Geotechnical Summary

Clinton Creek Remediation Project
Clinton Creek, Yukon
Project # VE52705D

Prepared for:

Government of Yukon Energy, Mines and Resource Assessment and Abandoned Mines

2C – 4114 4th Avenue, Whitehorse, Yukon, Y1A 4N7

26 August 2019

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Prepared for:

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2C – 4114 4th Avenue, Whitehorse, Yukon, Y1A 4N7

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26 August 2019

Executive Summary

This report summarizes the geotechnical studies that have been undertaken by Wood Environment & Infrastructure Solutions (Wood) in support of the Clinton Creek Remediation Project 10% Design Phase. This report serves as a collection of various engineering memorandums which have been completed as a part of the 10% design of the Clinton Creek remedial options. In addition, presentations made to Project Partners and the IPRP are cited This material is available in Project Partner archives and, while relied on in overall studies, is not directly included in this report.

Section 2 of this report provides an overview of our understanding of the basic objectives of the six Closure Concepts. This has been formulated based on the original scope, guidance from the Project Partners, and comments from the Independent Project Review Panel (IPRP).

Completed memorandums are presented in the Appendices of this report, as follows:

Appendix	Subjects
A	Geotechnical Design Basis Memorandum (DBM) - Geotechnical Criteria & Issues
B	Summary of Available Information from Provided InSAR Data
C	Seepage and Internal Erosion Considerations
D	Geothermal Analysis for Spillway and Dumps
E	Assessment of Liquefaction Triggering and Post Liquefaction Strength
F	Stability Analysis: • Clinton Creek Waste Dump and Porcupine Pit Geotechnical Assessment • Wolverine Creek Tailings Assessment
G	Storage and Excavation Volumes
H	WC2 Design Basis

It should be noted that this report is an update of Wood's "Status of Geotechnical Studies" dated 05 April 2019 and presents clarification of the work done to April 2019, as well as Appendix H dealing with the WC2 Option.

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1.0 Introduction

1.1 Scope

This report provides a summary and synthesis of all geotechnical work undertaken by Wood, is support of the in support of the Clinton Creek Remediation Project 10% Design Phase. This report presents engineering memorandums undertaken by several members of the Wood Project Team which have been completed as a part of the 10% design of the Clinton Creek remedial options. In addition, presentations made to Project Partners and the IPRP are cited. This material is available in Project Partner archives and, while relied on in overall studies, is not directly included in this report.

1.2 Background

There are several relevant Amec Foster Wheeler and Wood reports and workshop presentations, as well as previous reports by others, which are contained in Project Directories. Of particular interest, and as relied on for this report, the reader may wish to consider the following:

- Wood (September 2019) Geotechnical and Geological Site Characterization & Model.
- Amec Foster Wheeler (2018) Geotechnical Design Gaps
- PowerPoint Presentations at Workshops with Project Partners, and the IPRP

1.3 Report Contents

This report serves as a collection of various engineering efforts that have been undertaken as a part of the 10% design of the Clinton Creek closure concepts as a part of the Clinton Creek Remediation Project.

Section 2 of this report, “Project Partners 10% Design Options”, provides an overview of the Closure Concepts selected by the Project Partners which gave the underlying direction to this assignment.

This report has been assembled by preparing a series of memorandums on individual subjects, which are located in the report Appendices. These subjects and their corresponding Appendices presented in Table 1.

Table 1: List of Subjects

Appendix	Subjects
A	Geotechnical Design Basis Memorandum (DBM) - Geotechnical Criteria & Issues
B	Summary of Available Information from Provided InSAR Data
C	Seepage and Internal Erosion Considerations
D	Geothermal Analysis for Spillway and Dumps
E	Assessment of Liquefaction Triggering and Post Liquefaction Strength
F	Stability Analysis: • Clinton Creek Waste Dump and Porcupine Pit Geotechnical Assessment • Wolverine Creek Tailings Assessment
G	Storage and Excavation Volumes
H	WC2 Design Basis

2.0 Project Partner 10% Design Options

The intent of this section is to summarize Wood's understanding of the 3x3 Closure Concepts selected by the Project Partners. The following descriptions in ***italics*** are taken from the original scope of work from Yukon 10 17 2017 Amec Scope of Work (003), and as subsequently modified based on direction received from the Project Partners.

At this 10% design stage, our understanding may not appropriately address the Project Partners' evolving requirements over-arching compliance with the five guiding closure principles. As additional data is obtained by the Project Partners in the next design phase the specifics for each option can be enhanced as required.

2.1 Clinton Creek Side Closure Concepts

CC1: Water Passage and Catastrophic Failure Mitigation (LCCA Option D3, I2) – Conduct sufficient work on the waste rock pile to mitigate a catastrophic failure of the pile and construct a water conveyance channel to provide water passage from Hudgeon Lake to Clinton Creek.

This option has been interpreted by Wood as assumed to apply to maintaining the current lake level in Hudgeon Lake and the VERTICAL alignment of the spillway is more or less fixed, given the embayment that provides connection between the lake and the head of the spillway.

Wood has identified the following issues for consideration:

- The HORIZONTAL alignment has to be shifted as much as is practical to the south to allow the eroded north valley wall of Clinton Creek to be repaired.
- Internal erosion/piping has been identified as a failure mechanism for the current vertical alignment (i.e., for the existing drop structures).
- Other design issues include thaw settlement of permafrost under the spillway, seismic induced settlement, and liquefaction of spillway channel side slopes.
- Some drawdown of the lake may be required on a temporary basis to safely implement remedial works.
- As Hudgeon Lake is maintained in perpetuity, ongoing "active care" is required. This option is not intended to be a "walk-away" solution.

CC2: Water Passage, Catastrophic Failure Mitigation and Lowering Lake (LCCA Option E3) – Conduct sufficient work on the waste rock pile to mitigate a catastrophic failure, construct a water conveyance channel to provide water passage from Hudgeon Lake to Clinton Creek, and lower Hudgeon Lake as part of that concept.

This option has been interpreted by Wood as follows:

- Provides the ability to define a channel out of a reduced lake level towards an ultimate "regime channel", which may ultimately assist in arguing for a more acceptable walk-away solution.
- The avoidance of the issues with the steeper vertical alignment of Option CC1 are considered favorable.

CC3: Water Passage with Reduction of the Lake Level, Eliminating the Dam, and Mitigating Catastrophic Failure (LCCA Option F) – Conduct sufficient work on the waste rock pile to prevent it from acting as a Dam (i.e. as defined by the Canadian Dam Association) on Clinton Creek and to mitigate a catastrophic failure of the waste rock pile. Construct a water conveyance channel to provide water passage through the site.

This option has been interpreted by Wood as well defined at a 10% design in terms of requirements for final configurations.

2.2 Wolverine Creek Side Closure Concepts:

WC1: Sediment Control Only (Not in the LCCA) – Construct a sediment control structure downstream of the rock-lined channel in Wolverine Creek – no work on the tailings pile or the channel is required.

This option has been interpreted by Wood as follows:

- Natural processes left to erode and allow tailings to reach an equilibrium condition in the closure landscape.
- A sediment control structure to be emplaced downstream of current rock lined structure. However due to space limitations this control structure has been located to below the confluence of Wolverine and Clinton Creek.
- Long term maintenance will be required, and if a major seismic event occurs, there is a high probability of tailings release into Clinton Creek, and possibly the Fortymile River.

WC2: Water Passage and Stability Improvement (LCCA Option B, C, D, D2 – note that Option B does not have a remediation measure for the tailings) – Conduct sufficient work at the base of the tailings pile to minimize the tailings movement and provide a semi-stable surface to construct a water conveyance channel.

The main premise of the WC2 Option is to restore the tailings insitu within the confines of the Wolverine Creek valley.

- As the tailings are considered to be liquefiable the tailings slopes must remain stable in the long term so that a major release either due to oversteepening by toe erosion or seismic action is not triggered.
- This requirement applies to the main east facing slope down the valley wall, as well as the perimeter slopes.
- The main valley wall slopes can only be stabilized by a buttress within the Wolverine Creek valley which in turn requires a Buttress Fill Dam down valley.
- Perimeter slopes can either be flattened or buttressed by berms which also must be sub-cut below ice rich nearer surface permafrost.

The water conveyance structure must now be viewed as a dam spillway.

- Given that the Buttress Fill Dam could be up to 50 m high on a 5H:1V slope this is a Significant Level structure if designed to full CDA hydrotechnical requirements.
- The current design of the down-take contemplates a reduced flood event with a risk-based maintenance approach embracing the “semi-stable” approach.

A workable scenario for the WC2 Option is provided for the 10% design and cost estimate.

- One significant issue is that the current overall configuration requires more fill material than is required to be excavated to meet geotechnical criteria. Thus, more tailings could be excavated to meet demand.
- Alternatively, and depending on the Clinton Creek waste disposal plan, waste dump spoil could be used instead.
- There are optimizations that can be considered in the next design phase, should this option be selected by the Project Partners.
- Note that the Buttress Fill Dam requires select rockfill for long term sustainability, and that Waste Dump fills or Tailings not considered acceptable.

WC3: Isolate the Asbestos (LCCA Option E, E2) – Stabilize tailings pile to allow a cover to be placed or relocate the tailings pile.

The WC3 option basically removes all tailings out of the Wolverine Creek valley. The main disposal considered is Porcupine Pit.

- It is considered desirable to dispose of tailings into Porcupine Pit before excavated waste dump fills.
- There is some capacity for storage of some tailings in the geothermally disturbed Plant Site area.

2.3 P Summary

The CC options and WC1 are clear-cut in respect to overall objectives. As presented, WC2 and WC3 with tailings either completely sequestered in the Wolverine Creek valley, or completely sequestered in Porcupine Pit, respectively, are also clear-cut. However, there may be possible economic optimizations by considering a combination of WC2 and WC3. Wood also notes that for the WC2 option as presented there are internal optimizations possible.

3.0 Synthesis

3.1 Appendix A – Design Basis Memorandum

Appendix A provides a geotechnical update to the previous Amec Foster Wheeler DBM (Amec Foster Wheeler, 2018).

The main topics covered in the memorandum are:

- Discussion on landslide dams, and performance summaries by R.L. Schuster.
- Discussion on Active vs Passive Care, and what constitutes current practice for site closure.
- The basis for adopting hazard risk levels for seismic versus hydrotechnical.
- Models for determining mobilized strength
- Factor of safety criteria adopted.

3.2 Appendix B – InSAR

Appendix B provides a summary of Woods understanding of the provided InSAR data, including determination of both vertical and horizontal movement rate in waste dumps and the tailings deposits (where 2-D InSAR data was available). These movements have not been rationalized against the measurements taken on surface hubs. Ultimately it would be expected to be able to combine the InSAR and hub data sets with slope indicator monitoring results.

Wood has noted a reasonable correspondence between maximum settlement rates and foundation segments that may contain ice-rich permafrost in the InSAR measurements.

3.3 Appendix C – Seepage and Internal Erosion

Appendix C summarizes seepage and internal erosion considerations. The following topics are considered in the memorandum.

- Estimates of the hydraulic conductivity of waste dump materials is estimated based on particle size data. Preliminary attempts to back-analyze winter seepage out of Hudgeon Lake are still underway and will utilize this winter's data sets. The range in hydraulic conductivity is from a low of 1×10^{-6} m/s for high fines to as high as 5×10^{-5} m/s for low fines.

- The overall safe gradients through the waste dumps forming the landslide dumps is estimated to be ≤1/20 as compared to safe levels of 1/33 overall. However, seepage gradients of 1/10 and as high as 1/5 are predicted in the immediate vicinity of the gabion drop structures. This will lead to internal erosion and Wood observed one case in August 2018, on the left bank under the failed DS4 chute.
- Mitigative design measures to control internal erosion on the overall spillway where gradients are steeper than 1/20 will be required. A regime channel with overall slope of 1/33 (3%) will be intrinsically safe (ie closure concept CC2).
- Parametric analyses have been undertaken of allowable drawdown rates. The stability analysis reported in Appendix F have assumed that the water table within the dumps should not be higher than the dump toe, considering Hudgeon Lake as an example. These analyses indicate that drawdown rate may be a constraint in construction scheduling for Options CC2 and CC3

3.4 Appendix D – Geothermal Analysis

Appendix D presents two extensions on the initial geothermal analysis presented in Amec Foster Wheeler (2018). These are:

- A 2-D parametric assessment of a buried high ice content layer considering both convective heat input from the lake and conductive heat from an assumed outlet channel.
- A 1-D parametric assessment of a frozen layer buried under a deep dump where the thaw is generated by a constant heat flux.

The 2-D convective analysis essentially models a "slice" of unit width down the centerline of a spill way channel, under 15 m of dump/colluvium sloped at 6.2%. This layer is assumed to be 1 5m deep by 150 m long and was assumed to be 95 m from the lake bottom. The thawed layer was assigned a hydraulic conductivity of 1x10-5 m/s.

The results, presented in Tables 4 and 5 of Appendix D, show both thaw depth and settlement, accounting for thaw strains of 65% and 30% considered appropriate for analysis with w/c(ice) contents of 150% and 50%, respectively. There was more thaw in the icy soil closest to the lake. Settlement rates of 200 mm/yr. and 65 mm/yr. were predicted after a 20-year run for the 150% w/c (ice) case. The settlement rate dropped to 120 mm/yr. to 45 mm/yr. for the 50% w/c(ice) case. Table 6 presents two comparison cases where heat convection is cut off, and thaw is generated by either the typical measured water temperatures in the spillway, or a surface temperature boundary condition (as used in previous analysis). The spillway water temperatures predict a steady state settlement rate of 68 mm/yr. and 47 mm/yr. for the 150% and 50% w/c(ice) cases, and the surface air boundary condition 65% and 39%, respectively.

The results of the 1-D analysis are reported in Table 7. The predicted settlement rate for deeply buried soil peaks at about 12 mm/yr. for a w/c(ice) of around 50%.

These analyses suggest the following:

- High ice content permafrost can be still expected under the spillway and, if present today, will take decades to completely thaw out, depending on depth and w/c(ice) content.
- Thaw settlements of 50-60 mm/yr. can be expected where temperatures have not reached a thermal equilibrium (or are due to other causes).
- Settlement rates can be in the order of 10 mm/yr. if the sole cause is heat conduction from the surface.

3.5 Appendix E – Liquefaction

Appendix E presents results of studies related to the potential for liquefaction of the landslide waste dumps in the Clinton Creek Waste Dump and the Wolverine Tailings Dump. It is concluded that the waste dumps are vulnerable to both static and seismic (dynamic) triggering. Static liquefaction could be triggered by rapid drawdown, in particular in relation to Hudgeon Lake, but also due to toe unloading, especially if pore pressures within the dump are slow in responding. This may result in scheduling issues if a Closure Concept involving lake drawdown is adopted.

The tailings are also susceptible to liquefaction, especially the upper slope. The lower part of the lobes may have been partly compacted by site operations or may be frozen. However, depending on the absence of a water table in the tailings in the upper slope, the tailings may not liquefy in a seismic event.

Appendix E also provides the post liquefaction undrained liquefied strength ratios being used in design.

3.6 Appendix F – Stability Analysis

Appendix F summarizes stability analysis undertaken to date. There are two memorandums included in Appendix F, the first memorandum examining the stability analyses undertaken for the Clinton Creek Waste Dump, and the second memorandum examining the stability analyses undertaken for the Wolverine Tailings Dump.

3.6.1 Clinton Creek Waste Dump Closure Concepts

For analysis purposes, the three Project Partner closure concepts have been related to lake levels used in the stability calculations as follows:

- CC1 corresponds to a lake level at El. 412 masl.
- CC2 corresponds to a lake level at El. 402 masl.
- CC3 corresponds to a no lake option.

This is not to say that at this point CC2 (aka the regime channel option) is fixed with a lake at El. 402 masl, it is what has been selected as potentially representative. The three alignments are summarized in the figures in Appendix F.

A significant feature in the preliminary scope of the three options is the plan location of the Clinton Creek channel. These locations have to, or will have to, recognize the following issues:

1. For option CC1, the present embayment leading to the weir (gabion baskets) at the head of the spillway is considered to be left in place. This limits the ability to shift the spillway alignment much away from the north slope, and south of the ice rich colluvium currently considered to underly the spillway. For option CC1, a permanent spillway will require pre-thawing of whatever permafrost is under the spillway, and other design measures.
2. For options CC2 and CC3, some offset from the north slope will be required, so as to rebuild, as much as is practical, the pre-development configuration of the north slope, in order to stabilize the slope.
3. Options CC1, CC2 and CC3 will all required consideration of the ice rich permafrost characteristic of the colluvium encountered in BH18-03.
4. The location of the regime channel in CC2, and the channel thalweg for CC3 on original ground, is also impacted by north slope instability, particularly in ice rich permafrost, as this colluvium is exposed or becomes near surface and thaws more rapidly.

5. All options require measures to cut back the dump slopes facing Hudgeon Lake. This is required to maintain long term static stability and post seismic liquefaction stability.
6. All current design slopes assume that the piezometric elevations (ie the water table) in the dumps lowers at the same rate the lake is lowered. If this is not accomplished, there could be significant safety issues with static liquefaction induced by rapid draw-down. This may have impacts on schedules, especially for option CC3. It may also be necessary to temporarily draw down Hudgeon Lake for CC2, to be able to safely construct whatever channel configuration, underdrainage, and internal erosion control is required to manage seepage.

Analyses, as reported in Appendix F, have indicated that overall slope angles required to manage seismic liquefaction control the overall slope angles, as compared to static analysis, per the DBM guidelines (Appendix A). Based on the analyses, overall cut slope angles of 6.0H:1V have been selected for dump slopes facing Hudgeon Lake and 6.5H:1V for cut slopes facing the Clinton Creek channel for all three options.

As can be seen in the analyses, some configurations of 6.5H:1V cuts cannot deliver the required FoS ≥1.0 for a liquefied strength ratio of 0.10. It is considered that the excavation volumes predicted per Appendix G are indicative of what is needed for each option (keeping in mind the residual uncertainties due to poor original topographic control). Measures that could be taken if slopes flatter than 6.5H:1V are required are:

1. Additional site investigation, given the current data base.
2. Better control of the water table with ditching, toe berms, or sub-cuts, especially for the CC3 option.
3. Flattening the slope further.

A preliminary assessment of the required shell for the Porcupine Pit storage is also provided in this memorandum.

3.6.2 Wolverine Tailings Dump Closure Concepts

Section 2.0 provides Wood's understanding of the Project Partners' options, which are adopted for the work products for Wolverine Creek. Additional assessment of the WC3 Option is presented in Appendix H.

3.6.2.1 WC1

The analyses presented for option WC1 summarize our understanding of the present static stability of three sections: TS1 (the south lobe), TS2 (the north lobe) and TS3 (a slope oriented to the north-east and skewed to Wolverine Creek). For realistic drained parameters, the current FoS for the south lobe and north lobe are 1.04 and 1.14, lower than a required 1.3. The current FoS for the TS3 section, using the same parameters as TS1 and TS2 is 1.36. These FoS may be somewhat refined with slope indicator results, but are subjectively supported by InSAR data, see Appendix B.

All sections are highly vulnerable to liquefaction failure under seismic loading, if a groundwater table develops along the base of the tailings, and the tailings / colluvial contact. Therefore, adopting this option is essentially equivalent to accepting a longer-term requirement to undertake considerable remediation. It is doubtful if a sediment control structure is viable under this level of potential loading into the creek.

3.6.2.2 WC2

Option WC2 requires a major buttress to obtain the necessary FoS. The buttress reaches a higher elevation for the liquefaction case, up to elevation 435 masl where the buttress intersects the east slope (left slope) of the valley (or the spill elevation). This results in a buttress retention structure, on the down-creek side of the buttress, that is in the order of 50-55m high. In addition, unless a buried rock drain or low level outlet is contemplated, the new Wolverine Lake would rise from 409 masl to 435 masl. If the risks and remedial action associated with liquefaction failure are accepted, the buttress or spill elevation required to control static stability is predicted at elevation 428 masl.

Details of the WC2 Option, based on the necessity of a buttress, are presented in Appendix H, and discussed in Section 3.8.

3.6.2.3 WC3 and LCCA(E)

One option for WC3 is complete removal of the tailings. The main geotechnical concern will be restoration of the slopes, potentially with several meters of waste fill ballast. However, all geotechnical design challenges are resolved, apart from the safe storage of re-located tailings, and other colluvial debris.

The WC3 LCCA E option was considered; this option flattens the tailings to a 3.75H:1V slope up from the centerline of the creek channel with a cut/fill operation.

The FoS for this option is estimated to be 1.5 under static loading; however, the FoS if liquefaction is accounted for is less than 1.0. This option was then considered not to be viable, and was further considered as a part of the WC2 option presented in Appendix H.

3.7 Appendix G – Volumes

Appendix G summarizes the estimated volume of the following:

- The existing volume of materials in the Clinton Creek Waste Dump, Wolverine Tailings Dump and the Porcupine Creek Waste Dump.
- The required volume of material that would be excavated for the closure concepts CC1, CC2, CC3 and WC3 (option of removing all of the tailings).
- The available storage volume in the Porcupine Pit, Snowshoe Pit and in the old mill site.

Based on the memorandum, the estimated volume of waste material present on site is 25 M m^3 and the estimated volume of tailings is approximately 7.6 M m^3.

The estimated volumes of material that would need to be removed from the Clinton Creek Waste Dump ranged from approximately 5 M m^3 for CC1 to 13.4 M m^3 for CC3. The estimated volume of tailings in the Wolverine Tailings Dump was 7.7 M m^3, which would all be removed as one potential option in WC3. Based on the maximum amount of material that would need to be moved, if Closure Concepts CC3 and WC3 were selected, an estimated volume of 21.1 M m^3 of waste and tailings would need to be moved.

The total amount of storage available in Porcupine Pit was estimated to be 23.9 M m^3, assuming that a shell at a 3H:1V slope was constructed above the current pit rim to elevation 570 m. Even if options CC3 and WC3 were both selected, and a 10% bulking factor were applied to the excavated volumes, there would be adequate storage in Porcupine Pit.

3.8 Appendix H – WC2 Option Details

Section 3.6.2.2 and Appendix F-2 indicate that for WC2 Option of stabilization of tailings insitu in the Wolverine Creek valley requires a buttress fill to stabilize the east facing slopes of the tailings. This option has several elements which are considered further in Appendix H. The main components are:

1. In order to contain the buttress fills to the south, down the Wolverine Creek valley, a Buttress Fill Dam is required across Wolverine Creek. The dam is configured to have a 440 masl crest and an overall slope of 5.2H:1V. The dam length is constrained by the necessity not to block a gulley that drains into Wolverine Creek from the east.

2. The stability of the remaining perimeter tailings slopes to the west, south and north must also be left in a stable condition. Many of these slopes exhibit on ongoing movements and tension cracks.
3. There must be a long-term stable outlet for Wolverine Creek along the buttress fills and a spillway down the Buttress Fill Dam; design of the spillway has been completed as a part of the hydrotechnical design and is presented in the final Design Report.

The Buttress Fill Dam being contained within the Wolverine Creek valley is supported by existing valley slopes which are likely blanketed with ice rich colluvium, based on available boreholes. In order to manage thawing of these foundations, the design requires sub-cutting of this ice rich material and replacement with compacted permeable backfill. Any existing tailings must be removed to an offset of 50% of the length of the downstream slope.

Dam fills require select free-draining rockfill from a borrow source. Tailings or random waste dump excavated spoil is not acceptable given the longevity requirement for this structure. Free draining rockfill is also considered to be required to manage seepage from the raised creek channel and the spillway down the dam face, which should not be routed down the left abutment.

An internal chimney drain and foundation blanket drain is configured to manage internal erosion from the foundations. Appendix H also provides the quantity estimates for the various dam components.

4.0 Closure

The geotechnical work undertaken to fulfil Wood’s scope for a 10% design has been focused on several components which are:

- In the absence of little factual data on mine plans, overburden operations, the sequencing of landslide development with time in both the Clinton and Wolverine Creek valleys, judgement and a general knowledge of mining operations was required to develop a site characterization. The results of the field investigation campaigns assisted in developing a history of the operations., but this has also been constrained by a lack of reliable pre-mining topography and no information on pit slope design data.
- Borehole information was spread out over extensive areas, which makes for considerable uncertainty in, for example, the distribution of ice rich colluvium in all valley wall segments. The necessary extrapolations were facilitated by the formulation of geological models.
- Making judgment calls as to what in fact is suitable for a “10% design” constrained by geotechnical factors was a necessity but is open to change as more data becomes available.
- Based on discussions with the Project Partners and the IPRP, a weighting towards identifying fatal flaws and ways to accommodate high risk components became a significant study component.
- The field investigation and drilling programs supported to 10% design in terms of general understandings of foundation geology and dump characteristic as a reasonable basis to address the Project Partners 3 x 3 Options. However, depending on the go-forward option selected by the Project Partners, additional option specific data will be required for design optimization.

This report is subject to the limitations attached in Appendix I.

Recommendations and assessments presented herein are preliminary in nature and based on limited subsurface information. The recommendations and assessments presented in this report should not be used for design.

This report has been prepared for the exclusive use of the Government of Yukon Energy, Mines and Resource Assessment and Abandoned Mines, for the specific application described herein. Any use which a third party makes of this report, or any reliance on or decisions made based on it are the responsibility of such third parties. Wood Environment & Infrastructure Solutions accepts no responsibility for damages suffered by any third party as a result of decisions made or actions based on this report.

Sincerely,

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5.0 References

Amec Foster Wheeler (2018) *Geotechnical Design Gaps*. VE52695. 29 March 2018.

Wood (2019) *Geotechnical & Geological Site Characterization and Model*. DRAFT Report issued 29 March 2018.

Appendix A
Clinton Creek
Design Basis Memorandum

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Memo

To:	File		
From:	Ed McRoberts, PhD, PEng	**Reviewed By:**	Brian Ross, PEng (AB)
cc:	Karen Hincks, MSc, PGeo	**Wood File No.:**	VE52705D.100.2
Date:	23 August 2019		
Re:	**Clinton Creek Site: Design Basis Memorandum (DBM) – Geotechnical Criteria & Issues**		

1.0 Scope of Geotechnical DBM

1.1 Project Parties "Scope of Work"10172017 AMEC Scope of Work (003)

The Project Parties have stated that "the intent of the 10% design development was to advance the design of the closure concepts to a level that would allow the Project Parties to select a single remedial option to advance for the site." The same closure concept would not necessarily apply to the Clinton Creek, Wolverine Creek, and Porcupine Creek valleys. From an engineering perspective, remedial action explicitly embraces a situation that unsatisfactory or unsafe performances are ongoing, and works must be implemented to correct such situations. Clearly this applies to the Clinton Creek site in general. On the other hand, specifically what is meant by closure is less well defined in current usage with respect to tailings dams and other structures. One currently held perspective is that closure means that the land is basically returned to a safe condition and will maintain that condition in perpetuity – this is often referred to as a "walk away condition". This would include no to little monitoring, or inspection, or future remedial interventions. On the other hand, the term "active closure" is used to describe a post-remedial phase, but with anticipated monitoring, inspection, and future remedial interventions for extended period of times, out to some less definable future.

Of the 3×3 options selected by the Project Parties, at least 2×2 of the options, such as maintaining Hudgeon Lake with a closure spillway, leads to explicitly committing to an active closure framework. Only one option for each area, a no lake option for any creek valley, leads to the ability to confidently argue for a completely passive landscape, or at least one no different than the pre-development hazards that existed. It is not within Wood's scope to make these determinations in option selection. For the two design scenarios requiring Hudgeon Lake, active care is therefore required, so Wood considers the over-arching objective is a reasonably maintainable facility where reasonable on-going maintenance can be implemented.

1.2 Scope of DBM

This document provides geotechnical criteria and related issues for the Clinton Creek Mine site. While the design perspective is dominated by consideration of the remedial requirements to meet the five main overall high-level guidelines as selected by the Project Parties for the Clinton Creek Remediation Project, these principles are qualitative and subjective. Earlier, Amec Foster Wheeler (2017, Section 3.0) listed these high-level general guidelines and sought to undertake a qualitative and subjective interpretation of each of the five main requirements. In addition, Amec Foster Wheeler (2017, Section 7.0 to 9.0) provides qualitative guidelines as to geotechnical, hydrotechnical and environmental factors. This DBM presents a more detailed discussion, where appropriate, for geotechnical factors and seeks objective and quantitative criteria, where possible. Documentation in respect to hydrotechnical and environmental disciplines is provided elsewhere in separate reports prepared by Amec Foster Wheeler and Wood.

