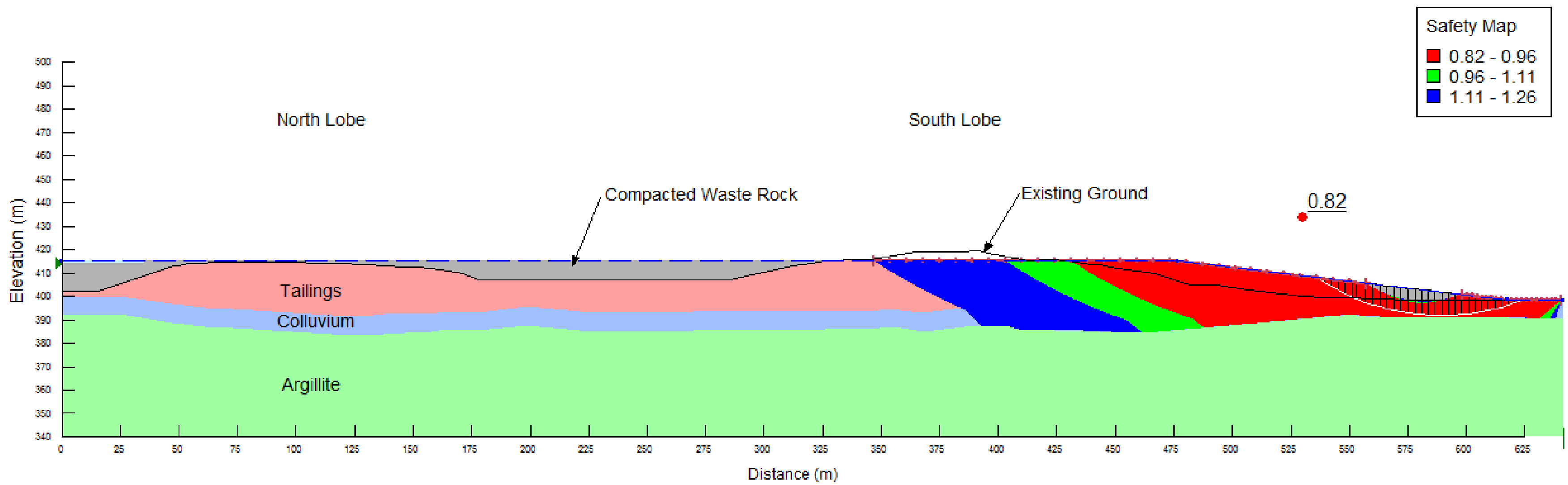
Figure D.20

Wolverine Tailings Pile - Section G-G'

Option D2 - Pseudo-Static Seismic Conditions

Name: Colluvium S Piezometric Line: 1 Unit Weight: 19.1 kN/m³ Cohesion': 0 kPa Phi': 14.2 °
Name: Tailings S Piezometric Line: 1 Unit Weight: 18.1 kN/m³ Cohesion': 0 kPa Phi': 21.3 °
Name: Argillite Piezometric Line: 1
Name: Waste Rock S (Compacted) Piezometric Line: 1 Unit Weight: 20 kN/m³ Cohesion': 0 kPa Phi': 28.4 °

Horz Seismic Coef.: 0.06



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Remediation Option D2
Pseudo-Static Seismic Conditions

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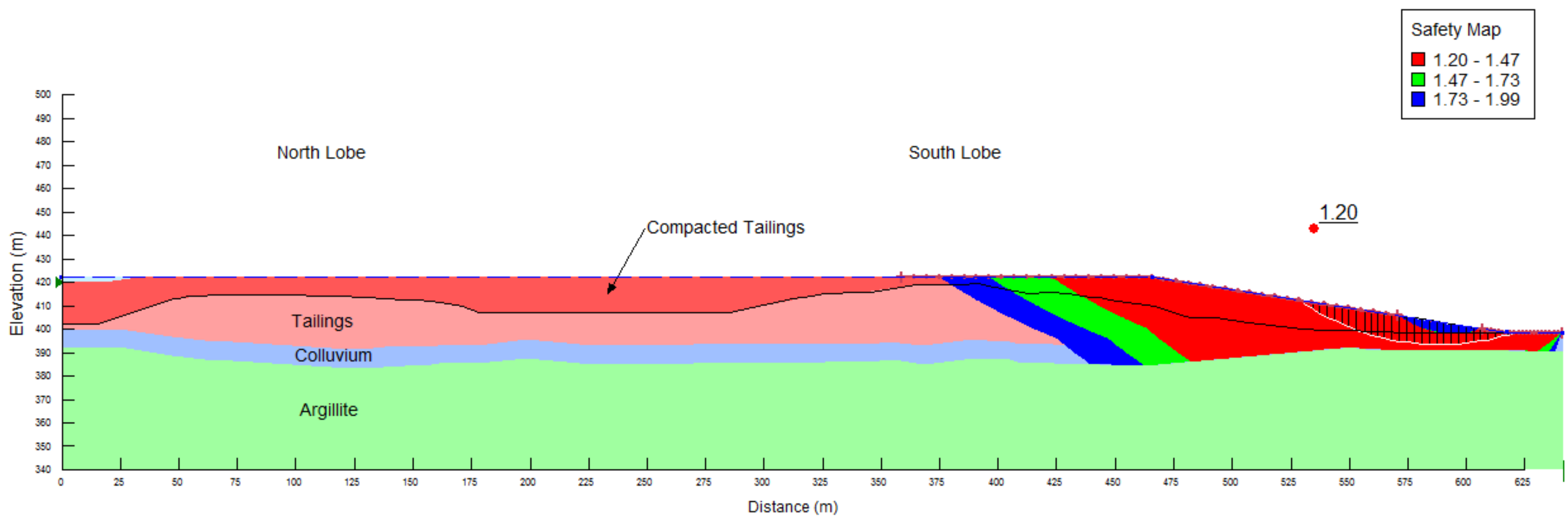
Figure D.21