2.0 Dam Classification

If the existence of the current landslide dams on Clinton, Wolverine and Porcupine Creeks are carried forward into the closure landscape, then these dams and related components, such as spillways, require consideration in setting geotechnical criteria. The available guidelines in a Canadian context are those found in documentation formulated by the Canadian Dam Association (CDA). While they do not embrace landslide dams, they do provide at least a starting point for discussion. Current Canadian practice for dam design has evolved over time to reflect a general consensus among practitioners. These guidelines had their genesis in consideration of water-retention structures and have evolved to also consider tailings dams; however, they do not consider landslide dams.

CDA *Application of Dam Safety Guidelines to Mining Dams* (CDA, 2014) considers different phases in the life of a dam, from design, construction, operations and through to final long-term closure. Criteria for closure are generally more restrictive than for operations. As the essential purpose of the current scope is to bring the Clinton Creek site to a long-term condition acceptable to Project Parties the closure criteria require consideration. CDA also make a distinction between "Closure – Active Care Phase" and "Closure – Passive Care Phase". The possibility of "Adaptive Management" implies an active phase approach is under consideration as a method for managing uncertainties.

2.1 Definition of Dam vs Landslide Dam

CDA defines a dam as any structure impounding more than 30,000 m^3 of fluid and over 2.5 m in height as being a fluid retention structure. CDA has developed criteria for a structure to no longer be considered a dam, and thus to become a mine waste facility, as follows:

- No upstream ponded water, or liquefiable tailings, is available to the failure mass. (this can be argued by some Regulators as stating that as long as a spillway is required the dam must remain in Active Care);
- Tailings upstream of the dam cannot flow if the dam is removed;
- Tailings cannot migrate through the structure; and
- The probability of the facility ever reverting to a configuration that meets the definition of a dam is nil. (for example, if the waste dump is partially excavated to restore a creek to initial gradients, subject dump failures cannot re-dam the restored valley).

It is generally accepted that for a typical dam or tailings dam the pond must be removed to gain a "Closure – Passive Care Phase" state due to the long-term risks typically associated with water or tailings storage.

CDA guidelines do not recognize the particular circumstances of a "landslide dam" which, from a negative perspective, denotes a non-engineered structure but, from a positive perspective, provides a very long (upstream to downstream) barrier. In the situation of the Clinton Creek Waste Dump, due to the length of the barrier, the structure is much less likely to offer the same degree of risk as a typical dam, especially if the outlet is appropriately designed and constructed. Risk, in a geotechnical sense, is a combination of the likelihood of failure with the consequence of failure. In CDA guidance, the consequences are determined first, and then used to define the likelihood levels for stochastic loadings or events such as seismic or flood, based on a consequence classification system.

Typically, most landslide dams which block the full valley width fail by overtopping, unless the reservoir level is controlled by an alternative outlet valley, or, if the valley infill is sufficiently long down gradient to allow formation of a stable outlet channel. In the case of Clinton Creek, early operations during the mining period deployed outlet channels, but continuing dump failure moved the channel higher and raised the level of Hudgeon Lake. This resulted in construction of a spillway consisting of 4 drop structures (note: details of this component are provided in the Hydrotechnical Design studies being undertaken by Wood). One option under hydrotechnical consideration is a lowering of Hudgeon Lake to accommodate a more sustainable regime outlet without civil engineering works or drop structures. This, arguably, could support a much more realistically manageable lake in a passive care scenario. In the case of Wolverine Creek, the current "dam", or potential modification if additional in channel works are required, would unlikely be moved into passive care. Currently there is limited ponded water in Porcupine Creek.

Of interest, in consideration of risk management scenarios, is a summary by Schuster (2000, 2006) on the safe performance of dams built on pre-existing landslides. Schuster summarized 254 Large Dams (Definition > 10 m height) worldwide which directly interact with landslides (either built on or have been subject to landslide activity during or after construction) of which only four failed due to problems associated with pre-existing landslides. Schuster summarizes his findings as:

1. Seepage through landslide abutments has been the most common source of failures. When carried to the extreme, seepage could possibly lead to piping, however no case was found in which this occurred because of a dam having been constructed on a landslide. (Wood interprets this conclusion as being due to the length, measured from the upstream to the downstream, of a landslide dam);
2. There is no clear indication as to which landslide types perform best or worst from a stability perspective when used as a foundation or abutment, with slides in shales probably performing the poorest;
3. In contrast all landslide types seem to be subject to seepage problems unless appropriate preventative or remedial measures are taken; and
4. While all aspects of the physical characteristic of the individual landslide should be considered carefully, "of particular importance is the permeability of the landslide mass."

While Schuster's summary does not contain specific guidance, his findings are important in providing subjective and qualitative support on relying on the Clinton Creek wastes and dump failures as cautious support of a "A Barrier not a Dam" concept. The length of the landslide dams on Wolverine Creek are insufficient to argue as being a barrier, and hence, should not be considered a dam. The dumps into Porcupine Creek are currently not impounding much water; this condition is being maintained by flow through or under the dumps, but this may not be sustainable in the long term, if the flow channels become plugged by in-flowing debris. Alternatively, if melting of ice rich permafrost were to lead to increased flow through the colluvium, and piping of fines lead to a loss of support and a subsequent collapse, this could result in cessation of flow.

All remnant waste piles would have to be shaped in a manner so that there could not be a future failure of the pile that would cause the dam to be re-established. Alternatively, the Project Parties would have to commit to removing the failure mass and re-establishing the channel if this occurred. But as this would mean a major future remedial effort in the case of seismic action, recommendations are made for flattening waste dump slopes so that the failure of any slopes could not create a new landslide dam.

2.2 Adaptive Management

"Adaptive Management" and its close relative the "Observational Method" does not mean a do nothing and fix it later methodology. Instead of seeking precise predictions of future conditions, and being paralyzed into perpetual inaction, adaptive management recognizes the uncertainties associated with forecasting future outcomes and calls for consideration of a range of possible future outcomes. Management policies are designed to be flexible and are subject to adjustment in an iterative, learning processes. Adaptive management is intended to increase the ability to fashion timely responses in the face of new information and in a setting of varied stakeholder objectives and preferences. It encourages stakeholders to bound points of contention and discuss them in an orderly fashion while environmental uncertainties are being investigated and better understood.

In this context, a major geotechnical focus can be to obtain a base design that is robust and sustainable – and, in this context, "fixable" with pre-planned effort. For the remote Clinton Creek site, the time required to mobilize in the resources required should also be considered.

2.3 Dam Classification – Consequence Based

Previous studies reported by TTEBA (2016) and Amec Foster Wheeler (2017) have presented discussion that has led to a "Significant" risk classification. The CDA Guidelines recommend a classification system based on assessment of consequences. Consequences of a dam failure may include loss of life, property and environmental damage, and general disruption of the lives of the population in the inundated area. In addition, the release of stored tailings, accumulated silt or impacted waters could also have detrimental environmental effects, including impacts on aquatic habitat, recreational property and activities, and various infrastructure. Table 2-1 presents this approach.

Table 2-1: CDA Classification System

Dam Class	Population at Risk [Note 1]	Incremental Losses		
		Loss of life [Note 2]	Environmental and Cultural Values	Infrastructure and Economics
Low	None	0	Minimal short-term loss. No long term loss.	Low economic losses; area contains limited infrastructure or services.
Significant	Temporary Only	Unspecified	No significant loss or deterioration of fish or wildlife habitat. Loss of marginal habitat only. Restoration or compensation in kind highly possible.	Losses to recreational facilities, seasonal workplaces, and infrequently used transportation routes.
High	Permanent	10 or fewer	Significant loss or deterioration of *important* fish or wildlife habitat. Restoration or compensation in kind is highly possible.	High economic losses affecting infrastructure, public transportation, and commercial facilities.
Very high	Permanent	100 or fewer	Significant loss or deterioration of *critical* fish or wildlife habitat. Restoration or compensation in kind possible but impractical.	Very high economic losses affecting important infrastructure or services) e.g., highway, industrial facility, storage facilities, for dangerous substances).
Extreme	Permanent	More than 100	Major loss of *critical* fish or wildlife habitat. Restoration or compensation in kind impossible.	Extreme losses affecting critical infrastructure or services, (e.g., hospital, major industrial complex, major storage facilities for dangerous substances).

Source: Dam Classification (Table 2-1, CDA 2007)

Note 1. Definitions for population at risk:
None – There is no identifiable population at risk, so there is no possibility of loss of life other than through unforeseeable misadventure.
Temporary – People are only temporarily in dam-breach inundation zone (e.g., seasonal cottage use, passing through on transportation routes, participating in recreational activities).
Permanent – The population at risk is ordinarily located in the dam-breach inundation zone (e.g., as permanent residents); three consequence classes (high, very high, extreme) are proposed to allow for more detailed estimates of potential loss of life (to assist in decision-making if the appropriate analysis is carried out).
Note 2. Implication of loss of life:
Unspecified – The appropriate level of safety required at a dam where people are temporarily at risk depends on the number of people, the exposure time, the nature of their activity, and other conditions. A higher class could be appropriate, depending on the requirements. However, the design flood requirement, for example, might not be higher if the temporary population is not likely to be present during the flood season.

TetraTech EBA previously completed a preliminary dam classification for the site (Tetra Tech EBA, 2016), and the dam classification for the Clinton Creek Waste Dump was determined to be "Significant" based on the following assumptions:

- There is no permanent population at risk;
- The habitat at risk is marginal and could be restored; and
- Only recreational facilities, seasonal workplaces, and infrequently used transportation routes are at risk.

As part of the same preliminary dam classification, a dam classification of "Significant" was also assigned to the Wolverine Tailings Pile. The analyses leading to consequence assessment and classification of the dams generally include characterization of a hypothetical dam breach, flood wave routing, inundation mapping, and evaluation of the impacts. A wide range of methods could be applied in each of these steps, the choice depending on the information available and the expected level of detail. A dam breach and inundation study was carried out as part of a risk assessment report for both the Clinton Creek Waste Dump and the Wolverine Tailings Pile (UMA Engineering Ltd., 2000), and was updated in 2016 (Tetra Tech EBA, 2016). The information was reviewed and deemed sufficient in Amec Foster Wheeler (2017) to make an informed assessment of the consequence classification for both the Clinton Creek Waste Dump and the Wolverine Tailings Dump. The same classification should be applied to the Porcupine Creek Waste Dump. Further Amec Foster Wheeler (2017, Table 5.2) presented a more detailed analysis of the factors involved and supported the conclusions reached by TetraTech EBA (2016).

It should be noted the incremental consequences of failure of the dams need to be evaluated for the following two scenarios:

- Sunny day (fair weather) – to determine the design earthquake which is used to develop the Earthquake Design Ground Motion (EDGM); and
- Flood – to determine the Inflow Design Flood (IDF); hydrotechnical issues are presented separately.

However, current design practice is to not conflate either the seismic or EDGM design criteria being operative in addition to IDF criteria at the same moment in time.

2.4 Considerations for Hydrotechnical and Geotechnical

While it is usual to define the Dam Classification as "fixed" for all components of a typical dam, the options involving a dam allow for a more logic-based approach when selecting the stochastic loadings which are discussed as follows. Hydrotechnical issues are discussed in more detail in other project documentation.

For those options considering the long-term presence of Hudgeon Lake, or a lake behind the Wolverine Tailings Dump landslide, the consequences of failure of the reservoir outlet has to be managed. Herein we consider a "spillway" being a civil engineered structure that controls the erosive forces of water on gradients steeper than a regime case. A "regime channel" describes a more natural river channel, one that is in balance with erosion or deposition in the channel. The complete failure of a spillway during a selected design event will create a downstream flood event of greater consequence than the failure of a regime channel for the same design event. Or, alternatively, it could be stated that while a dam breach consequent upon a spillway failure should have a "Significant" classification; it would be logical to adopt a "Low" classification for a regime channel.

For all options, maintaining the hydrotechnical design configurations means that the waste or tailings slopes need to be configured in such a way that the spillway or regime channel will remain open (ie, no post-construction sliding can occur that would re-block the newly constructed channel). That is, and especially due to the potential for liquefaction, significant slope failures could re-dam the valley, creating a new breach potential. Therefore, while it might be acceptable to design a regime channel with a Low classification, it would not be appropriate to use the same level of classification for slope stability impinging on the regime channel.

2.5 What does Closure mean for Stochastic Events

During the 22 January 2019 Workshop#2, a Project Partner raised the issue of "what does long term mean in respect to stochastic design levels", and should not a Project Maximum event be selected, given closure means forever? Ultimately, the Project Partners need to make this decision, however, the following issues are worth discussing.

For seismic loading, considering Significant classification loading levels (1/2475 or 2%/50 years), liquefaction is triggered, based on current SPT and CPT data sets, for both the Clinton Creek Waste Dump and the Wolverine Tailings Dump. The mobilized strength of liquefied deposits is not impacted by the triggering event, as such. If a 2%/50yrs triggers liquefaction, the design control is the post seismic strength; if a less probable design maximum event occurred, the triggering of liquefaction would just be even more certain.

In order to manage and survive Project Maximum Flood (PMF) levels, a spillway with a lake at elevation 412 masl is a commitment to long term maintenance, repair, and ultimate replacement. A regime channel with an approximately 10 m lake drawdown is a more survivable event, but likely will result in additional lake drop over time; however; consequences are judged to be much less than a spillway failure. The only sure long-term choice is complete removal of the lake and most of the mine wastes in the Clinton, Wolverine and Porcupine Creek valleys.

3.0 Seismic

3.1 Peak Ground Acceleration (PGA) for site Class C

The Clinton Creek Mine site is located in a moderately high seismic zone as per the 2015 Seismic Hazard Map published by Natural Resources Canada. A site-specific seismic hazard assessment was completed for the Clinton Creek site to determine the peak ground acceleration (PGA) (National Building Code of Canada (NBCC), (NRCan, 2015). Table 3-1 below provides the Earthquake Design Ground Motion (EGDM) PGA for firm ground for different annual exceedance probabilities (AEP) at the site, updated in 2018 in the NBCC (NRCan, 2015).

Table 3-1: EDGM (Earthquake Design Ground Motion) Peak Ground Acceleration Values: Clinton Creek

AEP	100 40%/50	475 10%/50yr	1,000 5%/50yr	2,475 2%/50yr
PGA (g) – Firm Ground, (Soil Class C)	0.05	0.11	0.15	0.27

For earthquake design, the AEP, or return period, will be used to select the site-specific PGA. Natural Resources Canada (NRCan) provided deaggregation of the 2%/50yr statistical events leading to the PGA. A mean moment magnitude (Mw) of 6.2 at a distance of 24 km was predicted. There is a lesser probability of the EDGM being generated at a Mw of 7-8.

CDA (2014) provides the guidance for operational periods as per CDA Table 3-3 (presented in Table 3-2). For a Significant Classification, the AEP is recommended between 1/100 and 1/1000.

Table 3-2: Target Levels for Earthquake Hazards, Standards-Based Assessments, for Construction, Operation, and Transition Phases

(For Initial Consideration and Consultation Between Owner and Regulator)

Dam Classification	Annual Exceedance Probability – Earthquakes (note 1)
Low	1/100 AEP
Significant	Between 1/100 and 1/1,000
High	1/2,475 (note 2)

wood.

Dam Classification	Annual Exceedance Probability – Earthquakes (note 1)
Very High	1/2 Between 1/2.475 (note 2) and 1/10,000 or MCE (note 3)
Extreme	1/10,000 or MCE (note 3)

Source: (Table 3-3, CDA 2007)

For the situation of the three dams at the Clinton Creek Mine site, in consideration of closure, the recommendation for a Significant Classification is an AEP of 1/2750 per CDA Table 4-2, presented in Table 3-3.

If the tailings and waste dumps are no longer functioning as dams, then CDA criteria do not apply. The only related guidance available for waste dumps is a draft document prepared by the BC Mine Dump Committee, (1991), which is provided in Amec Foster Wheeler (2017). The design recommendation at the time was to use the 1/475 or 10%/50yr AEP.

Conclusions are as follows:

1. Use AEP at 1/2750 for all water retaining structures, or structures that could fail and recreate landslide dams, or significantly interfere with remedial design measures: PGA = 0.27; and
2. Use AEP at 1/475 for all other dumps with PGA = 0.11. This value is to be used for dumps that are sufficiently removed from creek valleys so as to not cause a dam in the event of failure.

Table 3-3: Target Levels for Earthquake Hazards, Standards-Based Assessments for Closure – Passive Care Phase

(For Initial Consideration and Consultation Between Owner and Regulator)

Dam Classification	Annual Exceedance Probability – Earthquakes (note 1)
Low	1/1000
Significant	1/2,475 (note 2)
High	1/2 Between 1/2,475 (note 2) and 1/10,000 AEP or MCE (note 3)
Very High	1/10,000 AEP or MCE (note 3)
Extreme	1/10,000 AEP or MCE (note 3)

Source: (Table 4-2, CDA 2007)

Acronyms: MCE - Maximum Credible Earthquake, AEP - Annual Exceedance Probability

Notes:

1. Mean values of the estimated range in AEP levels for earthquakes should be used. The earthquake(s) with the AEP as defined above is (are) then input as the contributory earthquake(s) to develop Earthquake Design Ground Motion (EDGM) parameters as described in Section 6.5 of *Dam Safety Guidelines* (CDA 2013).
2. This level has been selected for consistency with seismic design levels given in the National Building Code of Canada.
3. MCE has no associated AEP.

3.2 Stability Analysis West-East along Clinton Creek

Previous analysis has considered design motions along the entire cross-section along the Clinton Creek valley to be in-phase. As this dump cross section is in the order of 1.3 km long, it is highly unlikely that the shear wave train for any design event is coherent along this length of structure and that the possible ground motion will be in phase over the entire length of the dump.

4.0 Mobilized Strength

Stability analysis of waste rock and tailings dumps requires the identification of the appropriate strength and pore water pressure parameters. Due to the variable foundation conditions, there are several potential strength models. In conventional practice the appropriate questions are whether or not the strength model is either drained ("frictional") or undrained ("cohesive") and what are the effective stresses at the moment of presumed failure. Porewater pressures in excess of measured insitu pore pressures can be induced by undrained loading during construction (generally unlikely for closure design) or due to thawing. Excess pore pressures can also be induced during undrained shearing in contractant soils (unfrozen).

The presence of permafrost in both the foundations and the dumps complicates the designer's perspectives. The native or insitu permafrost soils are relatively warm with temperatures below but close to the freezing point of water; and could be in the range of -0.1°C to -0.5°C at the site. Based on observed temperature gradients these frozen deposits are predicted to be slowly thawing, particularly where covered by thick waste or tailings covers. Questions such as: can thawing create excess pore water pressures; what is the mobilized strength at a thaw interface; what is the strength of frozen but warm permafrost; is movement in dumps overlying frozen deposits due to rheological creep in the frozen zones or slippage at the thaw interface, need to be addressed.

At this stage of site characterization, absent information from slope indicator installations, some movement mechanisms cannot be ruled out. At this time, it is prudent to focus on what the lowest strength models might be. Various approaches are discussed in the following sections Note that in some analyses several strength models will be considered, given the uncertainties of long-term conditions, especially thermal state and long-term phreatic conditions.

4.1 Back Analysis

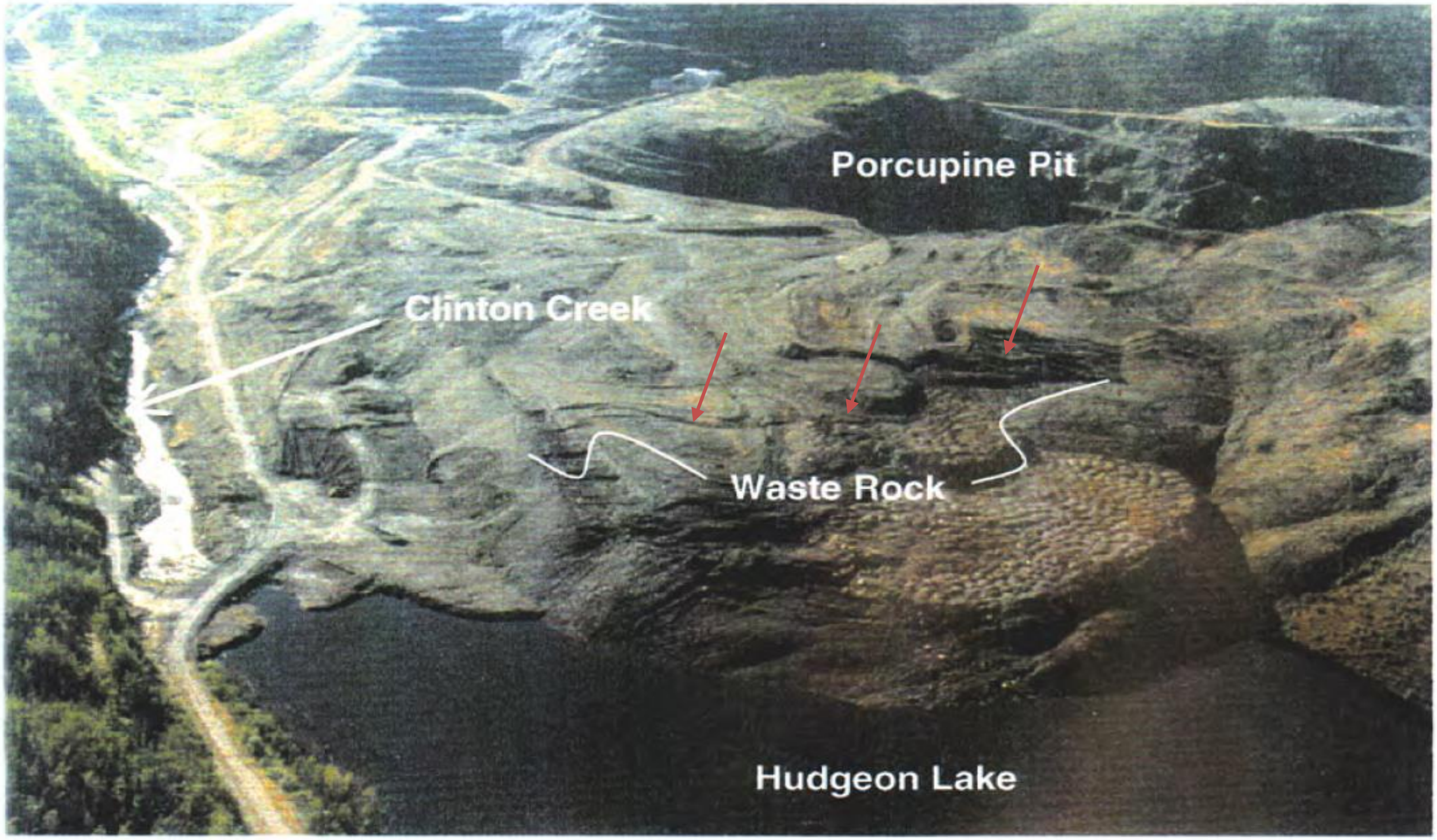
If a given slope cross-section is assumed to be essentially at the point of failure, then back analysis can suggest the lower bound <u>mobilized</u> insitu strength. At this time, and absent the inferences that could be drawn from slope indicators it will be assumed that movements could be occurring along the interfaces between waste rock or tailings and insitu ground, as that is where the most likely location of a weak layer can be found. Borehole data coupled with recent InSAR satellite movement measurements can be used to provide a range of zonation of ground condition, such as soil types, thermal state and low versus high ice content, along presumptive failure planes. However, seismic considerations cannot be "back analyzed" as EDGM level ground motions are not likely to have been experienced at this site since development began.

In this regard we need to keep in mind that the major slopes of interest have generally failed into their current configuration. Failures have pushed waste dump fills up to about 30 m up the north valley slope as measured from the original valley bottom. Figure 4-1 presents a view of this condition circa 2000, downstream of Hudgeon Lake.

Figure 4-2 provides a perspective looking east from Hudgeon Lake. The scale of the failures extending well back into the dumps is apparent. It is clear that these slopes have failed historically. The original mechanism considered in most, if not all, geotechnical reviews circa the 1970's was that the thawing of ice rich permafrost was the dominant cause of the movement. Site investigation has not encountered any ice-rich permafrost in the valley bottom but ice rich colluvial soils have been found along the valley walls. This absence can be explained either because it wasn't there in the first place (only locally present) or has since all thawed out. If it was permafrost thaw creating excess pore pressures, then this mechanism is no long present, which implies that the factor of safety of the slope has improved. The possibility of local static liquefaction of the dump material being the original cause was not previously advanced and, as was typical in the overall industry at this time, this mechanism was discounted, especially in the waste dumps in the coal mines of British Columbia. In any event, it is prudent to assume that the FoS of these slopes is near unity and a gain of 20% in the FoS would be required for closure.

View of the Clinton Creek Waste Rock Dump circa 2000, up against the north slope of the Clinton Creek valley. Note there has been downcutting through the waste rock and into the north valley slope.

Wood Environment & Infrastructure Solutions	PROJECT: Clinton Creek Design Basis Memorandum				
	TITLE: View of Waste Dump Moved Against Eroding North Valley Slope				
CLIENT: Government of Yukon	DATE: August 2019	JOB No.: VE52705D	FILE: DBM Figures.xlsx	FIGURE No.: 4-1	REV. A



View of the Clinton Creek Waste Rock Dump facing east. Red arrows show the location of visible cracking indicating the extent of the slope failure.

Wood Environment & Infrastructure Solutions wood.	PROJECT: Clinton Creek Design Basis Memorandum				
CLIENT: Government of Yukon	TITLE: Clinton Creek Waste Rock Dump facing east				
	DATE: August 2019	JOB No.: VE52705D	FILE: DBM Figures.xlsx	FIGURE No.: 4-2	REV. A

This approach is intentionally less than a FoS of ≥1.3-1.5 as would be required by CDA based on presumed stability and measured design parameters.

4.2 Frictional Strength

The project is well supplied with drained or frictional strengths. Use of these strengths signify that the insitu soils are sufficiently dense that shearing results in dilative response on any form of drained or undrained loading, including seismic action. Such strengths are essentially the highest strengths that could be used. Drained strength can be relied on for thawed dense deposits. In addition, frozen low ice (water content) permafrost, especially with temperatures near melting point, will also tend to be frictional. There is little guidance in the literature as to whether granular soils could also be susceptible to liquefaction post thaw. It is necessary to undertake insitu testing in such soils, and some CPT and SPT site data may be applicable; however, it is difficult to deduce if some tests were or were not thawed.

A summary of drained strength data undertaken on the project is summarized in Table 4-1. Estimating the frictional behaviour of weathered argillite is a significant consideration for waste dump designs. Site inspection, including exposures of failed waste dumps sub-cropping along the spillway channel and borehole samples indicate the prevalence material with fines contents from 15% to 40%, dominated by mechanically disturbed argillite. Testing reported by Golder Associates (1978) indicate peak friction angles in two test series of 26° and 27°, while one test at 690 kPa reported a high strain peak of 25°. For design purposes, we assume higher normal stress levels. The “working face” of a dump scree slope, at typically 35° to 40°, would represent the maximum friction angle encountered in this material at low stress levels.

The characteristics of alluvium and colluvium are presented in Wood (2019) Geotechnical & Geological Site Characterization and Model.

Table 4-1: Summary of Project Drained Test

Peak Shear				
Soil Type	**Number of Tests**	**Max (°)**	**Min (°)**	**Average (°)**
Tailings	2	45.0	35.0	40.0
Fluvial Lacustrine Gravel	9	32.5	27.5	29.6
Alluvium	1	-	-	38.6
Weathered Argillite	5	40.0	26.0	29.2
Waste Rock*	1	-	-	32.9
Residual Shear				
Soil Type	**Number of Tests**	**Result (°)**		
Tailings	1	30.0		
Overburden / Colluvium	1	23.0		

* One sample indicates c' of 10.7 kPa

4.3 Thaw Induced Excess Pore Water Pressure

When a deposit that contains excess ice thaws, a thaw strain potential is created. If it can be calculated that the water created by an increment of thaw (adjusting for volume change of ice on phase transformation) can dissipate away from the thaw front, the process is drained. If thawing is relatively fast and water cannot dissipate into overlying thawed deposits, then the superincumbent loading creates high pore pressures in the thawed zone due to undrained loading. These pore pressures are much higher than the hydrostatic pore pressure as predicted with depth below the water table, and are referred to as "excess" pore pressures. Analytical approaches for this mechanism are available in the permafrost literature.

During the original failure of both the waste and tailings dumps, higher rates of thaw would have been likely and was considered by investigators (at that time) to be a likely cause of instability. Amec Foster Wheeler (2018) speculated that there could have been other failures modes. However, the thick waste and tailings deposits overlying the frozen zones at the current time provides an insulating effect and much reduces the rate of thaw than what could have been expected during waste or tailings placement.

For most cover thicknesses, available temperature gradients predict current rates of thaw in the order of 5-10 cm/yr. It is considered unlikely that excess pore pressures can be generated at this rate of thaw; however, thaw consolidation induced excess pore pressures may be an issue for shallow slope failure modes in some cases. Locally, for example, under the spillway where warm lake water percolates deeper into the dump, it is possible that thaw rates are somewhat increased by heat convection.

4.4 Undrained strength of thawed insitu soils

The undrained strength of thawed soils, or a "total stress" approach, can be considered for stability analysis, in terms of Critical State Mechanics or CSM. Here we postulate that the soil is contractant and excess pore pressures are generated upon shearing (as distinct from upon thaw, or other undrained loading as discussed above). Undrained strength is normalized by the insitu effective stress in the thawed soil. This expressed by an undrained strength ratio:

$$S_{us}/\sigma'_v = 0.2[(OCR)^\Lambda]$$

where 0.2 is the strength ratio in a direct simple shear mode, OCR is the over consolidation ratio and Λ is an empirical factor usually taken as 0.85. For a normally consolidated material, OCR = 1, the strength ratio is therefore 0.2 and is equivalent to a friction angle of about 12 degrees.

There are limited data sets in the literature for undrained strength of thawing soils; one exception is Watson et al (1973), where the strength was based on thawed, consolidated and undrained testing. These tests can be interpreted to indicate a lower bound of $S_{us}/\sigma'_v = 0.2$ and an OCR of 1.8 up to a rather low vertical effective stress of 75 kPa. However, lower undrained strength ratios < 0.2 can be encountered at high post-peak shear strains.

Frozen and thawed clayey soils generally indicate over consolidation effect due to the freeze/thaw cycle and will typically result on the value of OCR > unity. For slope foundation segments located in thawed native soils previously considered to be ice-rich, an undrained strength ratio of 0.2 would be conservative, but not unrealistic, under substantial vertical loading.

4.5 Strength of Frozen Soil

If slope inclinometers eventually reveal rheological creep in ice rich permafrost is occurring, the stability issue becomes what applied stresses will result in creep rupture. The available guidance in permafrost engineering is to consider the "long term" strength of frozen soils. There are two sources of relevant strength data for determining the frozen strength: the first is geotechnical laboratory testing, the second is back analysis of glacier basal slippage.

Figure 4-3 provides a summary of long-term strength data for ice rich soils from data bases. Generally, no testing warmer than -0.5°C is available in the data bases. The long-term strength data for the individual data points was obtained using a Vialov / Sayles type log time extrapolation to 100 years. This is accomplished by conducting compression tests at various times to failure and projecting the reciprocal of the failure stress versus log time, as was the method used for the data in Figure 4-3. Equation 7.25, as cited in Figure 4-3, from Johnson (1981) uses other case records and provides a conservative long-term strength. Mobilized strength between 40 kPa and 80 kPa for temperatures warmer than -0.5°C could be expected for ice rich finer grained soils. Other data for sandy silty soils demonstrates mobilized strength 2 to 3 times these values. For ice poor soils the same procedures using varying confining stresses indicate long term frictional strengths.

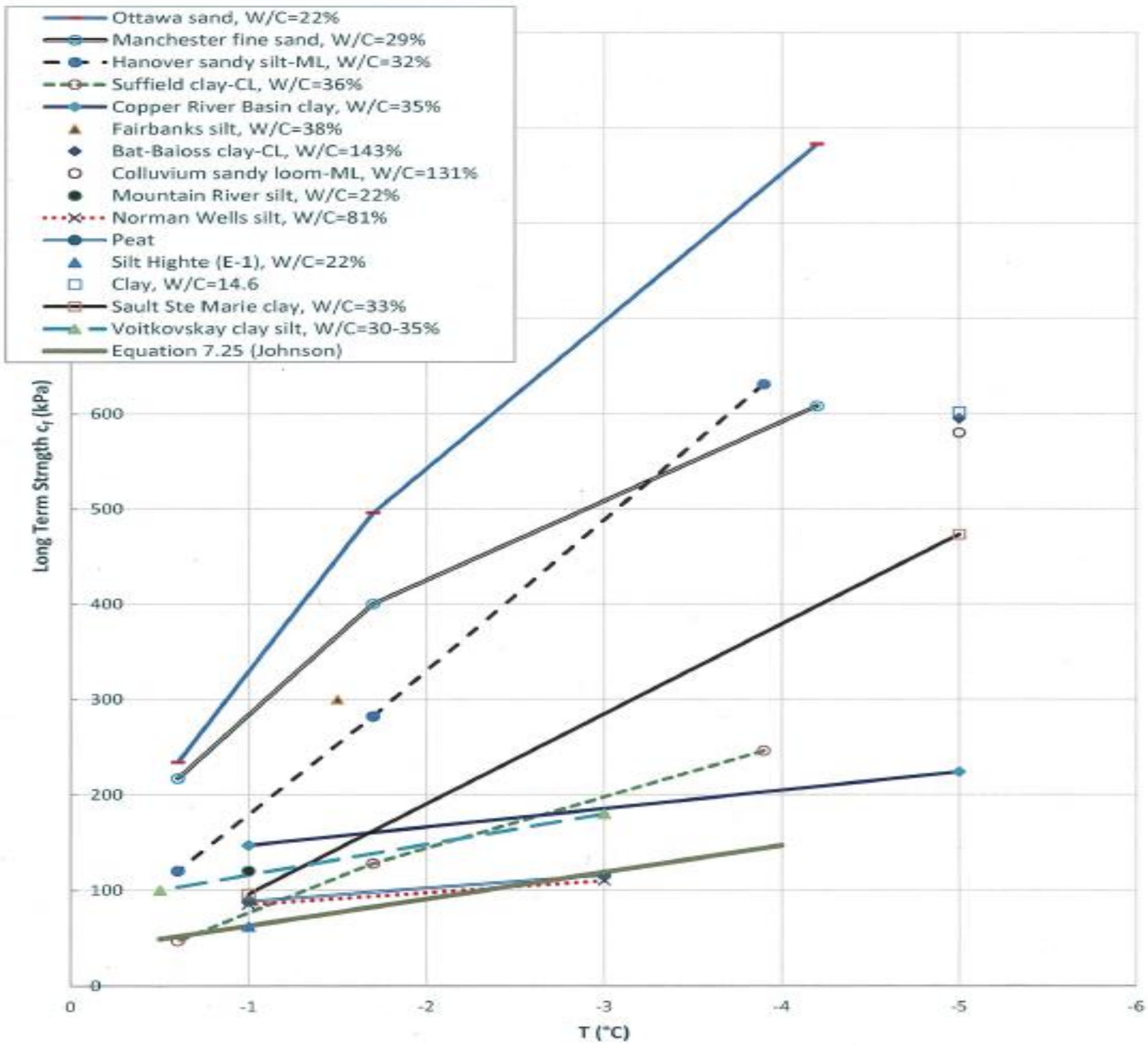


Figure taken from Johnson (1981)

Wood Environment & Infrastructure Solutions — wood.	PROJECT: Clinton Creek Design Basis Memorandum				
	TITLE: Long term strength data for ice rich soils from databases				
CLIENT: Government of Yukon	DATE: August 2019	JOB No.: VE52705D	FILE: DBM Figures.xlsx	FIGURE No.: 4-3	REV. A

Recent work by Meyer et al (2018) considers the driving stresses mobilized to rationalize the effects of pressure melting beneath continental and alpine glaciers. Figure 4-4 presents Figure 1 from Meyer el al (2018). The median driving stress for hundreds of glaciers is between 60 and 70 kPa, and a 33 percentile/ 67 percentile split is in the region of 40 kPa. As slip is presumed, then the mobilized strengths over a wide range of glacier thickness is therefore equal to the driving stresses. Note that glaciers move with a combination of slip at the contact, and creep throughout the mass.

The rationale for considering the back-analysis is that the basal temperatures are considered to be at or near the pressure meting point of ice. That is to say, the mobilized strengths are “equivalent” to icy very warm permafrost very near melting due to thermal, but not pressure, effects. The reasonable concordance between the laboratory data and glacier slip mobilized strength is interesting but may be fortuitous.

4.6 Liquefaction of Waste, Alluvium, and Tailings

Liquefaction of loose, coarser grained deposits is a potential stability threat. Triggering of liquefaction can be caused by undrained static loading or seismic action. Raising a structure higher relatively rapidly can trigger static liquefaction. Continued loading of loose granular materials is a possible mode for static liquefaction of Clinton Creek valley deposits, which is supported by the SPT data in the Clinton Creek Waste Dump. For an existing dump, a rising water level resulting in a decreasing in the mean stress (through a rising phreatic level) while maintaining a constant shear stress, can also trigger liquefaction. Dump failure caused by increasing water levels, due to Hudgeon Lake being impounded, is also a viable triggering mode.

Seismic action can also trigger liquefaction and given the PGA at 2%/50yrs and a Mw =6.2 at 24 km epicenter, or M =7-8 at more remote locations, is a realistic liquefaction threat.

Two sets of data are available to predict the undrained strength of loose soil if liquefaction is triggered. These are:

1. Standard Penetration Test (SPT) data presented in Amec Foster Wheeler (2016) for the tailings, waste dumps and insitu deposits underlying the dumps (in the Clinton Creek Waste Dump only); and
2. Cone Penetrometer Testing (CPT) data in the tailings in 2018 at four locations.

The 2016 program reportedly used a LPT (Large Penetration Test) rather than a SPT, because of the presence of rock fragment and to obtain a larger sample. LPT blow counts must be converted to equivalent SPT blow counts using appropriate methodology for empirical correlations. As the CPT data indicated less dense tailings versus converted LPT values reported in 2016, a review of the conversion approach was undertaken and modifications to the conversions are presented in Appendix I. Information on converted N1(60) SPT equivalents reported in Amec Foster Wheeler (2016) have been modified for all locations.

The selection of appropriate liquefied shear strengths can be considered in the 10% design stage considering Robertson (2010), Olsen and Stark (2003), and Idriss and Boulanger (2014). In general terms, a sensible lower bound of the data indicates that if deposits are saturated, then liquefaction concerns must be addressed.

It is noted that the granular deposits can liquefy during either what is referred to as static loading, or dynamic loading. While the trigger event will be different in these two cases, the resulting strength is the same.

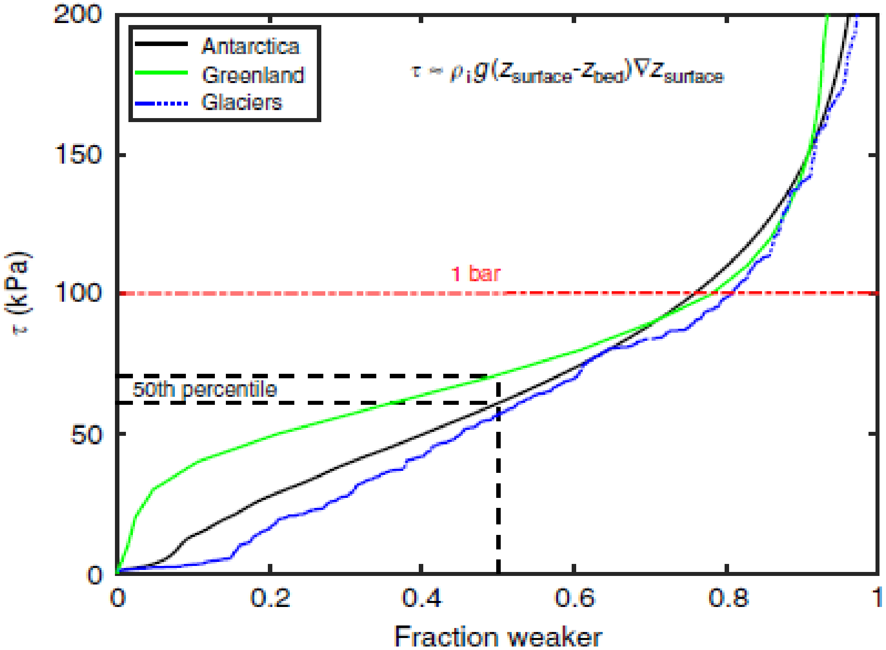


Fig. 1 Driving stress for glaciers and ice sheets. Average driving stress for Antarctica (black) and Greenland (green) plotted as a function of the fractional area that is weaker. Antarctic values were calculated from BEDMAP2 data reported on a 1 km grid[30]. Greenland values were extracted from MacGregor et al.[6] based on surface slopes from GIMP[31] and ice thicknesses from Morlighem et al.[27]. Also, shown for comparison (dotted blue) are approximate shear stress values calculated from average thicknesses and slopes (not colocated) plotted as the proportion weaker among 449 pairs of these parameters tabulated for Alpine glaciers in the GlaThiDa data set[8]

Figure taken from Meyer et al (2018)- Figure 1

Wood Environment & Infrastructure Solutions	PROJECT: Clinton Creek Design Basis Memorandum				
CLIENT: Government of Yukon	TITLE: Driving Stress for Glaciers and Ice Sheets				
	DATE: August 2019	JOB No.: VE52705D	FILE: DBM Figures.xlsx	FIGURE No.: 4-4	REV. A

5.0 Factor of Safety & Methods of Analysis

5.1 Static Stability Criteria

Static stability analysis will be undertaken using Slope/W software. At this stage of design there appears to be no benefit in considering 3-D analysis.

Factors of Safety (FoS) are a means of accounting for uncertainty. Uncertainty arises from the suitability of the analytical technique, the strengths being used, the confidence in the stratigraphy, allowable deformations, and the consequences of failure.

For a given cross section under consideration, if mobilized strength parameters are obtained by back analysis, assuming the section is on the point of failure, then a lower FoS than might be used under other circumstances is justified. For this case, a minimum **FoS = 1.2** will be adopted when arriving at the appropriate remedial measures. Keeping in mind that existing slopes will have a FoS > 1.0 (as the slope is still standing) then the resulting FoS will be somewhat above **1.2** in reality.

For slope analysis based on a "likely assessment" of stability parameters, a **FoS = 1.3** will be used to manage uncertainty in parameter selection. This is more generally what is expected in CDA guidelines for operational conditions, while the CDA guidelines consider a **FoS =1.5** for dams in closure. Depending on the design scenario considered, the closure FoS = 1.5 may be required.

Seismic liquefaction triggering will be considered, and post liquefaction strengths used in analysis. Slopes will be designed such that in the situation considering the post liquefaction strength the **FoS ≥ 1.0**. This may govern the design of some slopes, over-riding the static stability requirements.

For the tailings and waste dumps based on a back analysis, a **FoS ≥ 1.2** will be required, while the required FoS in consideration of liquefaction threats (either static or dynamic) will require a **FoS ≥ 1.0**.

5.2 Seismic Stability & Liquefaction Triggering

At this stage of design, the 10% design phase, seismic stability will be assessed using simple models. The issue of liquefaction requires consideration. Recent 2018 CPT testing in the tailings, as well as re-interpretation of the 2016-LPT/SPT conversions, indicate that, if saturated, the tailings and the waste would exhibit low undrained strength ratios. If required, more sophisticated triggering analysis can be considered.

The Cyclic Stress Method as recently reviewed by Idriss and Boulanger (2014) based on Idress (1971) will be considered to interrogate the potential for the EDGM liquefaction trigger. This method may have some utility for the waste dumps, but likely not for the steep tailings slopes. Overall, valley wall slopes in the tailings from 17° to 25° will likely exhibit shear stress very close to that required to trigger cyclic liquefaction.

5.3 Creep (FLAC) Model

Amec Foster Wheeler (2018) presented a FLAC based creep model of the Clinton Creek Waste Dump and the Wolverine Tailings Dump. Extensions to this work can be considered post assessment of slope indicator data.

6.0 Other Design Issues

6.1 Long Term Climate Variations

The presence of permafrost and associated long-term potential impacts introduce an additional level of uncertainty and complexity in the design. The following points are made:

1. The insulating effect of the tailings or waste and observed gradients near the unfrozen / frozen contact limit future thaw rates from heat conduction process to relatively low rates, in the order of 10 to 60 mm/yr. The potential for heat convection due to seepage out of Hudgeon Lake could, in principle, increase these thaw rates. Convective heat transport may also result from water flows out of the Clinton Creek spillway or regime channel;
2. Based on global warming projections, eventually all permafrost would be expected to degrade. Permafrost depths are likely in the order of at least 30 m at this site, and there still is a potential for some long term thaw based on the deep geothermal gradient from the Earth's core. That is, the existence of waste dumps and tailings 30 to 60 m thick would result in a new surface temperature equilibrium, and the geothermal gradient would then slowly thaw deeper permafrost from below, thinning, if not actually eliminating, the original now deeply buried permafrost;
3. With either model, long term thaw settlement of ice rich layers will take many decades and could result in up to 4 to 5 m of settlement. Heat convection from seepage may increase thaw rates and, therefore, settlement rates; and
4. While settlement is a design issue especially for the Spillway/Outlet, locally we might expect excess pore pressures under relatively thin surface covers due to thaw consolidation effects. These could have locally destabilizing effects.

6.2 Seepage Considerations in Clinton Creek and Porcupine Creek Valleys

A seepage model from Hudgeon Lake through the Clinton Creek Waste Dump to the downstream spillway outlet to Clinton Creek and, similarly, down Porcupine Creek, requires consideration for several design related tasks:

1. Does the presence of any frozen waste dump fills have any control on current seepage performance, relative to absence of any notable seepage exiting the dumps to the downstream (ie, are there frozen zones that are currently acting as seepage barriers)? If, in the long term, all permafrost thaws, then could adverse conditions develop;
2. How much convective heat flux is generated by flow out of Hudgeon Lake? It is noted that in summer months the temperature of the water in the top layers of the lake could be as warm as 9°C at surface to 4°C at 10 m depth and approaching the freezing point at the lake bottom. As warmer water is decanted over the spillway, it might be expected that the thaw potential contribution from outflow could be more significant than overall seepage into the dumps; and
3. The Porcupine Creek Waste Dumps have not significantly dammed the creek, as flow exits through the dumps. How the flow interacts with Creek Pit and Snowshoe Pit lakes further downstream is not currently determined.

6.3 Internal Erosion

Consideration of internal erosion issues are being undertaken under separate cover. While Schuster (2003) provides considerable qualitative support for stable conditions in landslide dams, recent summaries by Chapuis (2016) will be considered to assess the potential from a quantitative perspective.

Overall, internal erosion, as well as localized seepage / internal erosion into a spillway or regime outlet, requires consideration.

6.4 Reservoir Seiche

Reservoir seiche, or landslide induced waves, in Hudgeon Lake require consideration. It is judged that the likely failures of the natural slopes surrounding the lake would tend to be shallow detachment slides (ie, deep seated bedrock failures are not expected). Seiche developed by failure of the dumps directly into Hudgeon Lake, following closure construction, is controlled by design.

The most significant potential threat from seiche would be during a drawdown of the lake moving towards either a reduced lake level or no lake scenarios (ie, rapid drawdown could result in failure of the waste slopes, or reservoir slopes into the lake).

Sincerely,

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28 August 2019

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Appendix I
LPT to SPT Conversion

LPT to SPT Conversion

The SPT (Standard Penetration Test) is used to both collect disturbed samples for soils description and logging borings, but also as an empirical guide based on the number of standard blows "N" to drive a standard sampler for a 12 inch penetration. In order to enhance the ability to obtain large grain size samples a LPT (Large Penetration Test) with a large diameter sampler can also be used. However, as the LPT is often used with a larger driving weight, the LPT-N must be corrected to obtain an equivalent SPT-N, in order that empirical correlations can be used. Re-calculation of 2016 Amec Foster Wheelers LPT to SPT correction factors used in previous Amec Foster Wheeler reports was undertaken. This was initiated by noting the differences between 2018 CPT (Cone Penetration Test) and the 2016 SPT equivalents, expressed as a normalized and energy corrected $N_{1,60}$, and led to a review of previous calibration.

The LPT used in the 2016 Amec Foster Wheeler program, as initially determined by scaling off of sampler dimensions in the original core photographs and confirmed with Midnight Sun Drilling, employed a 3-inch OD and 2-3/8 inch ID LPT shoe This size of sampler is often referred to as a NALPT (North American LPT), see Daniels et al (2003). The SPT uses a sampler with a 2-inch OD and 1-3/8 inch ID and a 140 lb hammer dropped 30 inches. Usually the NALPT utilizes a 300 lb weight dropped either 20 inches or 30 inches.

Initial testing in 2016 began at the tailings deposits at Wolverine Creek. A NALPT was deployed but with a SPT trip hammer (2 tests were done with a SPT sampler). This means that an energy correction is not required for hammer energy. However, the previous corrections were based on an assumed sampler of 2-1/2-inch OD and 2 inch ID shoe; whereas a 3-inch OD and 2-3/8 inch ID LPT shoe was used. Therefore, re-correction for this factor was required. As the 2016 field work moved into the waste rock holes, an additional 140 lb weight was added into the trip hammer for a total of 280 lbs, not 300 lbs. Moreover, as for the Midnight Sun device, the two weights drop together one over the other, but are not mechanically connected. It is considered that some energy is lost due to the softness of this system requiring an empirical adjustment given the absence of field energy measurement.

Daniels et al (2003) provides the methodology to convert LPT to SPT. For the tailings tests the correction factor following the Daniels methodology predicts a correction factor of 0.65. Rogers (2006) presents a detailed comparison of the calculated Burmister correction factor (identical to Daniels for the same NALPT sampler) and a regression analysis which reports a correction factor of 0.55. It was concluded that a correction factor of 0.60 be used (that is applied to the raw field LPT results). Further correction for stress level and other factors provide the final $N_{1,60}$. Previously a correction factor of 0.94 was used.

In order to consider the effect of sampler size for the waste rock pile testing, if a conventional 300 lb hammer was used, with the correct NALPT sample size, the correction factor to be applied to the raw LPT blow counts would be 1.38. To account for some energy inefficiency, the 1.38 correction has been arbitrarily reduced to 1.2. Previously, a correction factor of 2.0 was used. The net effect of using a correction factor of 0.60 (vs 0.94) in the tailings deposits and 1.2 (vs 2.0) in the waste rock is to significantly reduce the $N_{1,60}$ values calculated for both deposits. These corrected values are presented on the Borehole Information Sheets in the Site Characterization Report (Wood, 2019) and used in geotechnical assessments of both the tailings and waste dump deposits.

<u>References</u>

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Appendix B
Summary of Available Information from Provided InSAR Data

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Memo

To:	VE52705D		
From:	Karen Hincks, MSc, PGeo	**Reviewer:**	Brian Ross, PEng (AB)
cc:	Ed McRoberts, PhD, PEng	**Wood File No.:**	VE52705D.100.3
Date:	26 August 2019		
Re:	**Summary of Available Information from Provided InSAR Data**		

1.0 Introduction

1.1 Scope of Work

TRE ALTAMIRA Inc (TRE) was contracted by Tetra Tech Canada (Tetra Tech) to use Interferometric synthetic-aperture radar (InSAR) techniques to perform an analysis of ground deformation over the Clinton Creek Mine. TRE provided a report to Tetra Tech in November 2018 (TRE, 2018) summarizing the work that was completed, including the methodologies and a preliminary summary of the data, which the Yukon Government (YT) then provided to Wood to provide additional data for the Geological & Geotechnical Site Characterization and Model (Wood, 2019). In addition to the report, TRE has made the Clinton Creek data available to Wood on their online tool TREmaps™, a webGIS web platform which enables the visualization of all measurement points identified and their respective displacement velocity and time-series.

The scope of work for this memorandum involved reviewing the available data and using the TREmaps™ tool to obtain settlement and/or movement information at select locations based on the collected and processed InSAR data. The data presented in the TREmaps™ tool is either 1-D data (ie the location was only identified on either the ascending or descending orbit but was not detected by both orbits) or 2-D data (locations that could be identified on both the ascending and descending orbits). In general, the 1-D data provides an indication of movement toward a satellite or away from a satellite only, and cannot be resolved into horizontal and vertical movements, while the 2-D data can be resolved into interpreted vertical and horizontal movements. Satellite data used in the analysis was acquired between October 2014 and September 2018.

1.2 Methodology

The interpretations presented in this memorandum are based on data provided on the TREmaps™ tool. A high-level screening of areas that have been identified as moving is presented in this memorandum. Following the high-level screening, some additional analysis of the provided data is summarized in this memorandum focusing on areas of specific interest, in particular near boreholes where high ice content permafrost was identified in the drilling program, and areas where there are indications of the highest movement rates identified as a part of the high-level screening. The TREmaps tool allows for the creation of plots of movement at distinct points, an average over a defined area, or the creation of a cross-section across a series of data.

Select screenshots and commentary are provided in Appendix A. Screenshots include plan view images and plots of settlement or horizontal movements over time at select locations. A brief summary of the available information is also presented in Appendix A.

It should be noted that the ascending and descending data points represent an area approximately 8 by 12 m in size, so while a point a shown, that movement could be anywhere within that 8 by 12 m area (8 m in the north-south direction and 12 m in the east-west direction). The grid coverage for the 2-D interpretations is a 100 by 100 m grid, so the point that is shown could represent an average over that 100 by 100 m grid.

1.3 Outline of Memorandum

The data presented in this memorandum is presented in the following order:

- Clinton Creek Waste Dump
- Wolverine Tailings Dumps
- Porcupine Pit and Porcupine Creek Waste Dump

2.0 Clinton Creek Waste Dump

Plan views and select data plots for the Clinton Creek Waste Dump are presented in Appendix A, pages 6-14. General commentary on the provided figures is provided below.

BH18-03 (page 8): This location has about 8 m ice rich permafrost covered by about 29 m of waste rock. The InSAR settlement rate around this borehole over the period between 2014 and 2018 is about 38 mm/yr., with total settlement of 149 mm. The horizontal movement rate is about 8 mm/yr. to the west with a total movement of 34 mm.

Profile South of Spillway (page 13): The InSAR Profile indicates over the period between 2014 and 2018 the settlement rates were approximately 57 mm/yr. at the west end to 40 mm/yr. at the east end of the profile.

Waste Dumps at Hudgeon Lake Shoreline (page 9): Within the grid represented by the point on page 9, the InSAR settlement rate over the period between 2014 and 2018 is approximately 53 mm/yr. with a total settlement of 209 mm. The horizontal movement rate is about 17 mm/yr. with a total of 71 mm of movement towards the lake.

East of 16-BH12 & BH18-01 and West of BH18-02 (page 10): The settlement rate over the 2014 to 2018 period is 47mm/yr. with a total settlement of 183 mm and 21 mm/yr. rate and a total of 79 mm of horizontal movement towards the lake. There was no ice rich permafrost reported in BH18-01 or BH18-02. The cause of this settlement does not correlate with ice rich permafrost and suggests that there are other settlement mechanisms. As there are slope indicators at these two locations, future data may assist in interpretation.

Near BH18-04 (page 11): At this location on the south original valley wall there is about 6 m of ice rich permafrost underlying 33 m of waste rock and mixed colluvium. The settlement rate over the period between 2014 and 2018 is 27 mm/yr. with a total of 106 mm of settlement and a movement of only 5 mm/yr westward, with a total movement of 26 mm over the period.

3.0 Wolverine Tailings Dump

Plan views and select data plots for the Wolverine Tailings Dump are presented in Appendix A, pages 15-21. General commentary on the provided figures is provided below.