Wolverine Tailings Pile - Section G-G'

Option E2 - Steady State Seepage Conditions

Name: Colluvium Piezometric Line: 1 Unit Weight: 19.1 kN/m³ Cohesion': 0 kPa Phi': 17.5 °
Name: Tailings Piezometric Line: 1 Unit Weight: 18.1 kN/m³ Cohesion': 0 kPa Phi': 26 °
Name: Argillite Piezometric Line: 1
Name: Tailings (Compacted) Piezometric Line: 1 Unit Weight: 19 kN/m³ Cohesion': 0 kPa Phi': 32 °



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Section G-G'
Remediation Option E2
Steady State Seepage Conditions

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Figure D.22

Wolverine Tailings Pile - Section G-G'

Option E2 - Pseudo-Static Seismic Conditions

Name: Colluvium S Piezometric Line: 1 Unit Weight: 19.1 kN/m³ Cohesion': 0 kPa Phi': 14.2 °
Name: Tailings S Piezometric Line: 1 Unit Weight: 18.1 kN/m³ Cohesion': 0 kPa Phi': 21.3 °
Name: Argillite Piezometric Line: 1
Name: Tailings S (Compacted) Piezometric Line: 1 Unit Weight: 19 kN/m³ Cohesion': 0 kPa Phi': 26.6 °

Horz Seismic Coef.: 0.06

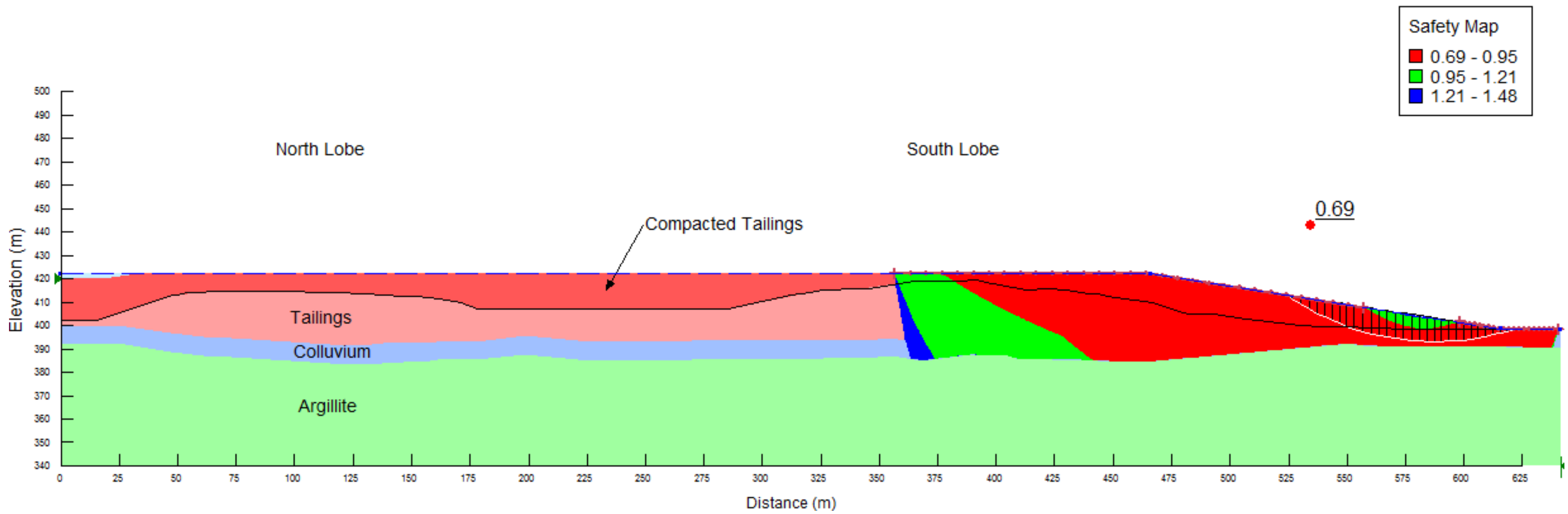

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Figure D.23