South Lobe (page 19 and 20): The south lobe is moving at about 24 mm/yr. over the period between 2014 and 2018, with a total movement of 97 mm, with the direction assumed to be eastward. Due to signal shading the horizontal and vertical movements cannot be determined.

North Lobe (page 17 and 18): The north lobe is moving at about 36 mm/yr. eastward (with a total of 135 mm of movement) with 13 mm/yr. settlement (and total settlement of 47 mm) over the period 2014 to 2018.

North Face of North Lobe (page 21): The north side of the north lobe appears to be moving toward the northeast. The data indicates movements of 20 mm/yr. (78 mm total) of settlement and 5 mm/yr. (19 mm total) of movement eastward over the period 2014 to 2018. It should be noted that northward movements cannot be detected by InSAR.

Generally, it appears that there is more movement of the piles in the lower regions of the slope (page 18 and 20). This is may be due to the "survival" of more prevalent and icier permafrost in the cooler valley bottom.

In addition to these specific areas with the higher deformations, the typical movement rates over much of the tailings is 25-30 mm/yr. in areas on the lower slope with apparently little to no ice rich permafrost or permafrost. The overall movement rates in the upper portion of the pile are around 10 mm/yr., with most of the nodes in the upper pile indicating less than 25 mm of total movement over the time interval.

4.0 Porcupine Pit and Porcupine Creek Waste Dump

Plan views and select data plots for the Porcupine Creek Waste Dump are presented in Appendix A, pages 22-29. General commentary on the provided figures is provided below.

Porcupine Creek (pages 25, 26 and 27): Near the 2 ice rich holes in Porcupine Creek colluvium, the settlement rate is in the order of 50 mm/yr. over the 4-year period between 2014 and 2018, with an average settlement of about 170 mm. There was essentially no horizontal movement over the period of data.

Porcupine Pit East wall (page 28): The average settlement along the east wall of the Porcupine Pit is about 9 mm/yr. (total of 36 mm) with about 6 mm/yr. (total of 25 mm) of westward movement (over the period of data between 2014 and 2018 – historical air photos indicate that there may have been a large slope movement here prior to October 2014).

Porcupine Pit West wall (page 29): There is very limited data on the west wall of the Porcupine Pit, on both the ascending and descending data sets. This slope should not be in shadow for both data sets which implies that there may be too much movement on the west wall of the pit to pick up phase shifts on any identifiable zone.

5.0 References

TRE ALTAMIRA. 2018. InSAR Ground Deformation Analysis for the Clinton Creek Mine. November 2018 Report. 16 November 2018. Delivery Reference JO18-614-CA. Issued to Tetra Tech Canada.

Sincerely,

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28 August 2019.

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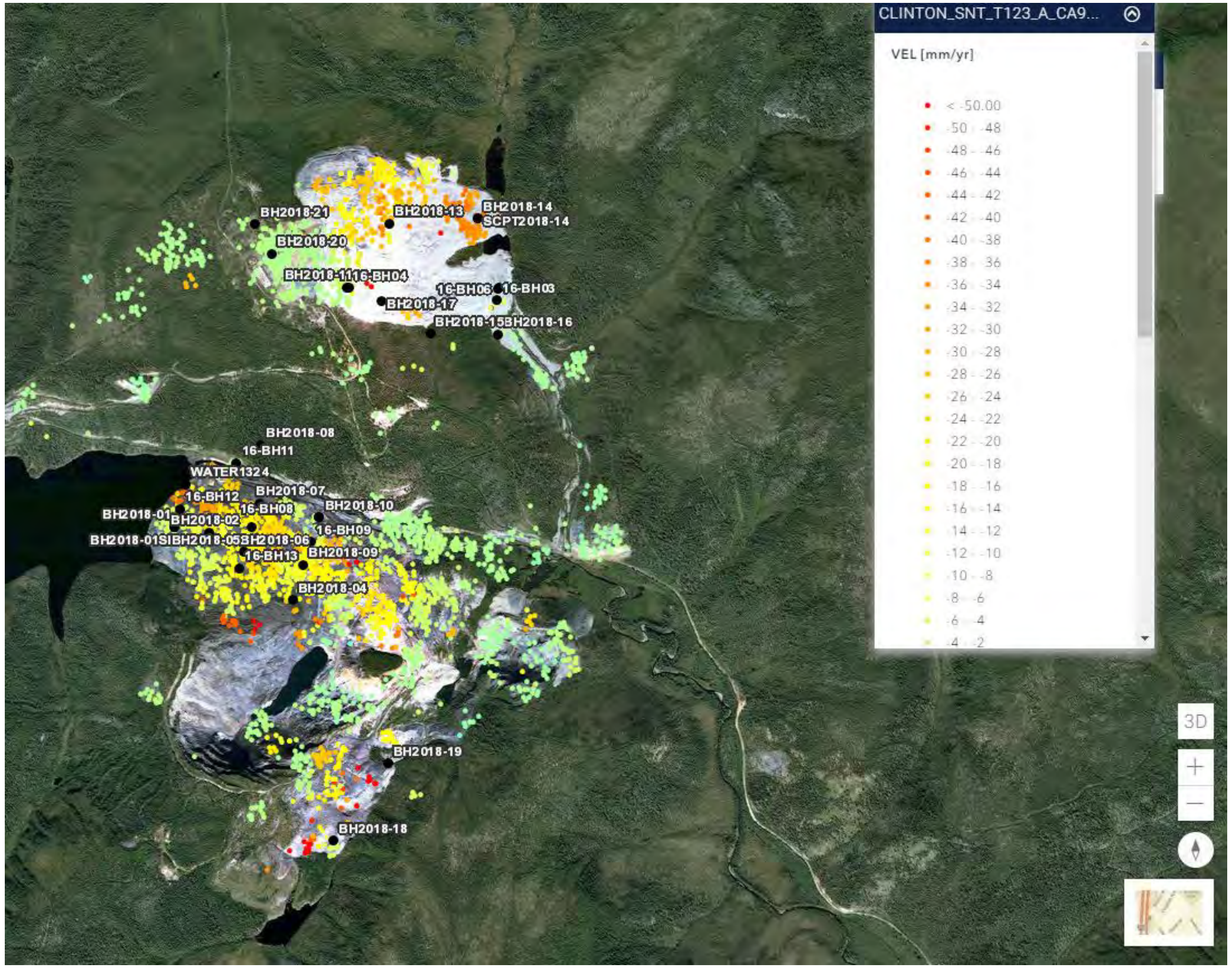
Appendix I
InSAR Interpretations of Clinton Creek

InSAR Interpretations for Clinton Creek

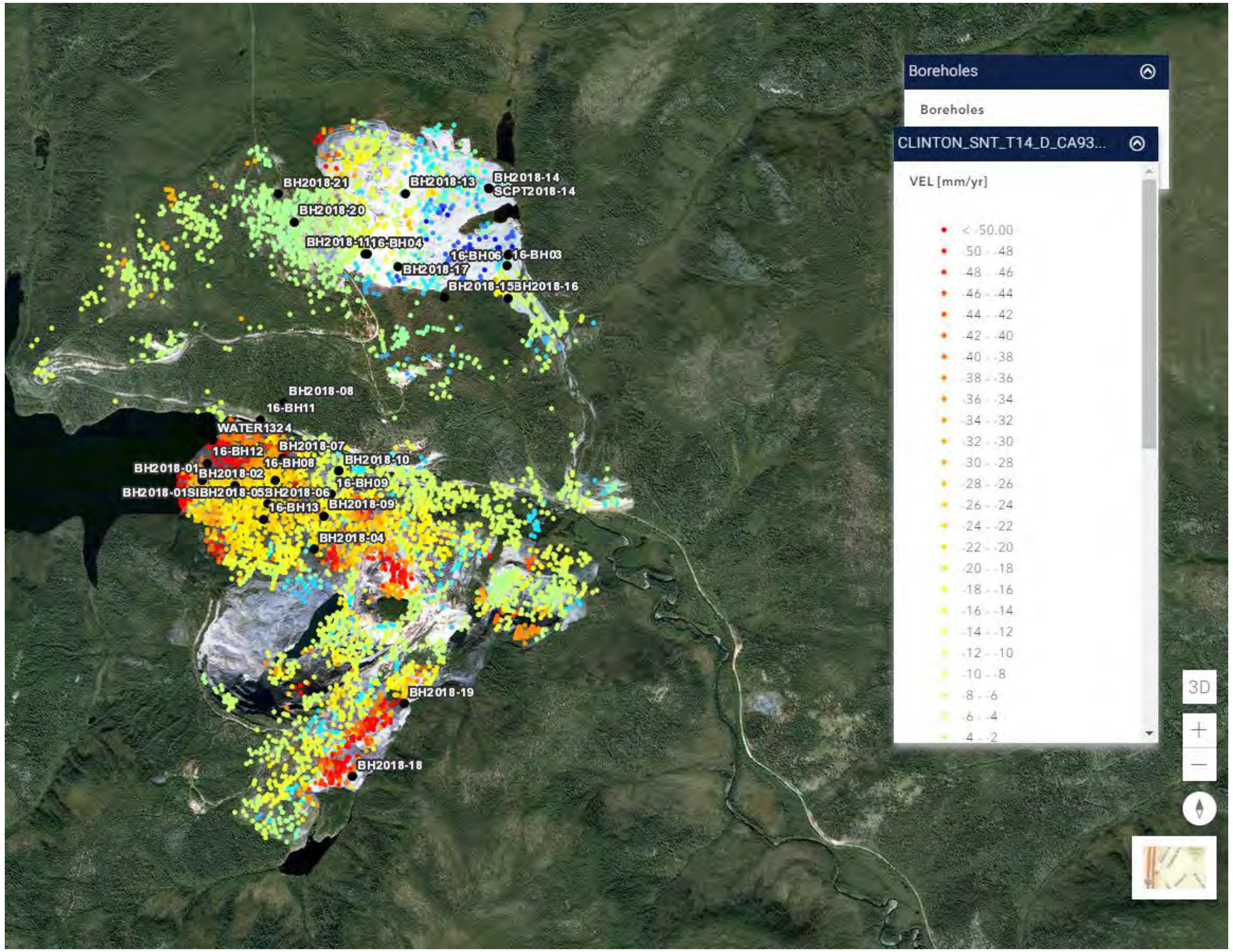
Appendix I to Summary of Available Information from Provided InSAR Data

Overall Site 1-D Data - Ascending



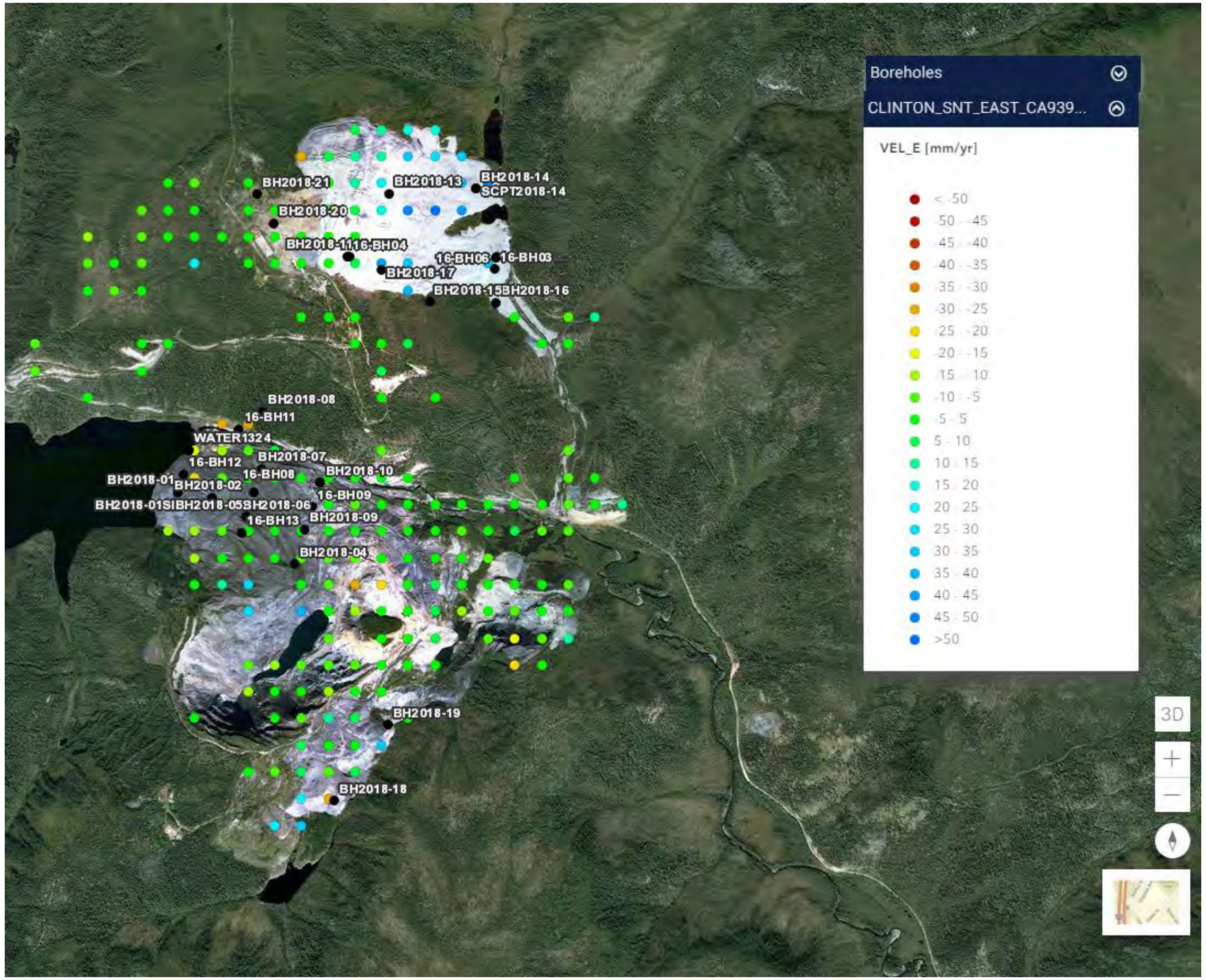
- Movement vectors are TOWARD the satellite, so positive (cold colours) movement is toward the west and/or up while negative (warm colours) movement is toward the east and/or down
- Limited coverage compared to the descending satellite data
- No coverage of the south tailings lobe, likely due to a shadow effect from the steep lobe at the top of the slope
- Limited data on the west slope of Porcupine Pit, either due to rapid movements or shadow effect
- Data collected between 14 October 2014 and 29 September 2018

Overall Site 1-D Data - Descending



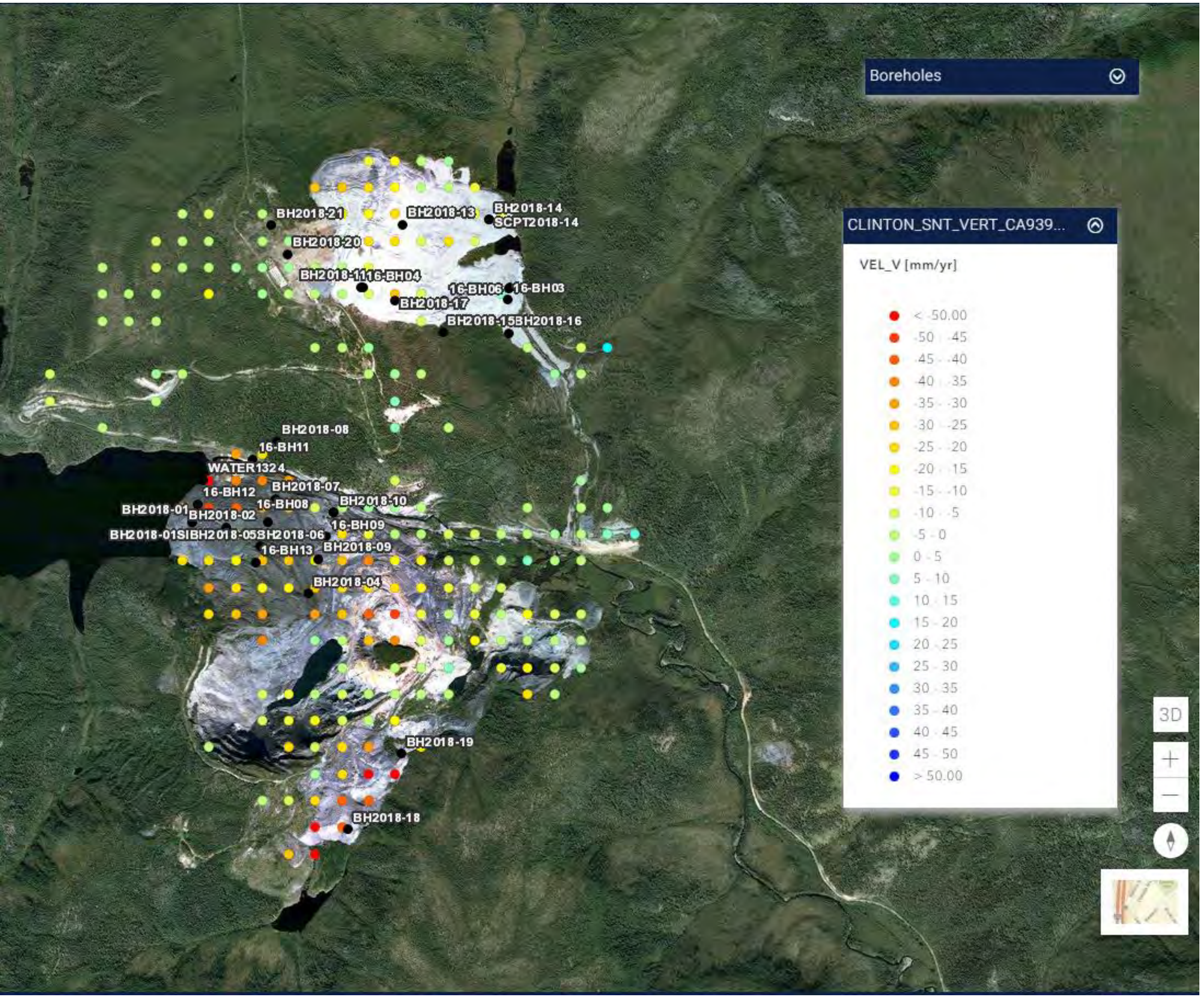
- Movement is TOWARD the satellite, so positive (cold colours) movement is toward the east and/or up while negative (warm colours) movement is toward the west and/or down
- Better overall site coverage, including the south lobe of the tailings pile
- Still no coverage of the west slope of Porcupine Pit, likely due to greater rates of movement than the technology can manage
- Data collected between 30 October 2014 and 27 September 2018

Overall Site 2-D Data – Horizontal Movement



- Requires both ascending and descending data be collected in the pixel, so limited in coverage by the ascending data
- Positive movement (cold colours) is toward the east and negative movement (warm colours) is toward the west.
- Generally eastward movements in the tailings area, the west slope of Porcupine Pit and in Porcupine Creek
- Generally westward movements near Hudgeon Lake and the current spillway, with some north of Porcupine Pit and south of Snowshoe Pit

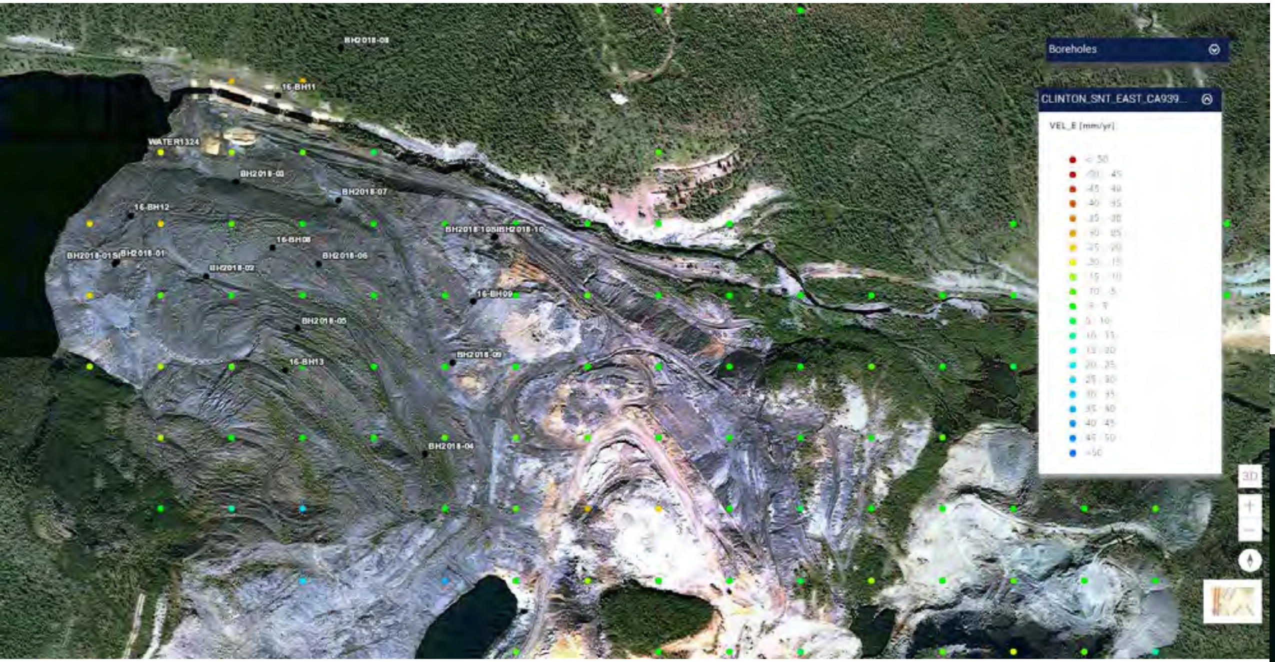
Overall Site 2-D Data – Vertical Movement



- Requires both ascending and descending data be collected in the pixel, so limited in coverage by the ascending data
- Positive movement (cold colours) is upward (heave) and negative movement (warm colours) is downward (settlement).
- Generally there is settlement in waste and tailings
- Greatest settlement rates are adjacent to the spillway and Hudgeon Lake, around Porcupine Pit and in Porcupine Creek

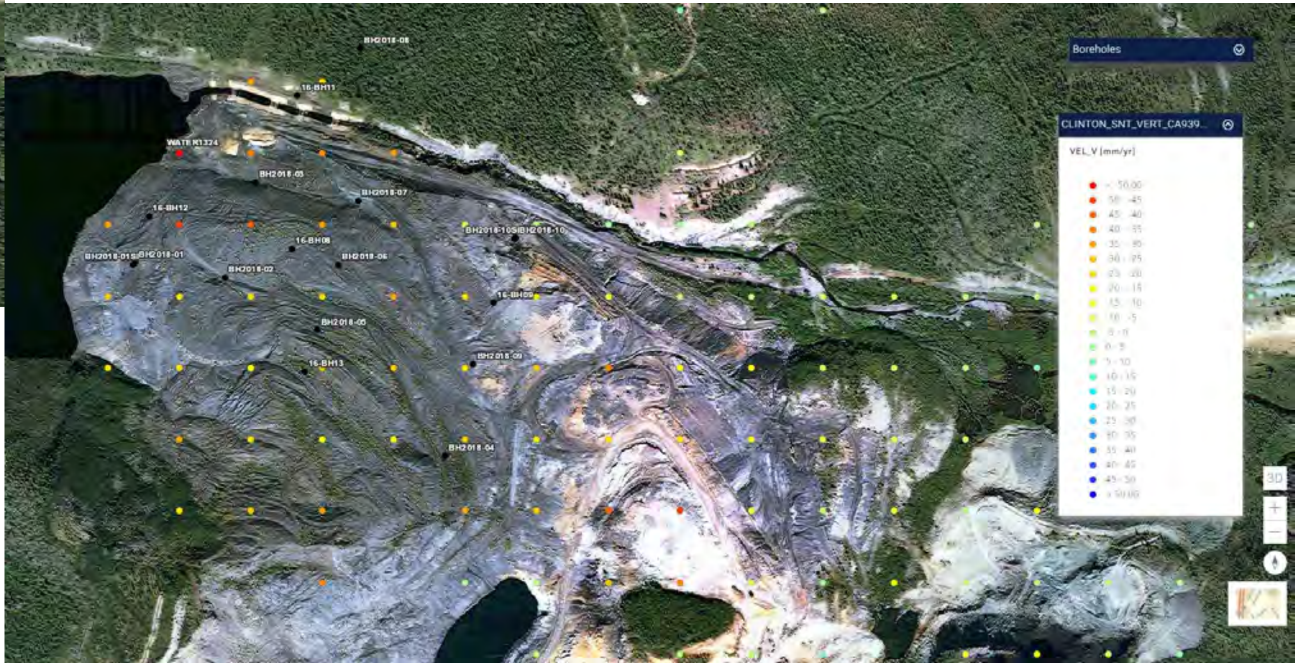
Clinton Creek Waste Dump

Clinton Creek Waste Dump – 2D Data

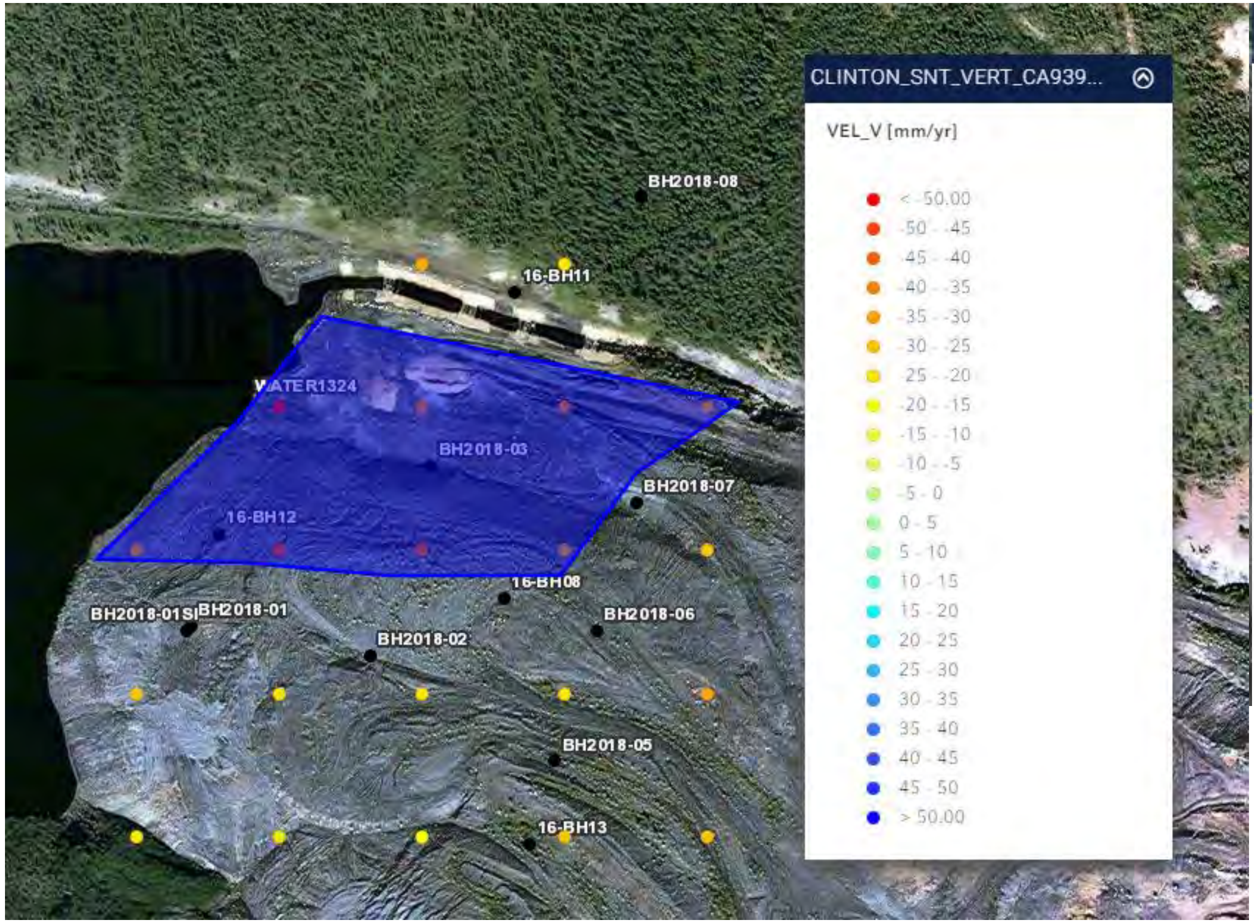


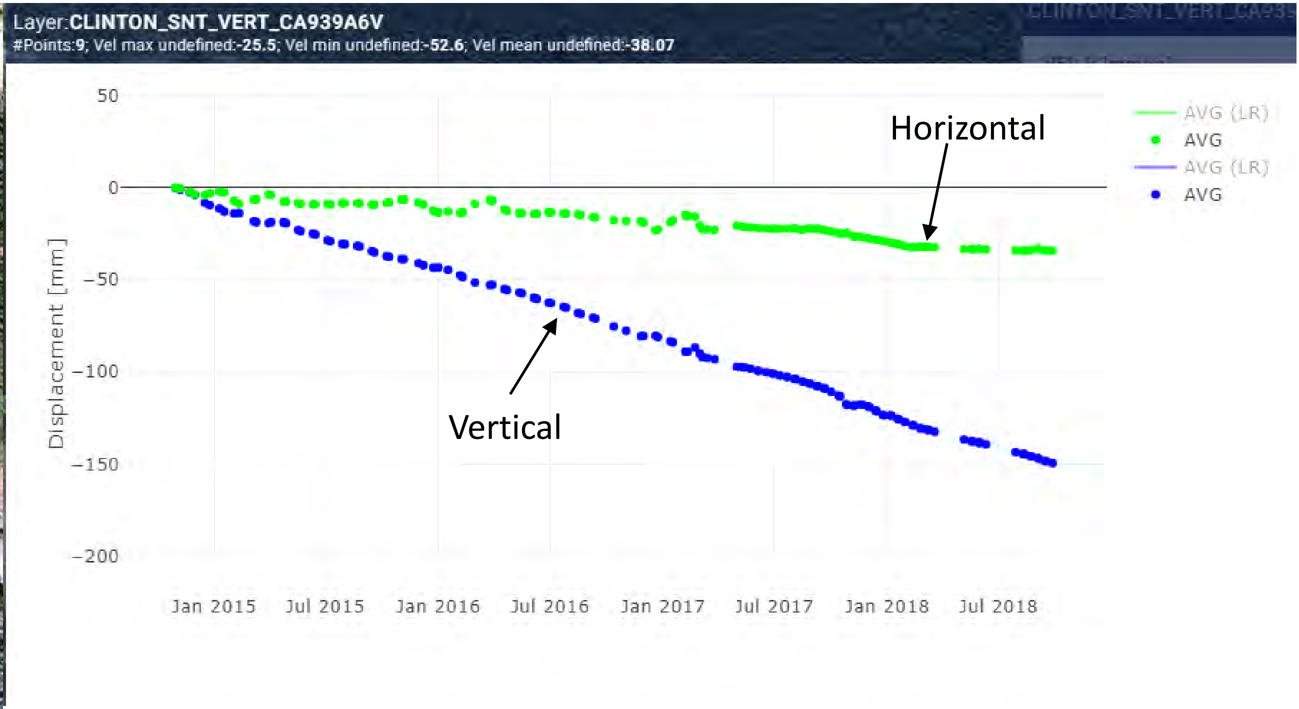
Horizontal Movements

Vertical Movements



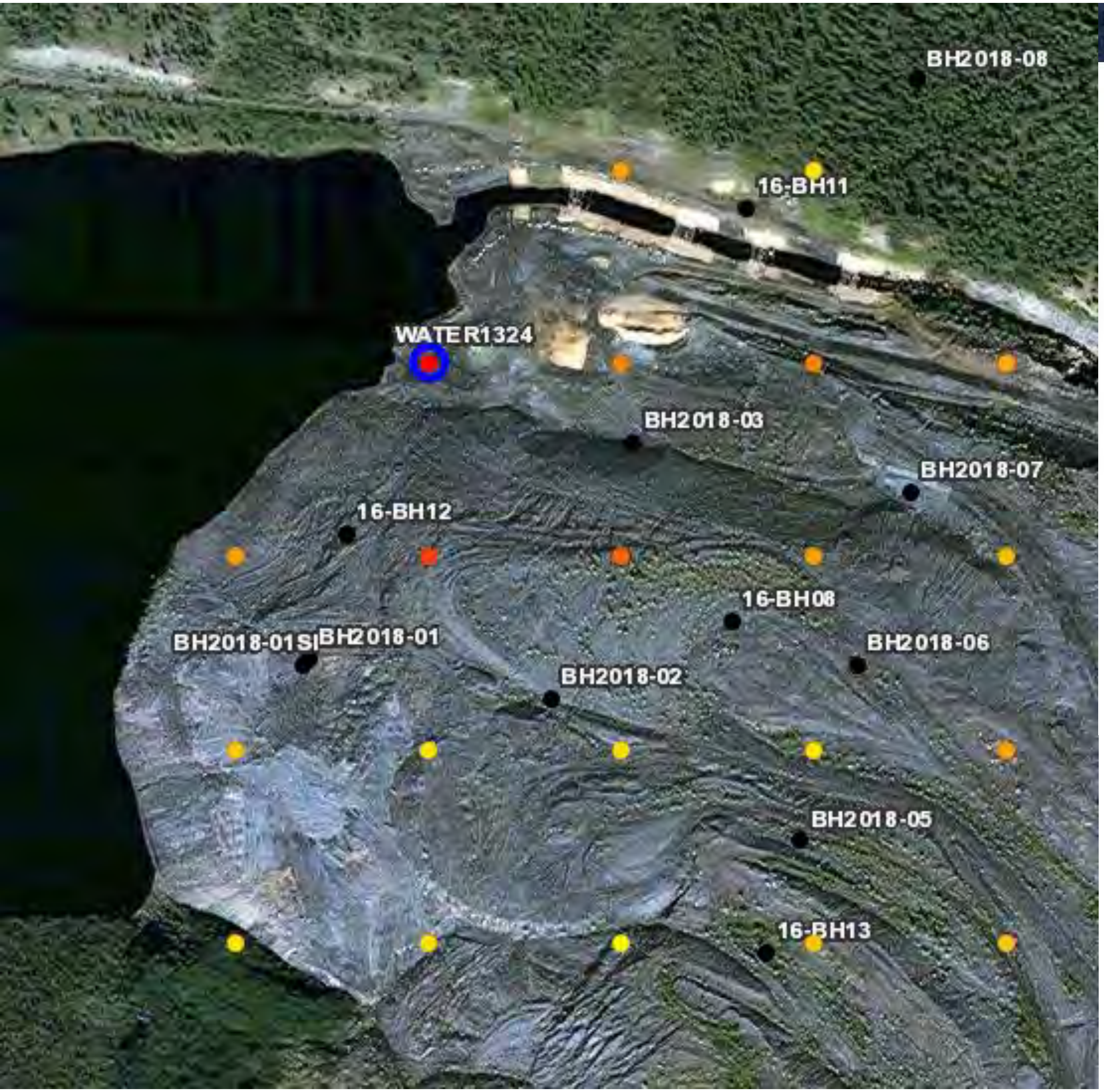
Average Settlement around BH18-03





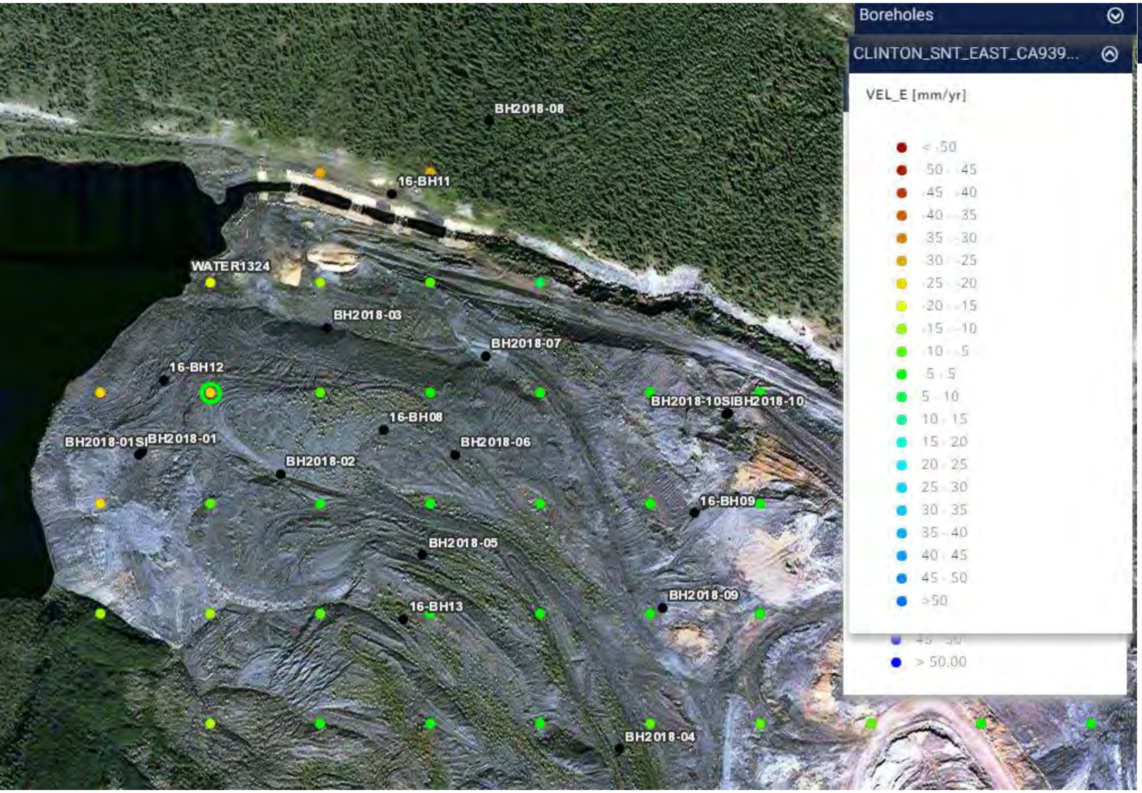
- Average vertical settlement of -149 mm, average rate of -38 mm/year
- Average horizontal movement of -34 mm, average rate of -8 mm/year

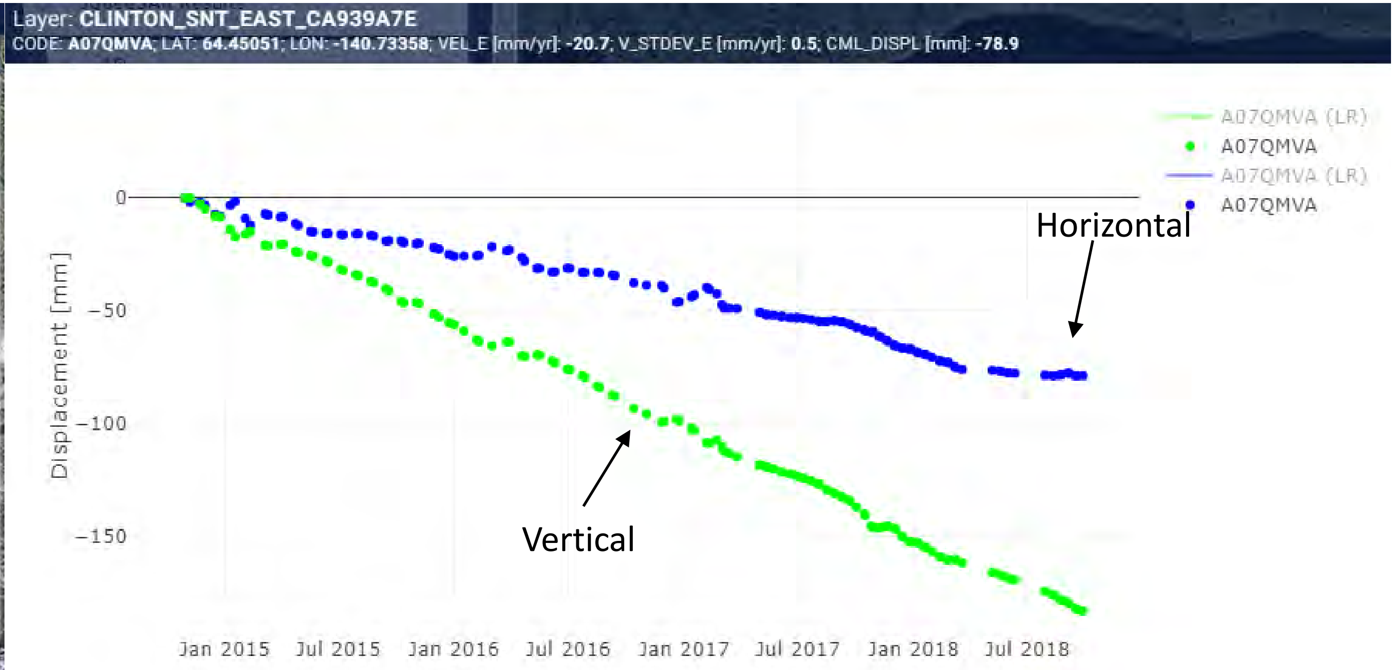
Settlement Near Hudgeon Lake



- Vertical settlement of -209 mm, average rate of -53 mm/year
- Horizontal movement of -71 mm westward, average rate of -17 mm/year

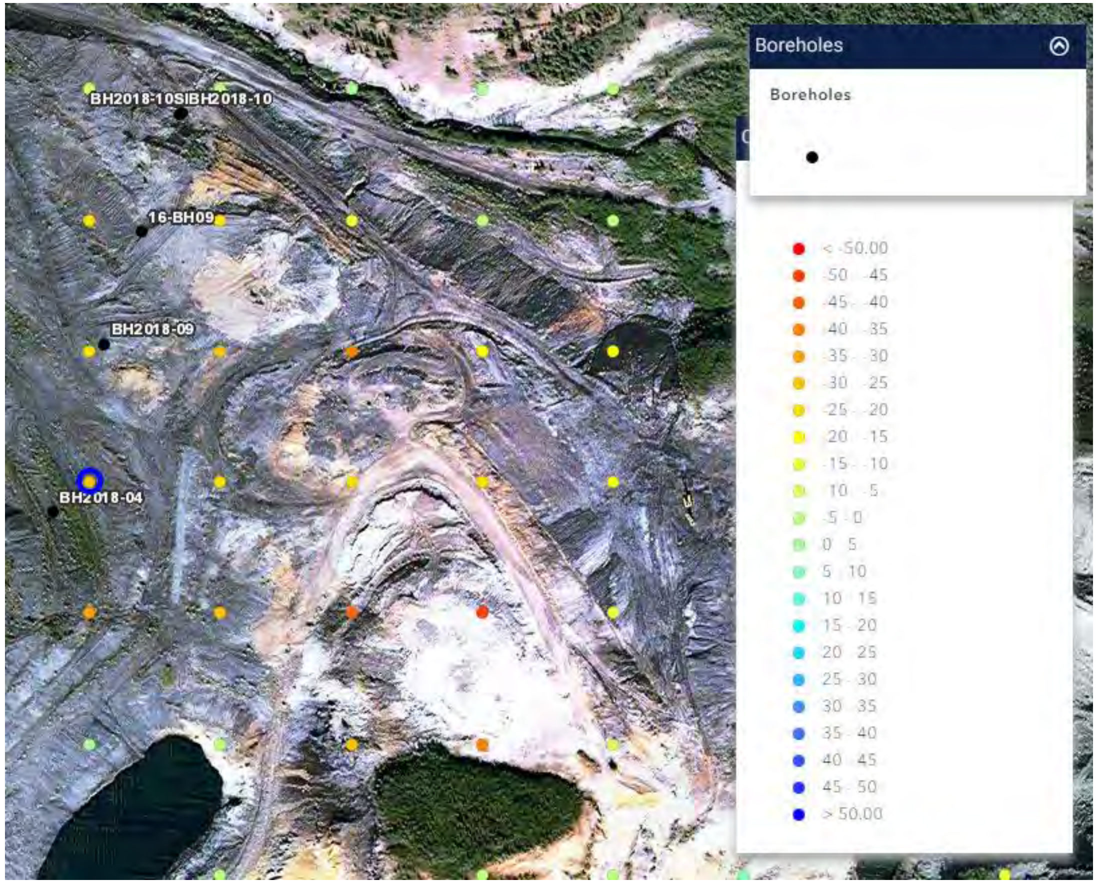
Settlement Near Hudgeon Lake

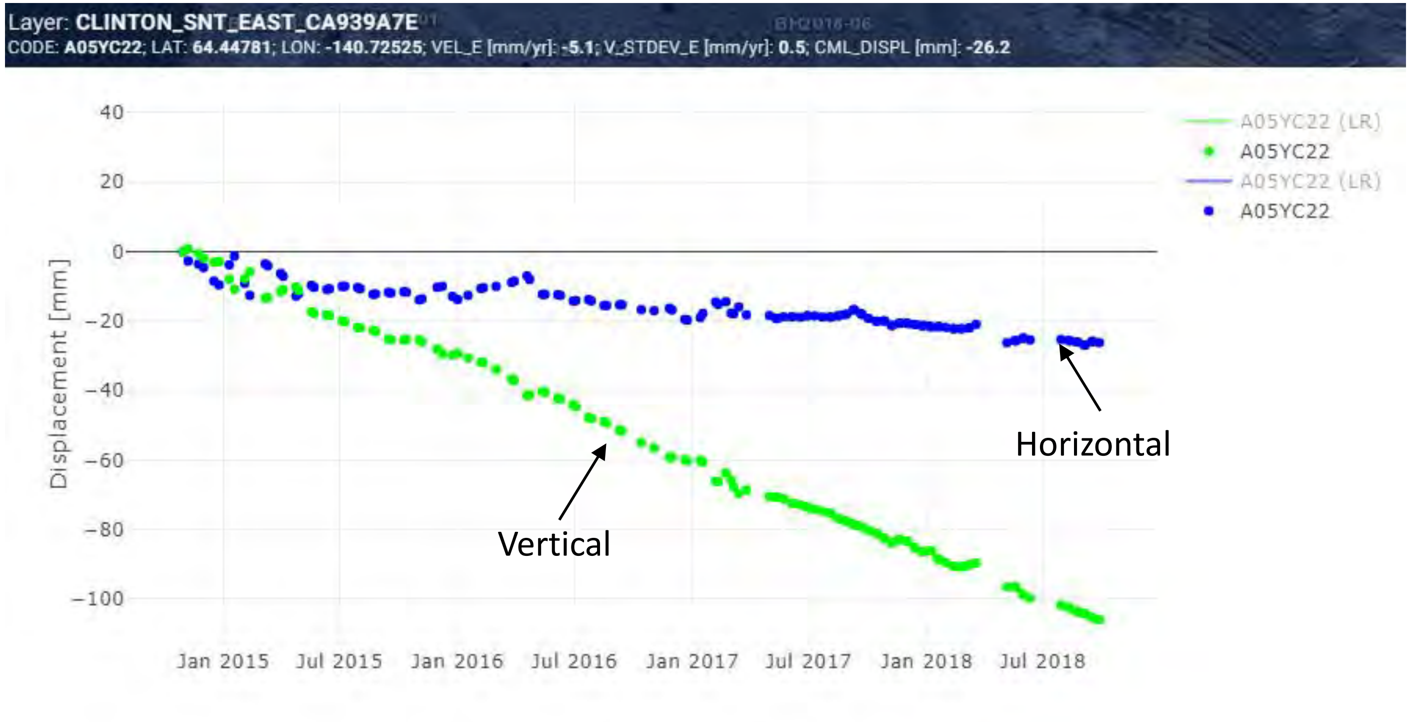




- Vertical settlement of -183 mm, average rate of -47 mm/year
- Horizontal movement of -79 mm westward, average rate of -21 mm/year

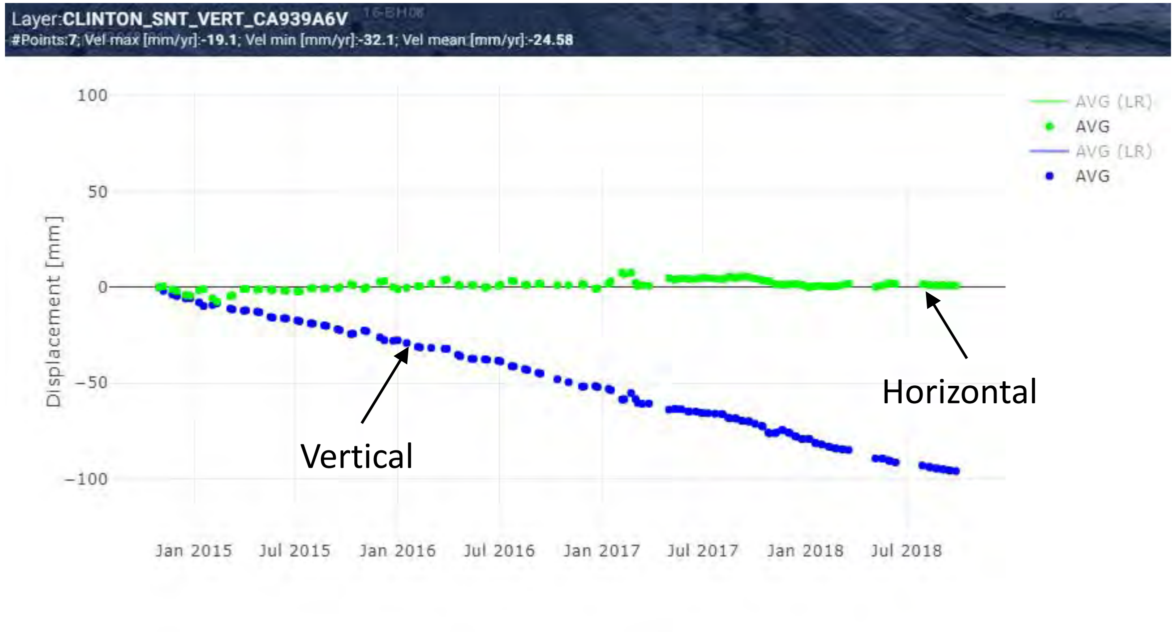
Settlement Near BH18-04





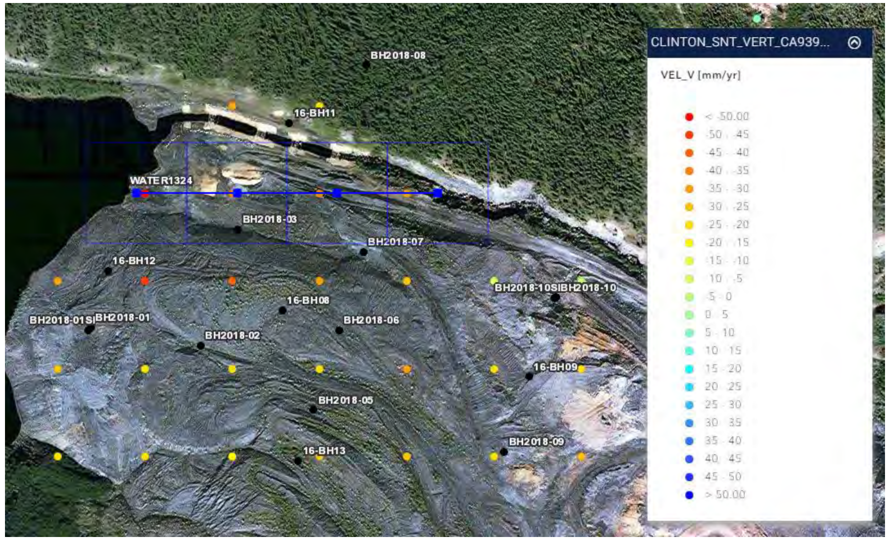
- Vertical settlement of -106 mm, average rate of -27 mm/year
- Horizontal movement of -26 mm westward at a rate of -5.1 mm/year

Settlement Along pre-construction south slope in area with visible cracking

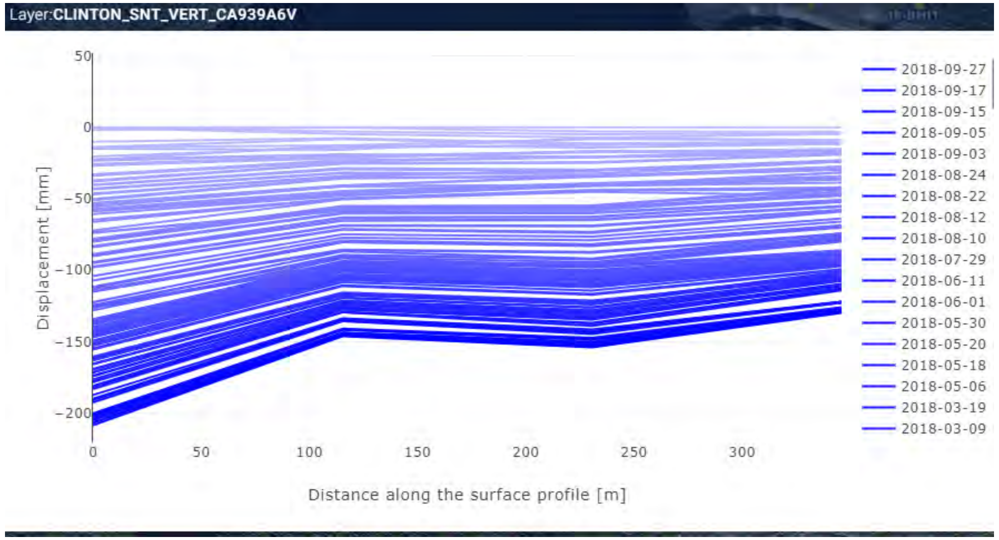


- Average vertical settlement of 96 mm, average rate of -32 mm/year
- Essentially zero horizontal movement
- Possible that ice-rich permafrost encountered in BH18-04 extends along the south slope to the west into this area.

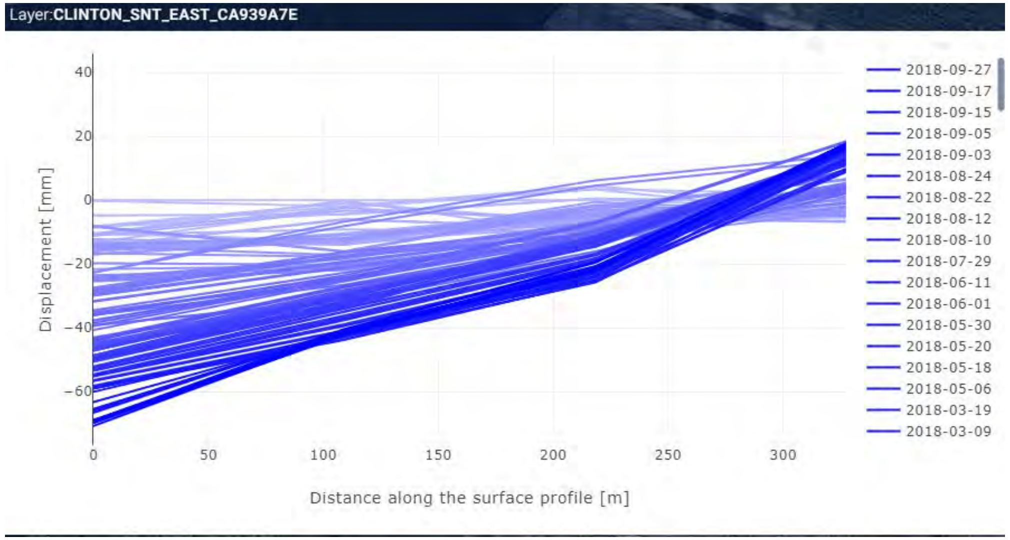
Settlement Profile Along Southside of Spillway



- Greater total vertical settlement closer to Hudgeon Lake
- About 130 mm of total vertical settlement closer to the spillway
- Appears to have a fairly constant settlement rate close to the spillway
- Horizontal movements are toward the west at the west end, with very minor movements to the east at the east end of the section

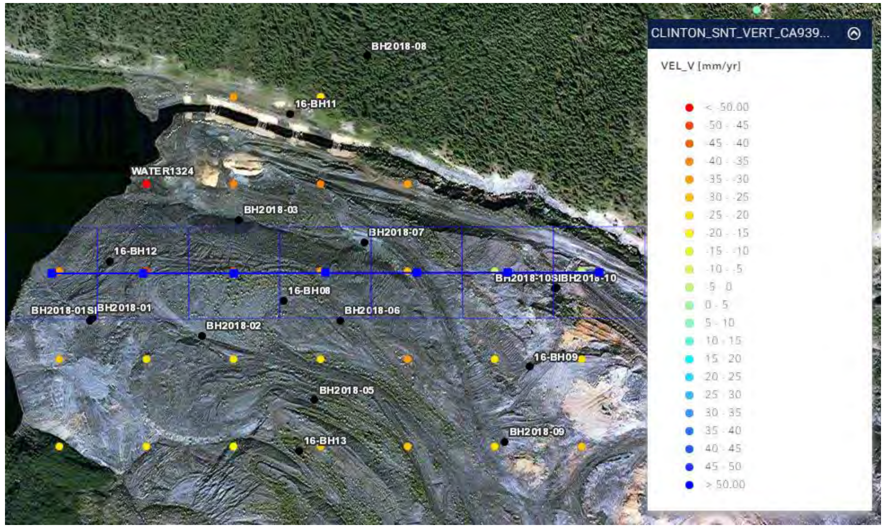


Vertical Settlement

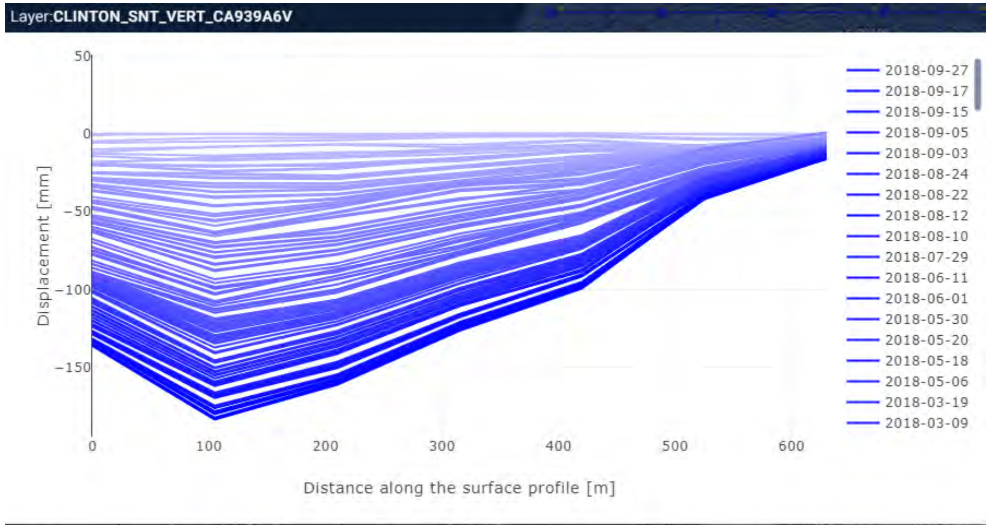


Horizontal Movement

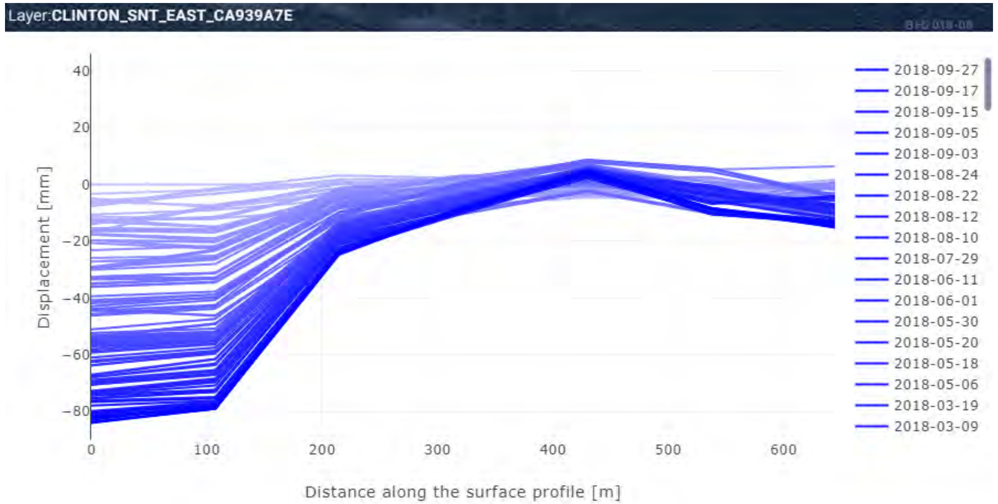
Settlement Profile Along Southside of Spillway



- Greater total vertical settlement closer to Hudgeon Lake with minimal settlement at the east end of the section
- Horizontal movements are greatest closest to Hudgeon Lake, with movement toward the east between 79 and 84 mm, with less than 20 mm of settlement 300 m back from Hudgeon Lake



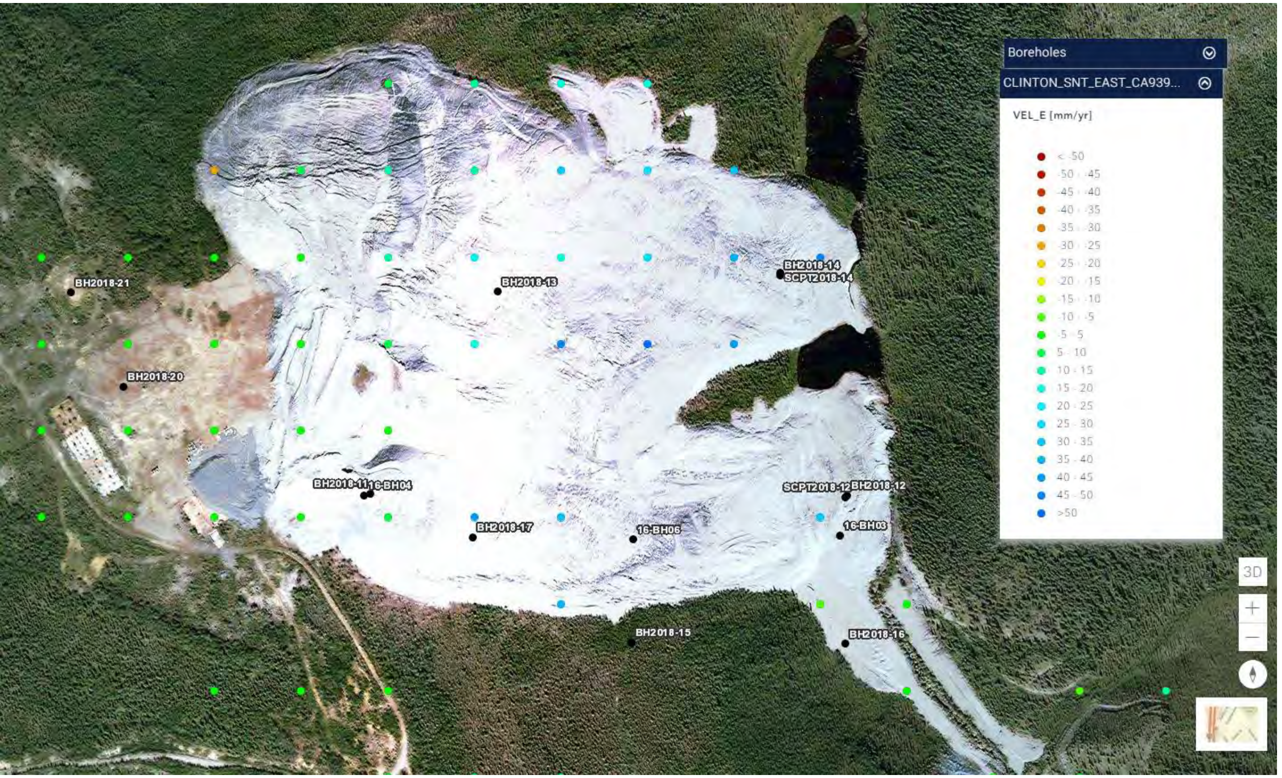
Vertical Settlement



Horizontal Movement

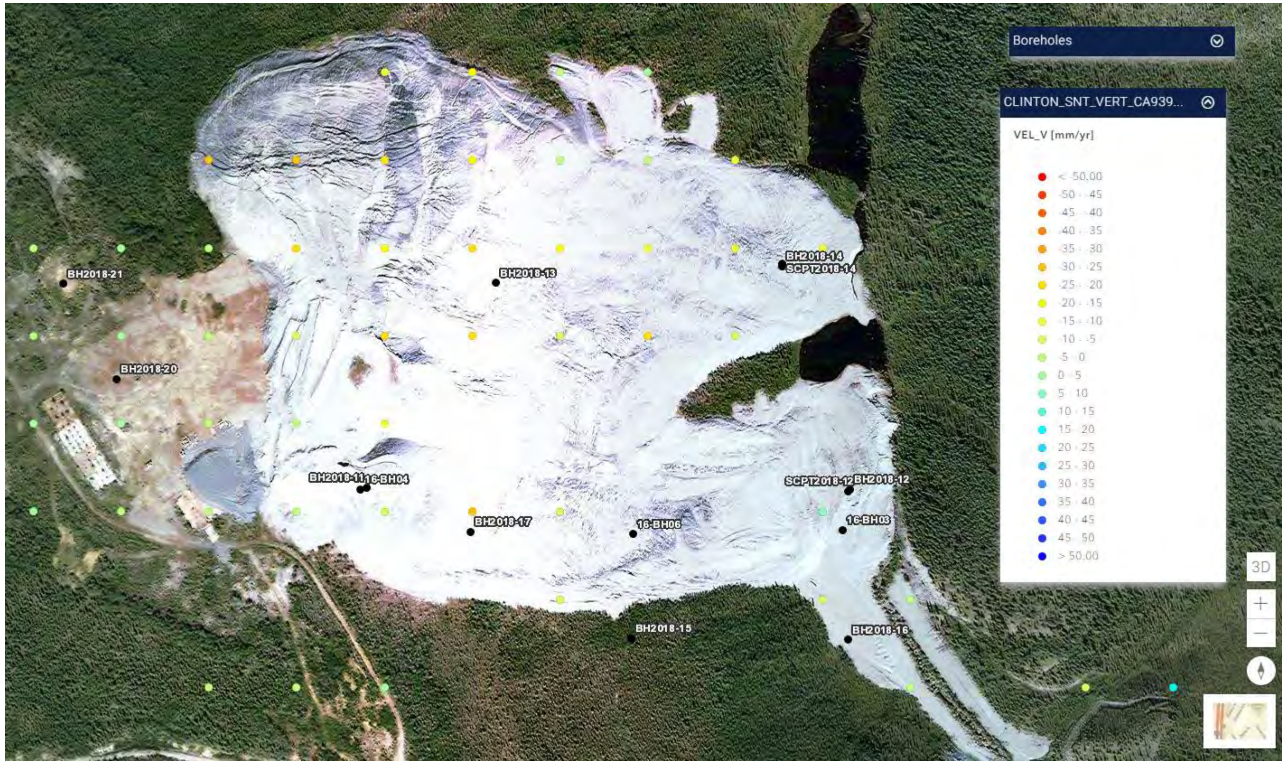
Wolverine Tailings Dump

Tailings Dump – 2D Data



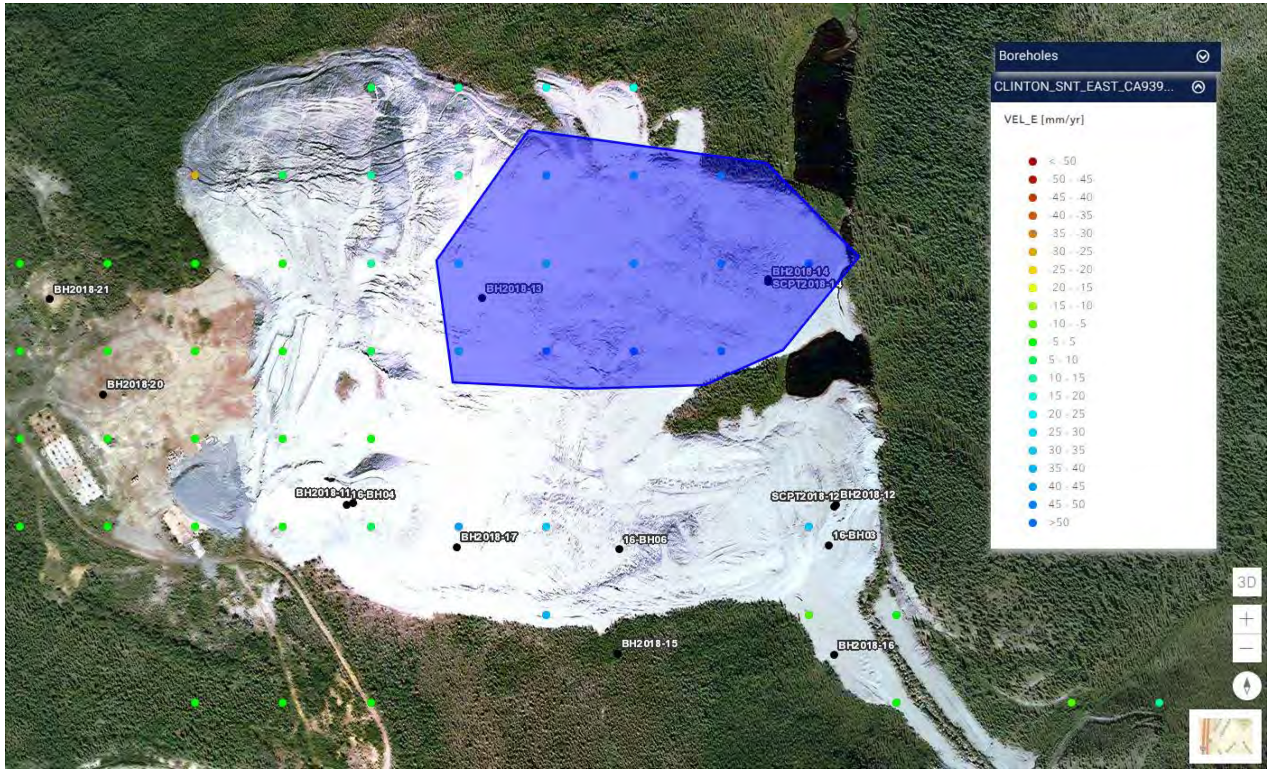
Horizontal Movements

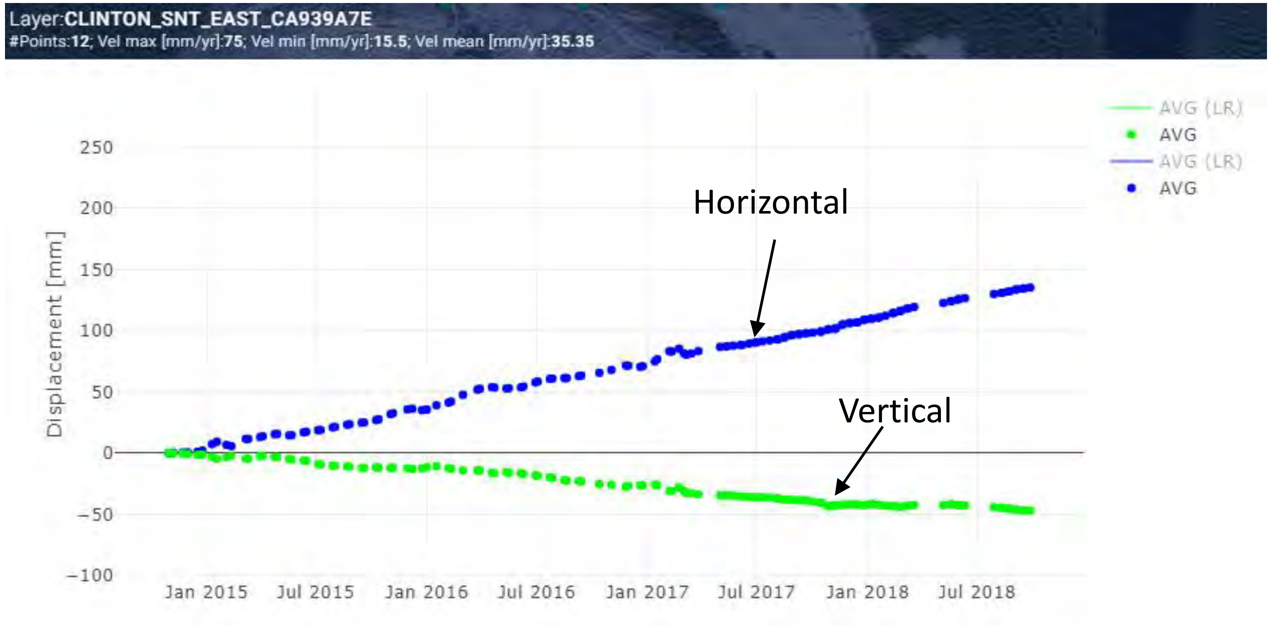
Vertical Movements



Note: almost no 2-D coverage of the south lobe, due to either slope shadow effects or movement rates greater the phase shift can detect.

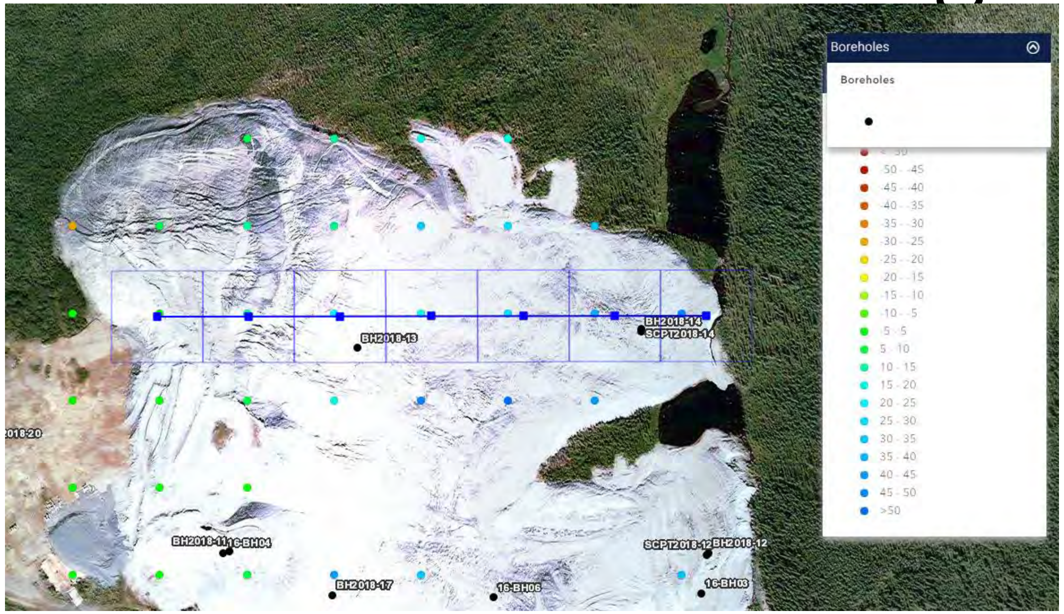
Average Movement of the North Lobe – Middle and Lower Slope



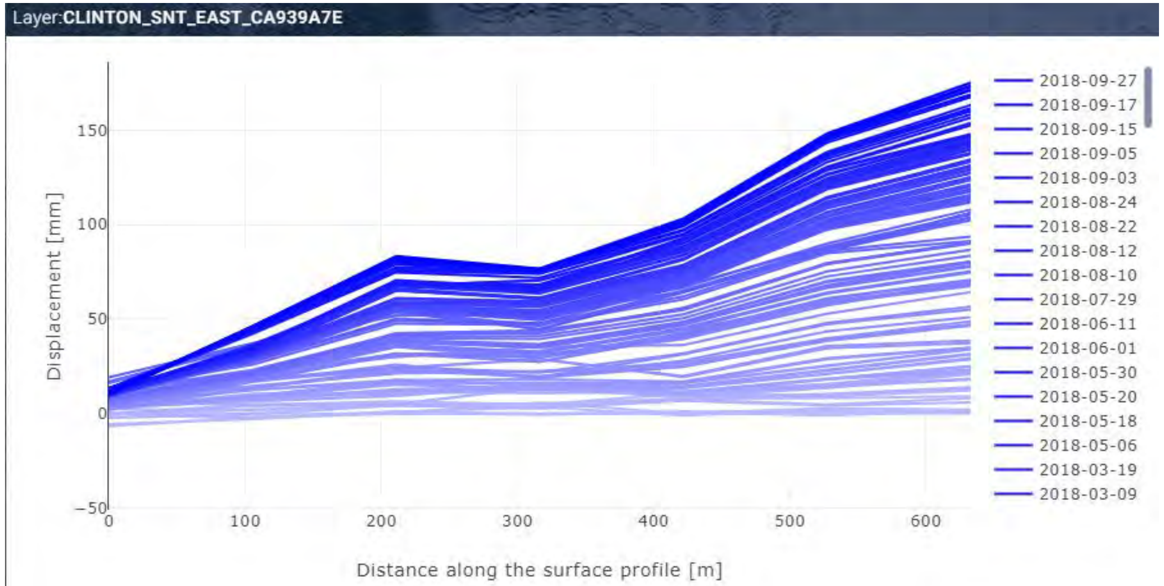


- Vertical settlement of -47 mm, average rate of 13 mm/year
- Horizontal movement of 135 mm eastward, average rate of 35.5 mm/year
- Overall movement downslope to the east

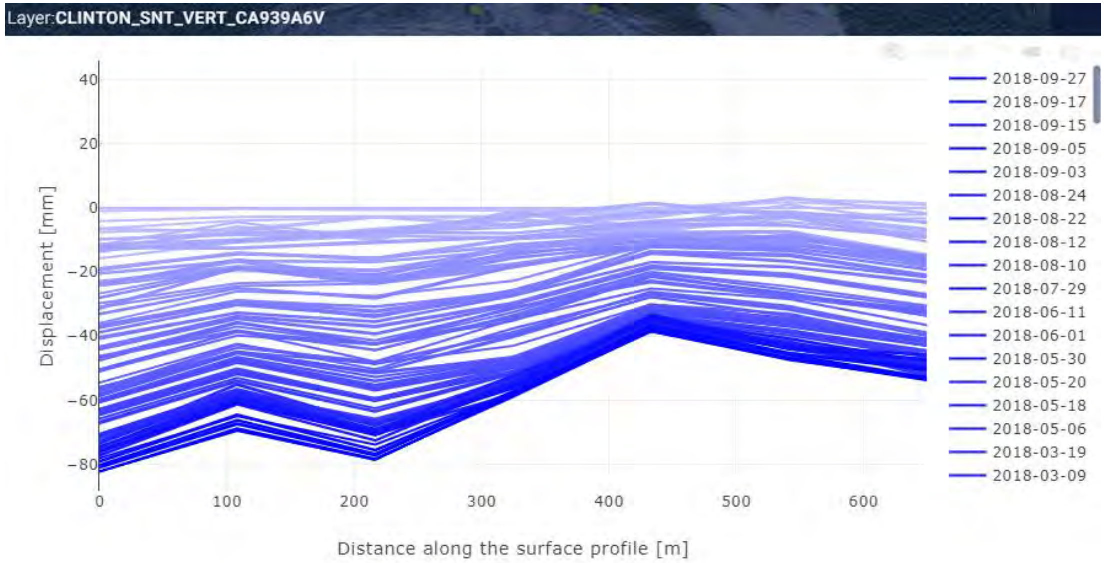
Movement Along the North Lobe



- Greatest magnitude of vertical movement is at the top of the slope, with 82 mm maximum settlement near the crest
- Eastward movement magnitude increases as you move downslope, with 175 mm of eastward movement near the toe

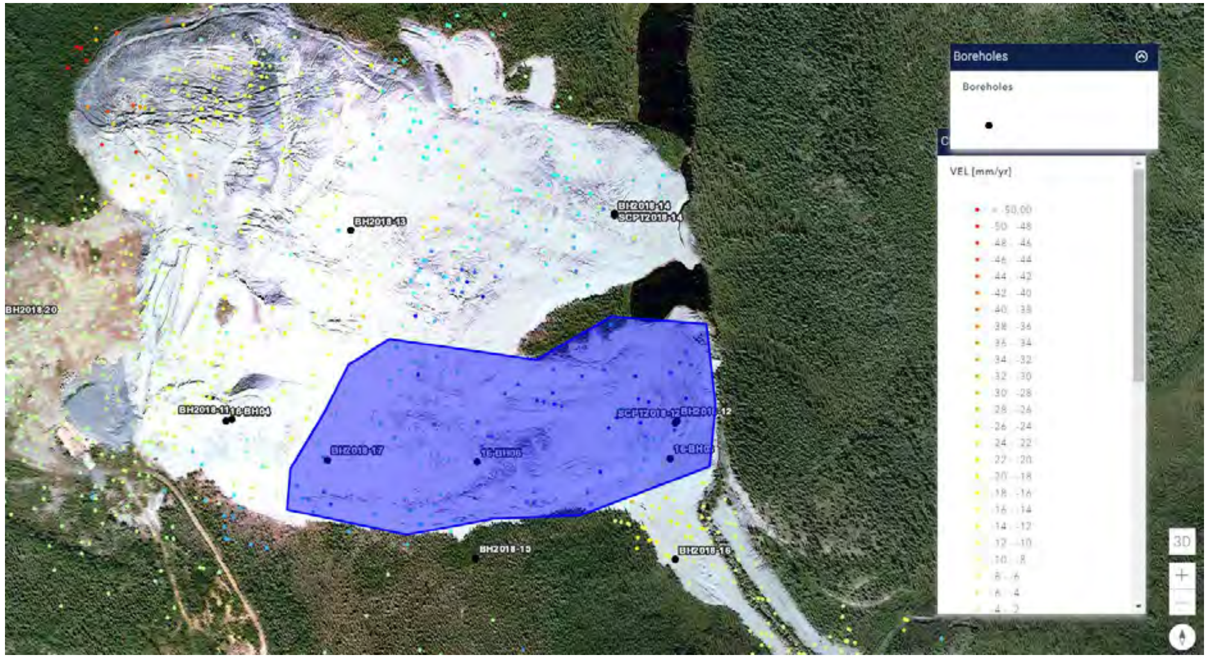


Horizontal Movement



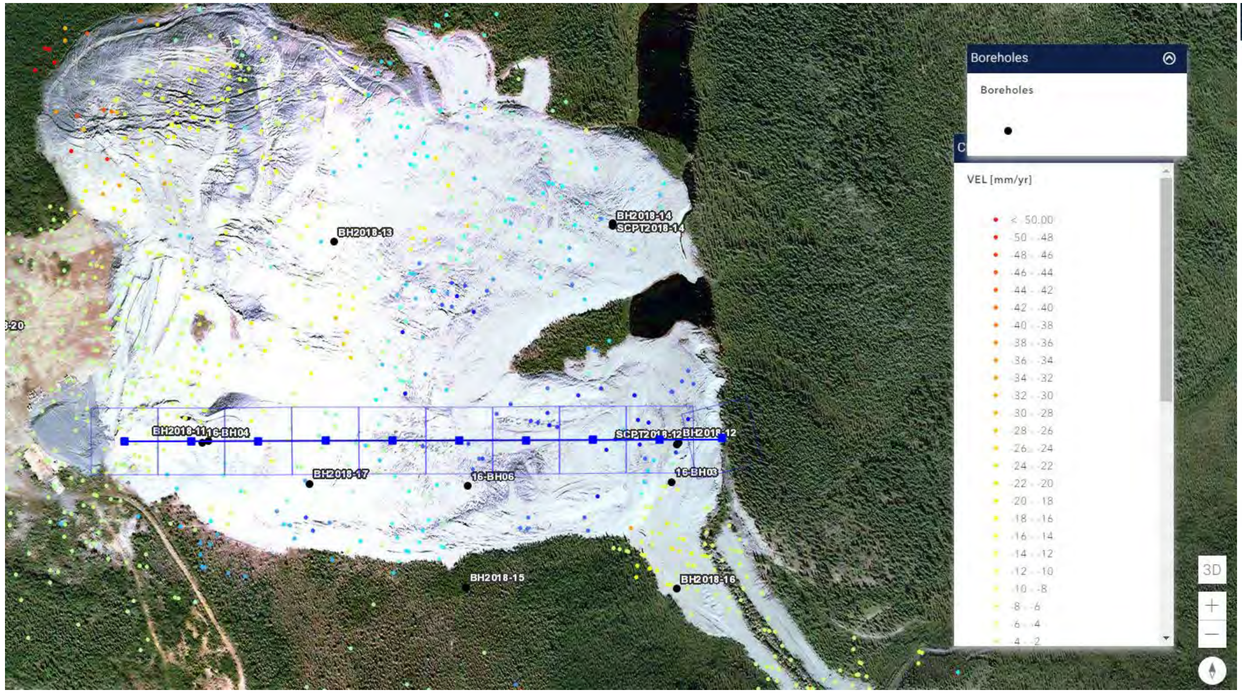
Vertical Settlement

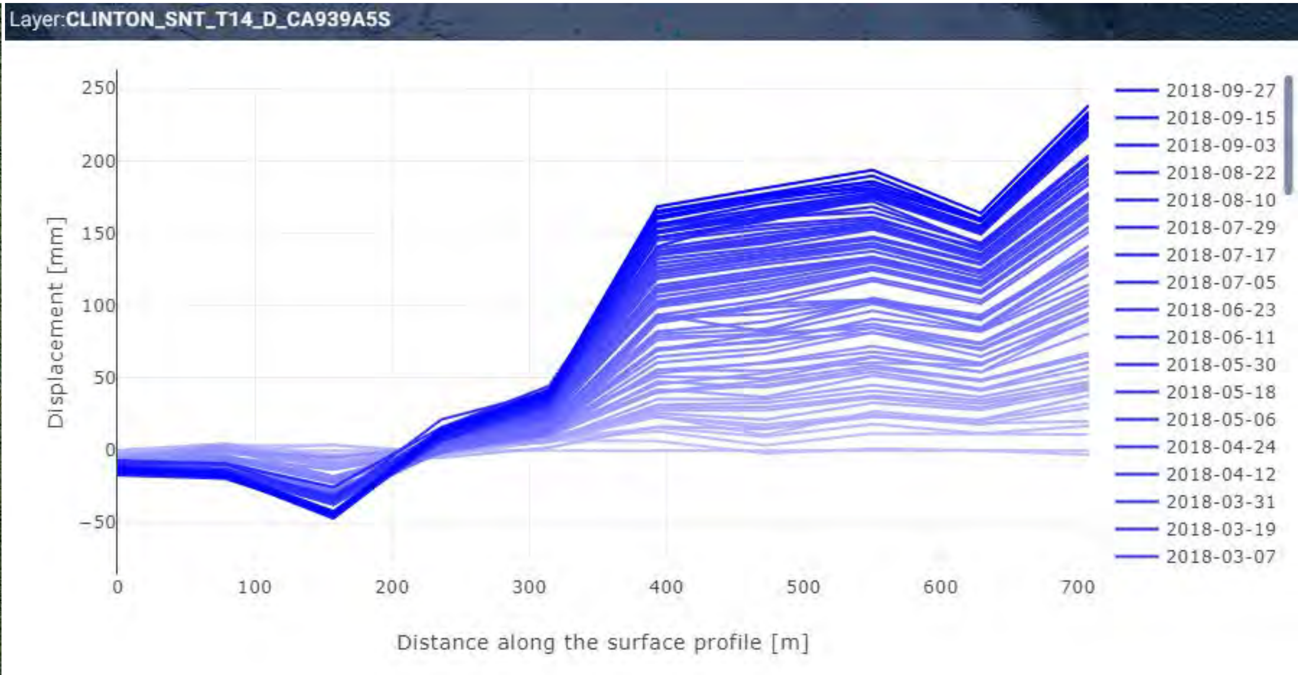
South Lobe – 1-D Descending Data Only



- Positive movement is toward the satellite, so toward the east (but unlikely upward)
- Cumulative 97 mm of movement, average movement rate of 23.6 mm/year

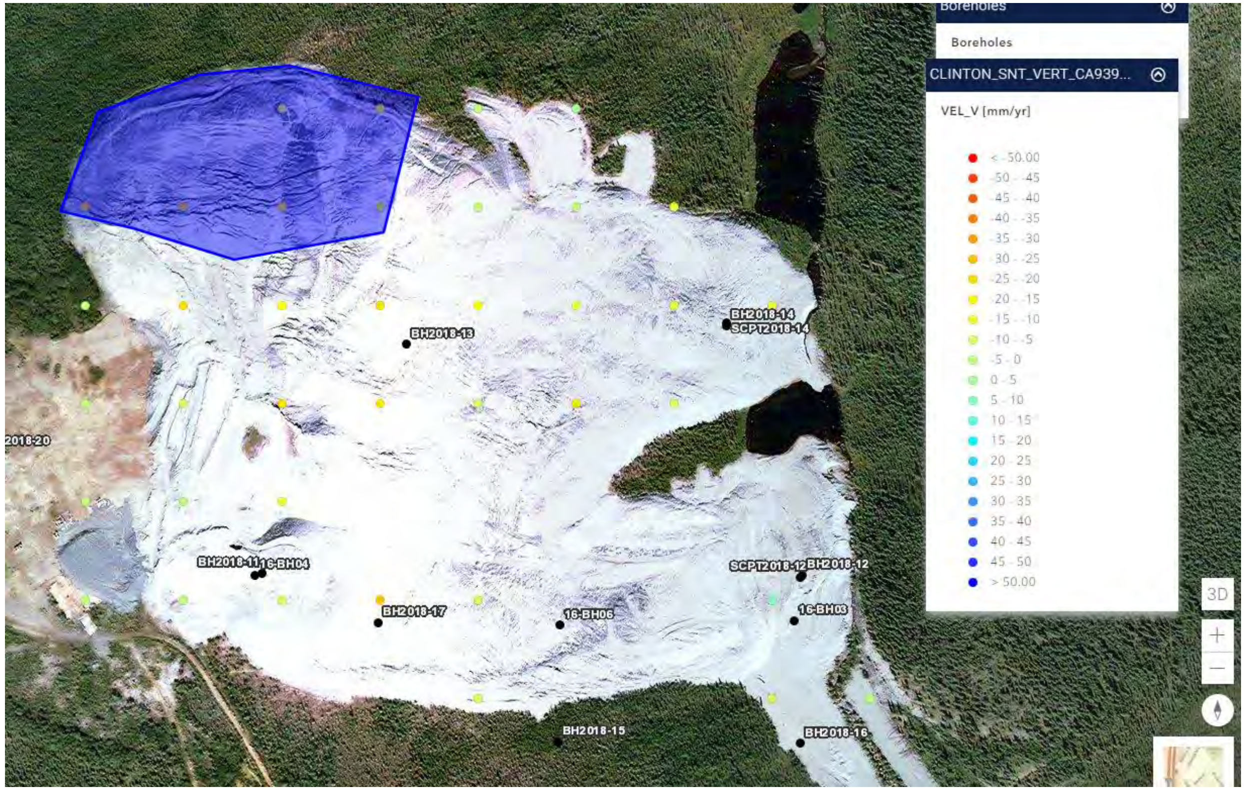
South Lobe – 1-D Descending Data Only

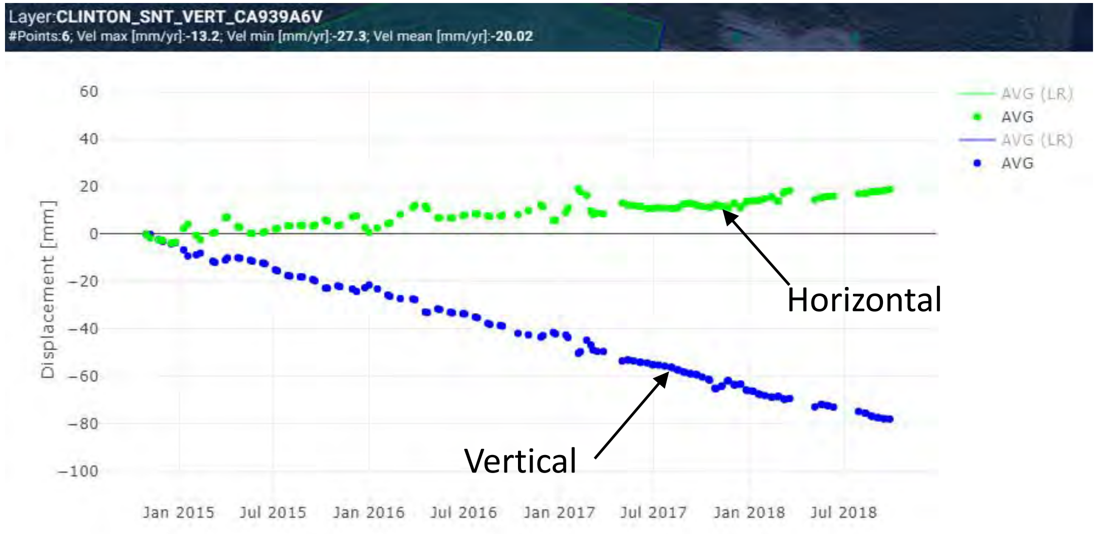




- Positive movement is toward the satellite, so toward the east and upward
- Maximum movements are detected near the toe of the lobe
- Maximum detected movement of 238 mm at the toe

Average Movement of the North Lobe – North Facing Slope

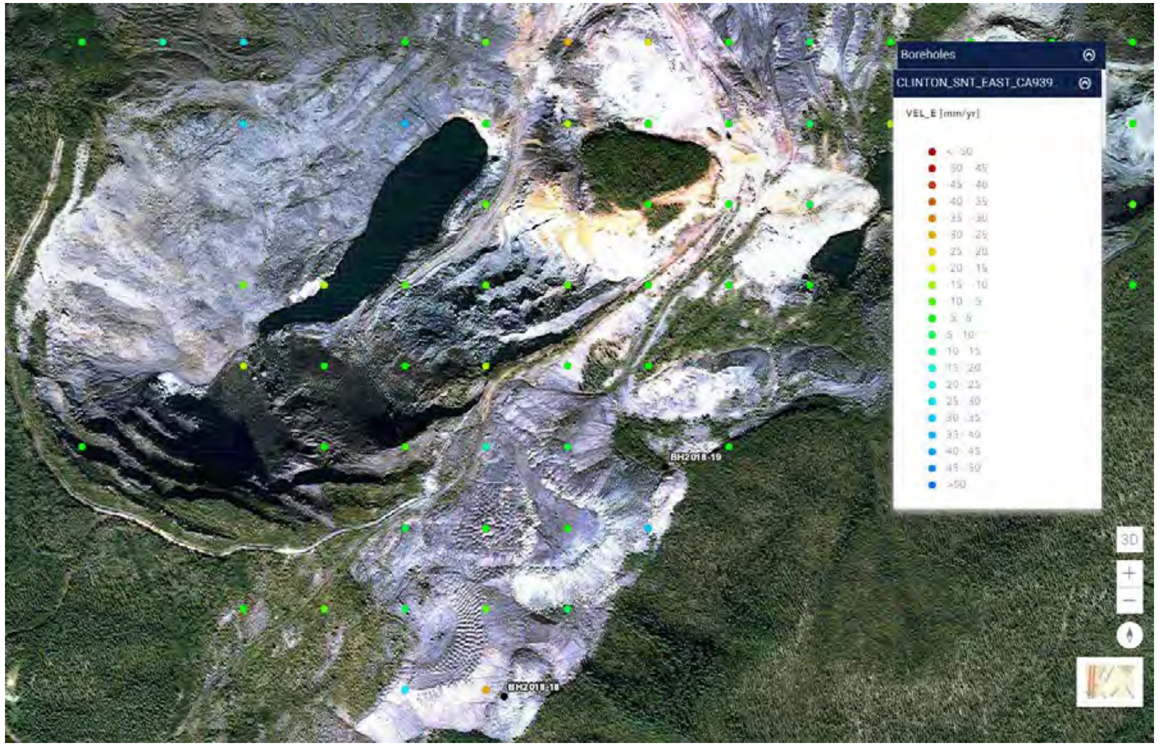




- Vertical settlement of -78 mm, average rate of 20 mm/year
- Horizontal movement of 19 mm eastward, average rate of 5 mm/year
- Likely that data is limited due to the ascending and descending data being limited to generally east-west movement, and cannot detect north-south movement

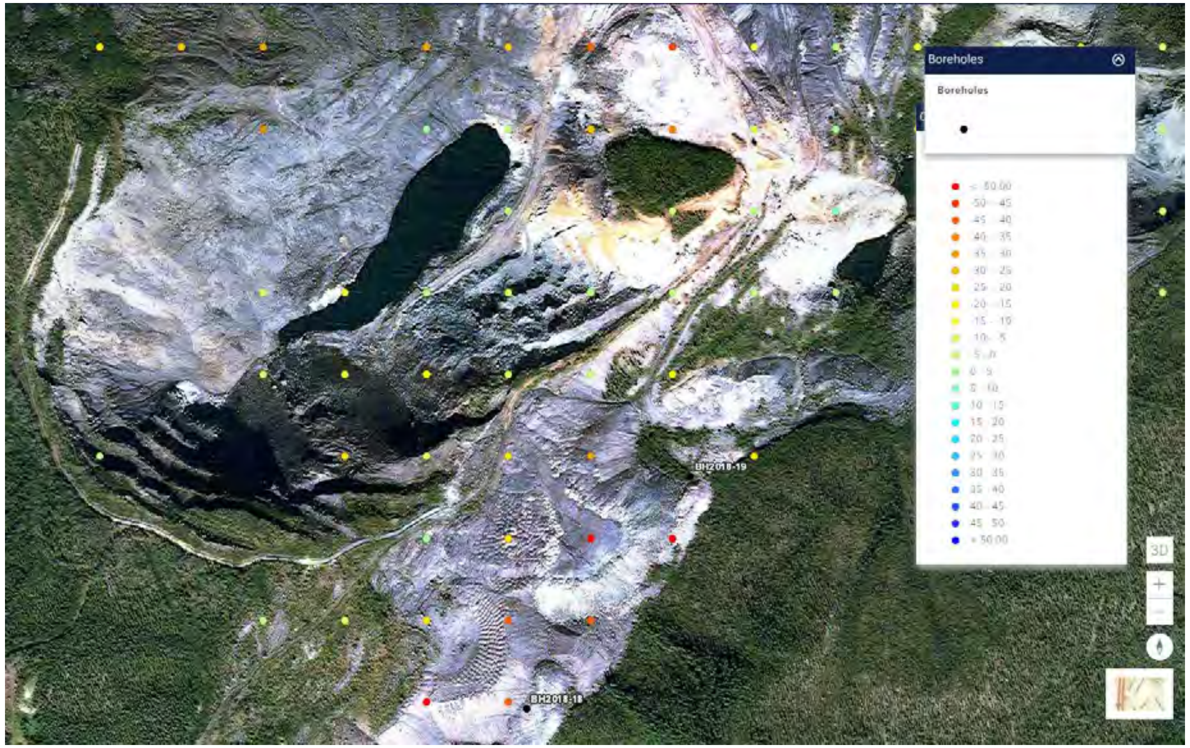
Porcupine Pit and Porcupine Waste Dump

Porcupine Pit and Porcupine Creek Waste Dump – 2D Data



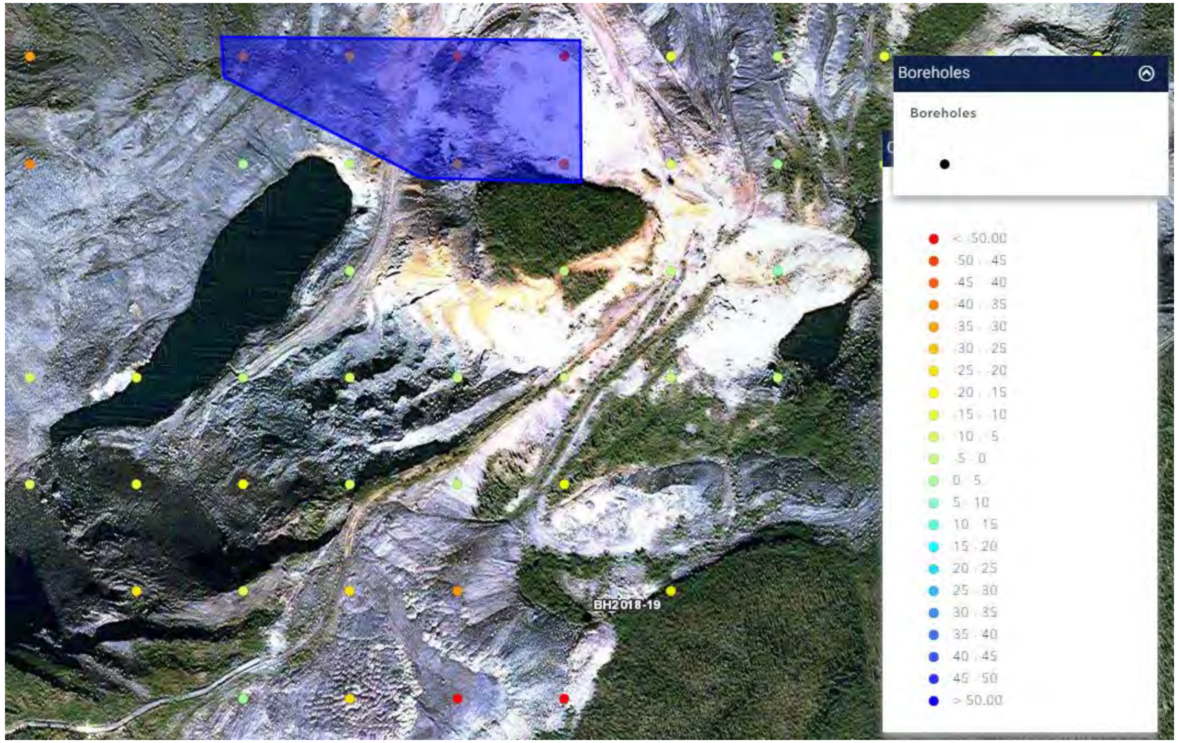
Horizontal Movements

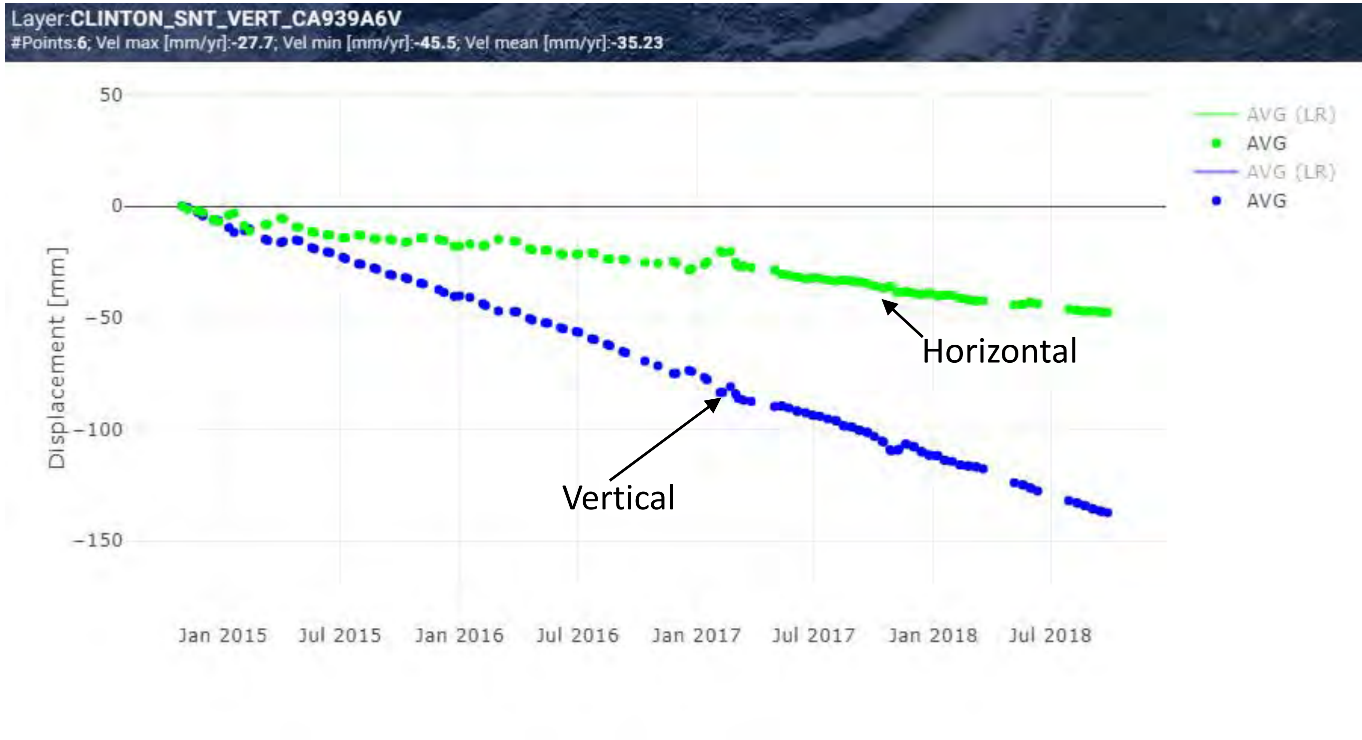
Vertical Movements



Note: almost no 2-D coverage of the south lobe, due to either slope shadow effects or movement rates greater the phase shift can detect.

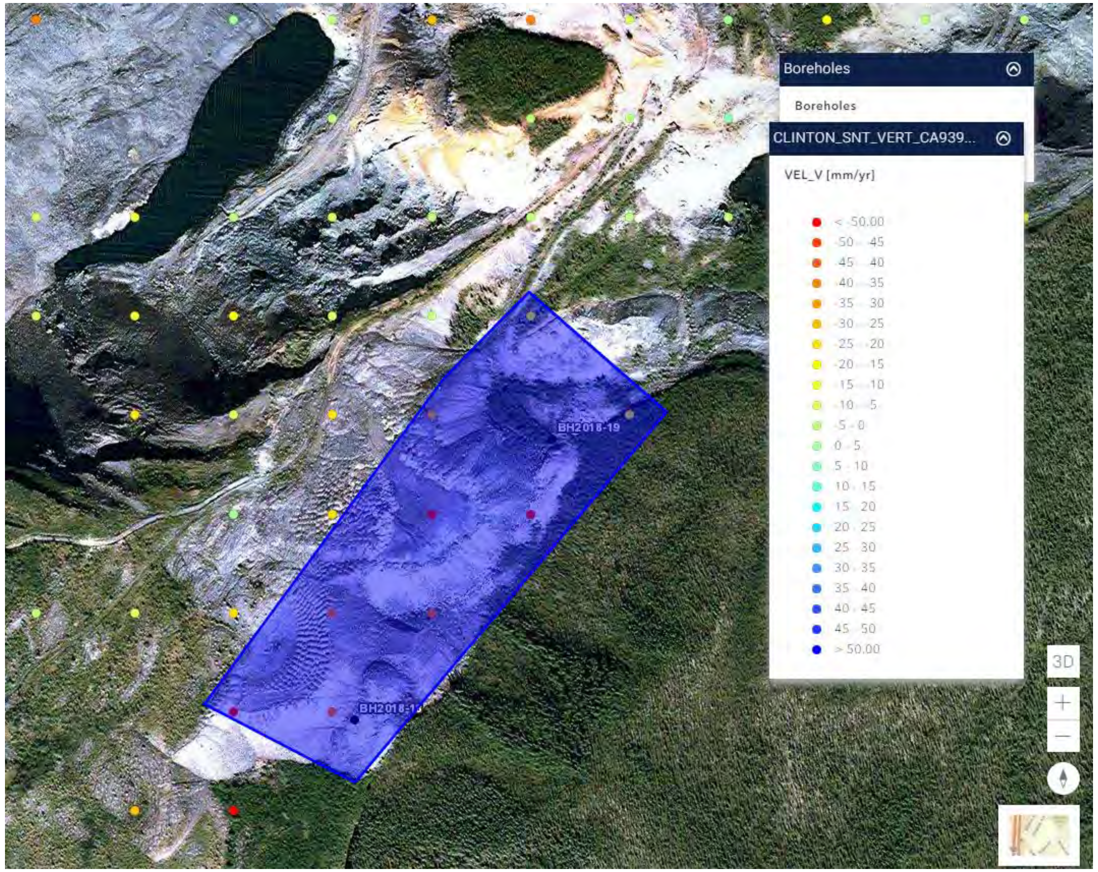
Average Movement Northeast of Porcupine Pit

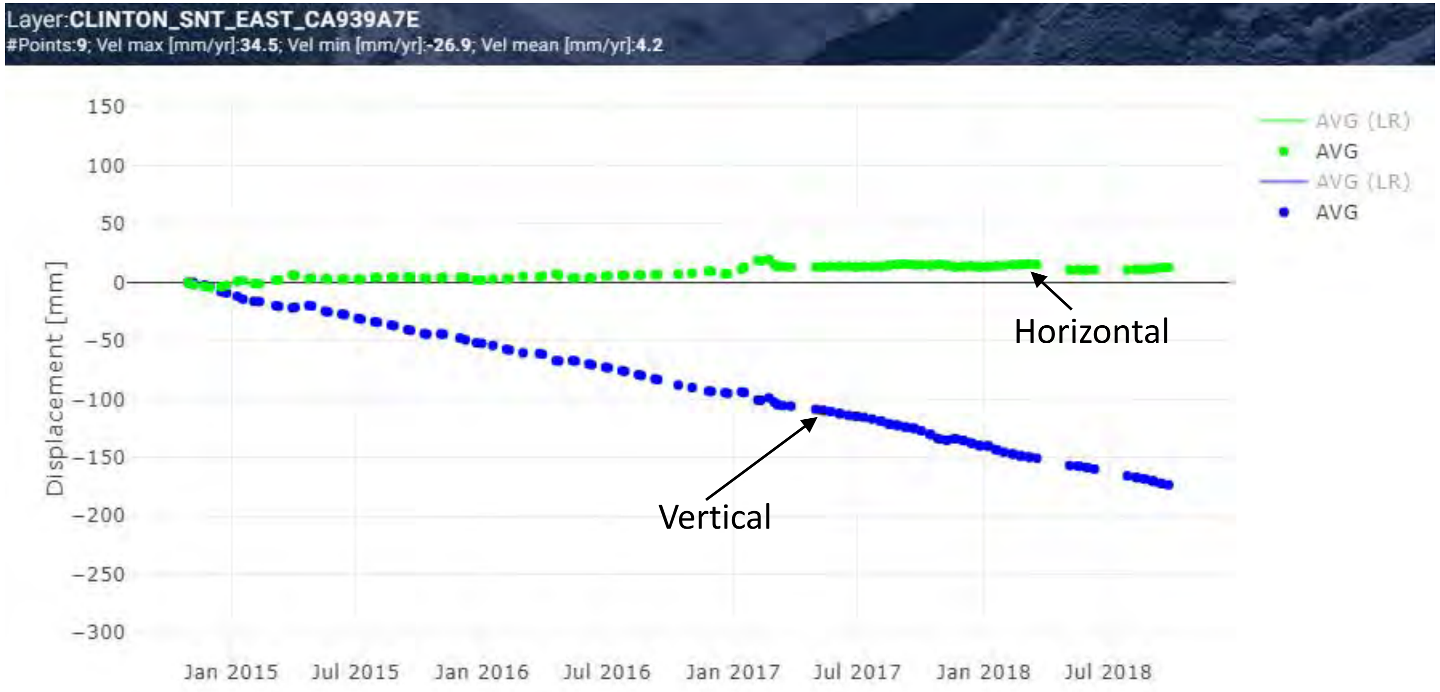




- Vertical settlement of 137 mm, average rate of 45.5 mm/year
- Horizontal movement of 47.5 mm westward, average rate of 12 mm/year
- Possible settlement due to high ice content colluvium along the pre-construction Clinton Creek valley south slope

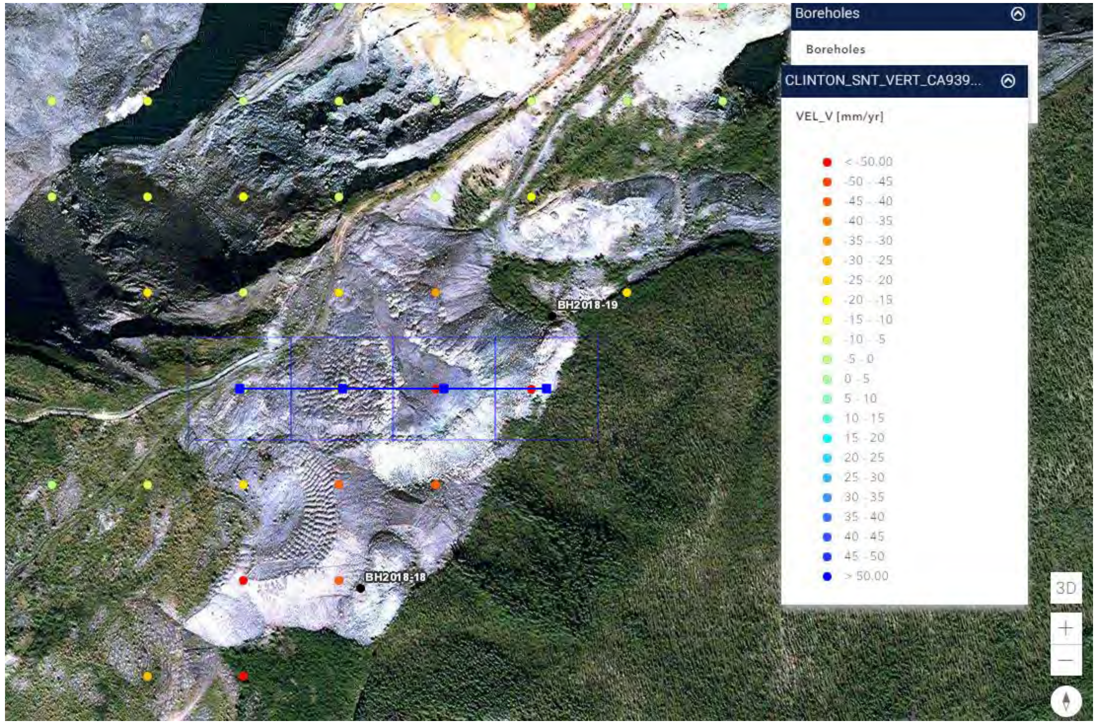
Average Movement Porcupine Pit Waste Dumps

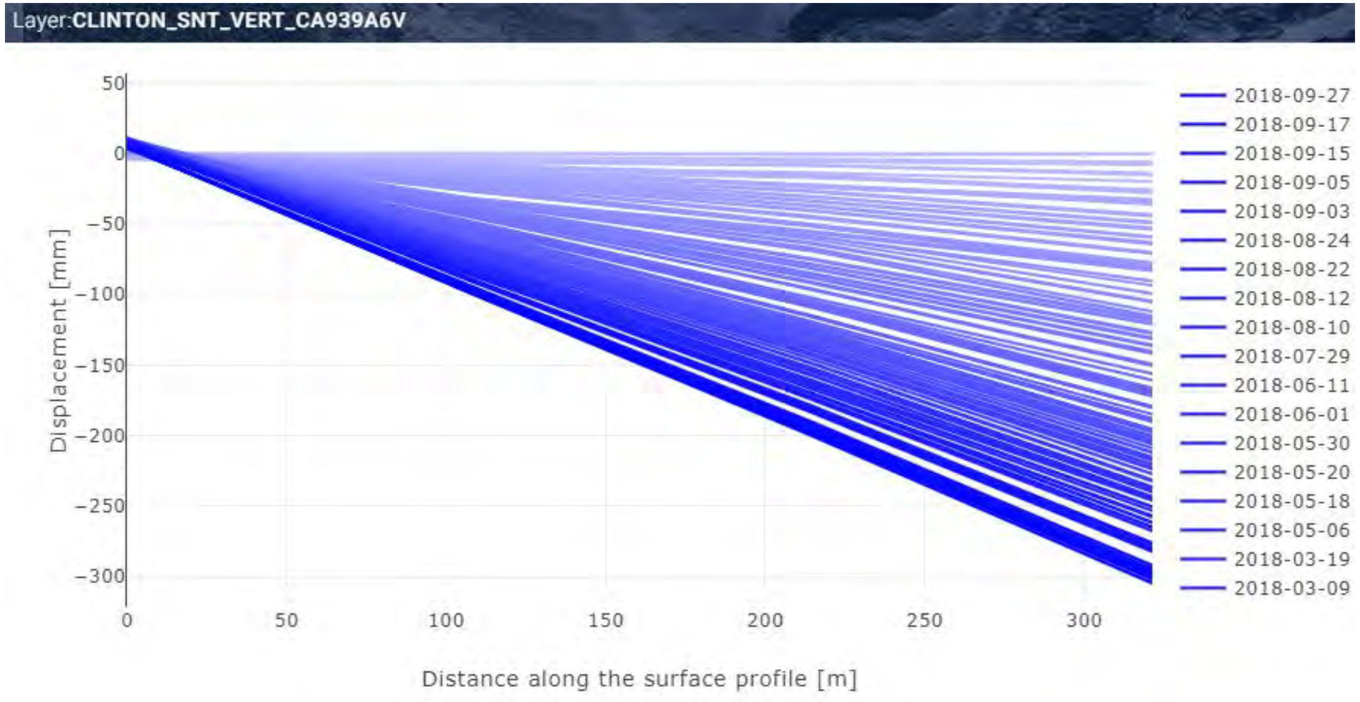




- Vertical settlement of 137 mm, average rate of 45.5 mm/year
- Essentially no horizontal movement
- Possible settlement due to high ice content colluvium as logged in BH18-18 and BH18-19

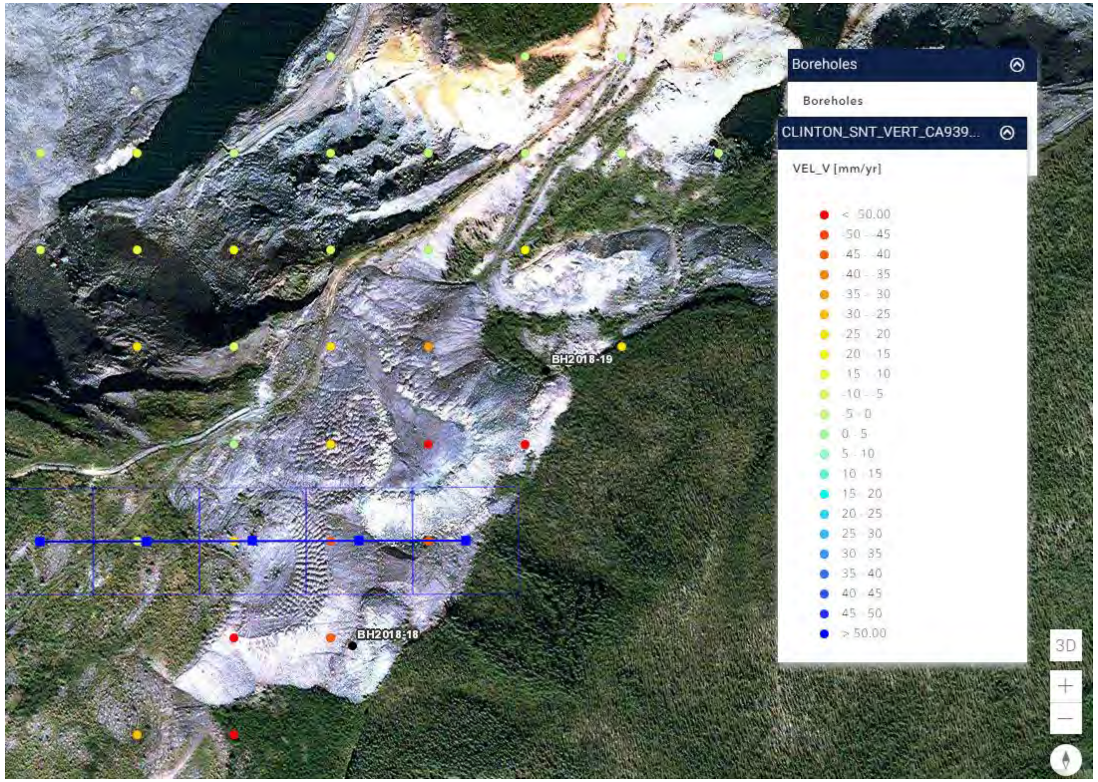
Average Movement Porcupine Pit Waste Dumps

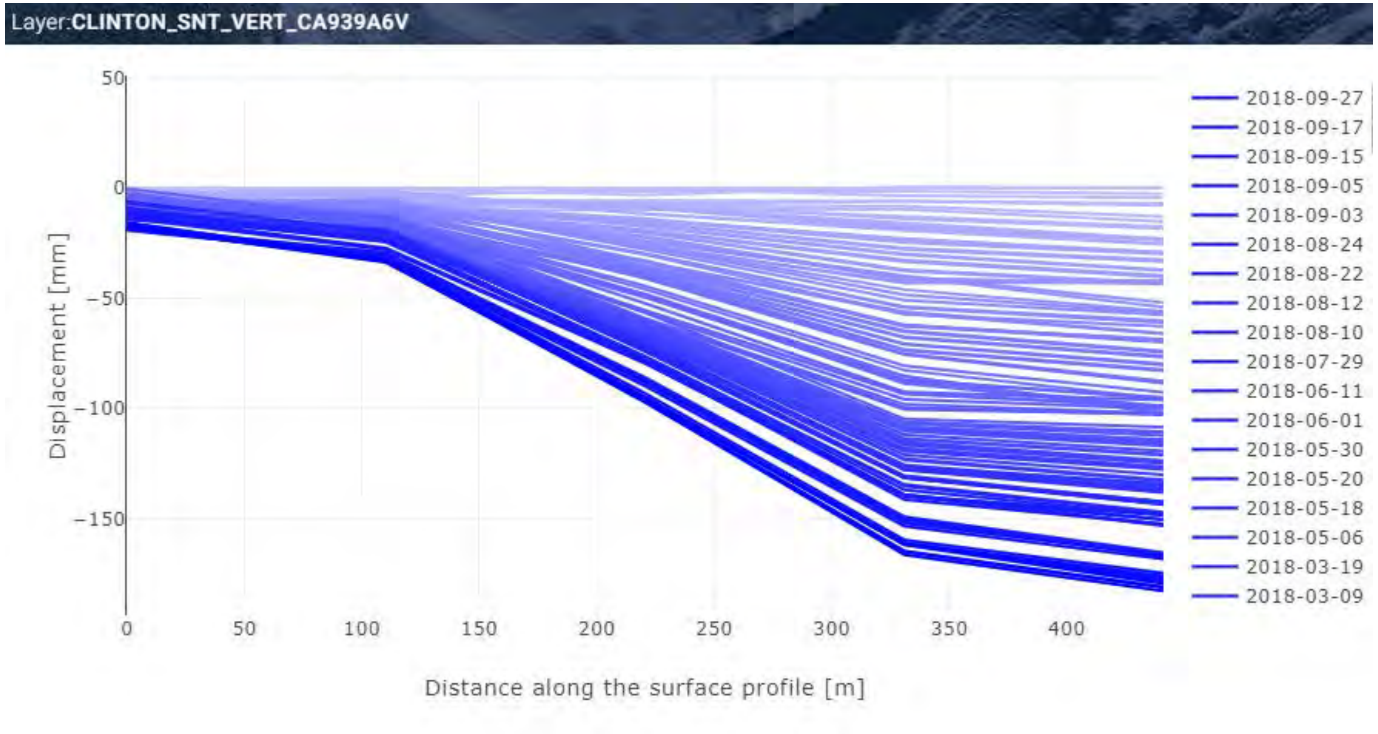




- Increased vertical settlement from west to east, moving into the pre-construction Porcupine Creek valley
- 305 mm of vertical settlement at the east end of the cross-section
- Essentially no settlement at the top of the waste pile adjacent to the pit

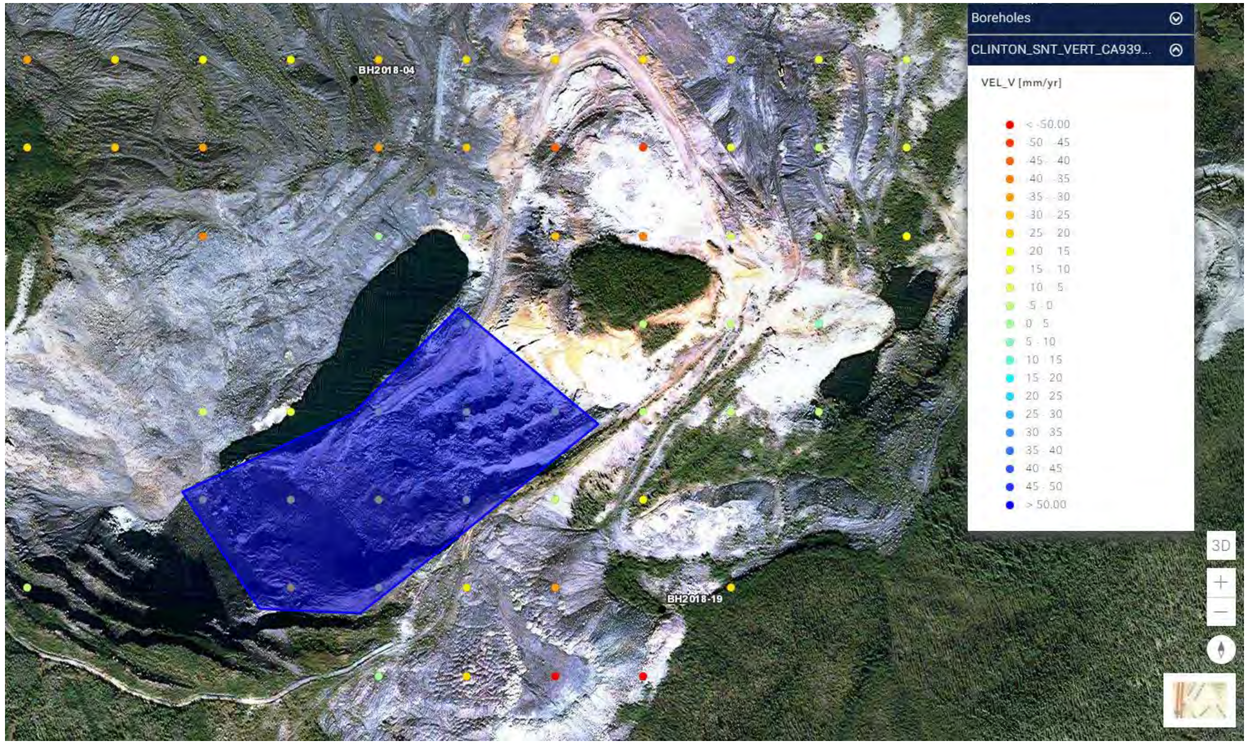
Average Movement Porcupine Pit Waste Dumps

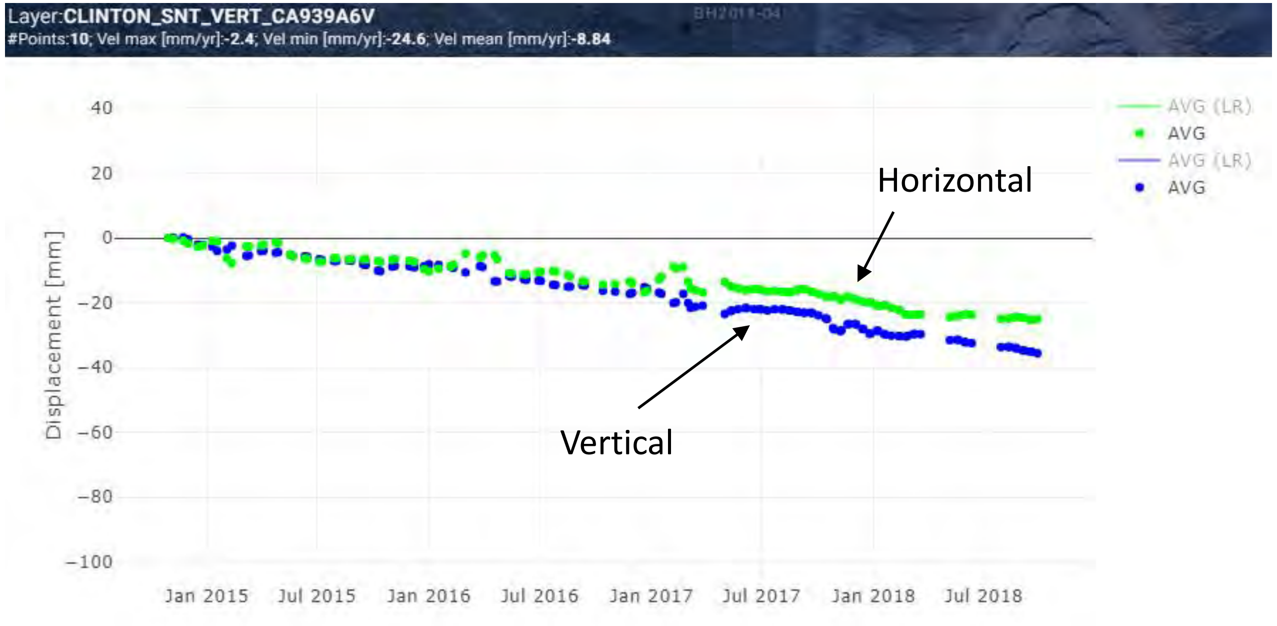




- Increased vertical settlement from west to east, moving into the pre-construction Porcupine Creek valley
- 183 mm of vertical settlement at the east end of the cross-section
- Less than 20 mm of settlement at the top of the waste pile adjacent to the pit/around the old mine roads

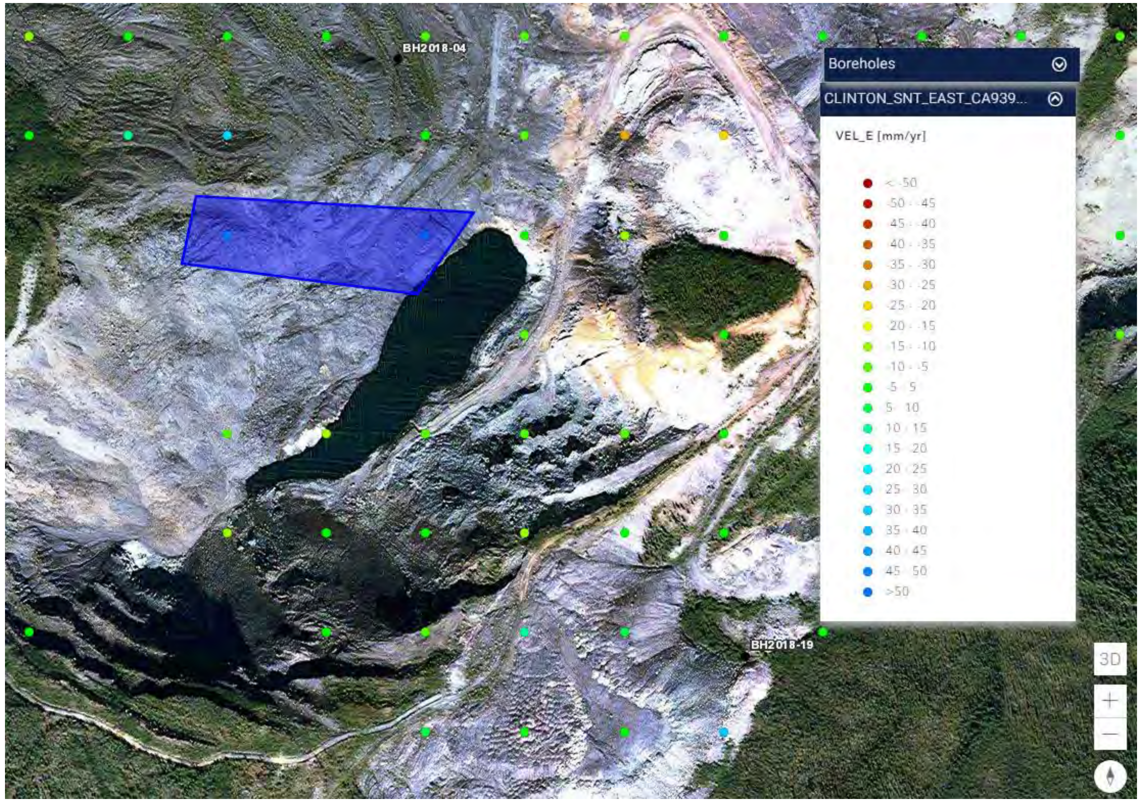
Porcupine Pit East Pitwall

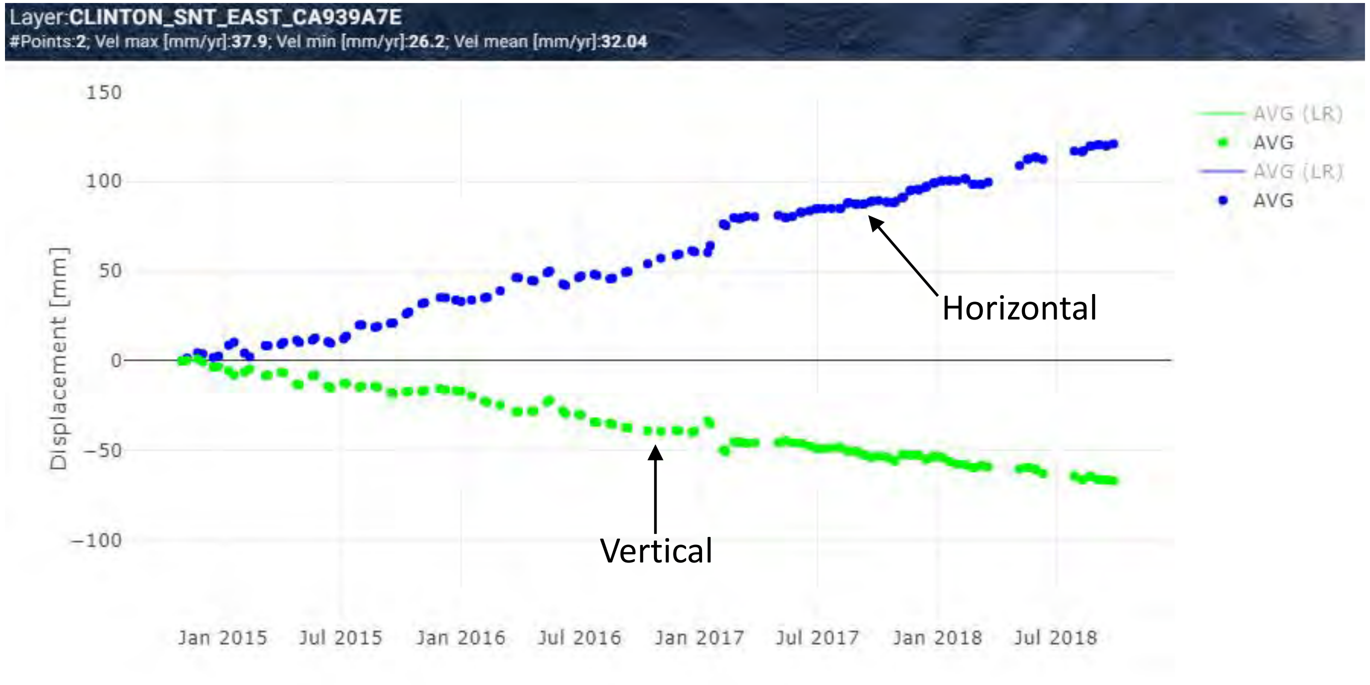




- Settlement of 36 mm at a rate of about 9 mm/year
- Horizontal movement of 25 mm westward at a rate of about 6 mm/year

Porcupine Pit West Pitwall





- Very limited data, likely due to movement in excess of what InSAR can process (ie greater than a single phase shift between orbits)
- Two points that are available indicate 121 mm of movement eastward and 67 mm of downward movement

Appendix C
Seepage and Internal Erosion Consideration

Wood Environment & Infrastructure Solutions,
a Division of Wood Canada Limited
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Burnaby, BC V5C 0E4
Canada

T: 604-294-3811

Memo

To: VE52705D

From: Elnaz Amirzehni, PhD, PEng
Ed McRoberts, PhD, PEng

Reviewer: Brian Ross, PEng (AB)

Wood File No.: VE52705D.100.3

Date: 26 August 2019

Re: **Seepage and Internal Erosion Considerations**

1.0 Introduction

1.1 Scope of Work

The intention of this memorandum is to address the following issues:

- Evaluating the hydraulic conductivity of the Clinton Creek Waste Dump from available grain-size distribution curves;
- Estimating hydraulic conductivity of the waste from Hudgeon Lake level drops;
- Assessing seepage defects and susceptibility to internal erosion in the Clinton Creek Waste Dump; and
- Conducting rapid drawdown analysis, to assess allowable lake drawdown rates so as to not initiate dump failures into Hudgeon Lake during lake lowering.

1.2 Outline of Memorandum

This memorandum summarizes the methodologies used to determine hydraulic conductivity from available grain-size distribution curves from 2016 and 2018 investigation programs in the Clinton Creek Waste Dump to assess rapid drawdown stability of the Clinton Creek Closure Concepts, in particular CC2 and CC3. Hydraulic conductivity from Hudgeon Lake level winter drawdown is also evaluated and presented. The potential for internal erosion caused by water flow and Clinton Creek site seepage gradients are assessed in this memorandum. Based on the considerations of internal stability, it can be expected that there could be an internal erosion problem, depending on seepage gradients. Therefore, seepage gradients through the waste in the Clinton Creek valley (i.e. from Hudgeon Lake to the downstream side of the Clinton Creek Waste Dump), into the Hudgeon Lake Spillway, Wolverine Creek (ie the gradient through the tailings blocking the creek), and Porcupine Creek (ie through the waste from the lake to the outlet) are evaluated. Rapid drawdown analysis is also conducted to asses static liquefaction susceptibility.

2.0 Hydraulic Conductivity Estimates from Grain-Size Distribution Curve

Obtaining an estimate of hydraulic conductivity is required to assess rapid drawdown stability of the CC2 and CC3 closure concepts. It may also be required for drainage collection systems under the spillway for the CC1 closure concept.

Chapuis (2016) summarized his earlier methodologies documented in a series of publications, see Reference List. The work has extended the widely used methodology began by Hazen (1892, 1911). The following two methods were chosen in this memorandum to estimate the hydraulic conductivity of soil from the grain-size analysis:

- Chapuis (2004) method:

  This method is used for natural non-plastic soil, including silty soil. The hydraulic conductivity in this method is calculated as:

  $$K_{sat}(cm/s) = 2.4622\left(\frac{d_{10}^2 e^3}{1+e}\right)^{0.7825}$$

  Where,

  K_{sat} = saturated hydraulic conductivity in cm/s,

  e = void ratio,

  d_{10} = effective diameter.

- Kozeny-Carman (Chapuis and Aubertin, 2003) method:

  This method is used for either non-plastic or plastic soil. The hydraulic conductivity in this method is calculated as:

  $$\log\ (K_{sat}) = 0.5 + log\left[\frac{e^3}{G_s^2\ S_S^2(1+e)}\right]$$

  Where,

  K_{sat} = saturated hydraulic conductivity in m/s,

  G_s = specific gravity of solids (non-dimensional),

  S_S = soil specific surface (m^2/kg of solids). For non-plastic soil, the soil specific surface is calculated using the following equation proposed by Chapuis and Légaré (1992):

  $$S_S(d) = \frac{6}{\rho_s}\sum\frac{P_{No.D} - P_{No.d}}{d}$$

  Where,

  ρ_s = solid density (kg/m^3),

  $P_{No.D} - P_{No.d}$ is the percentage of solid mass smaller than size D ($P_{No.D}$), and larger than next size d ($P_{No.d}$). For a non-plastic soil, using equation above requires having a complete grain-size distribution curve (sieving and sedimentation) to determine S_S .

Figure 1 shows all available grain-size distribution curves from 2016 and 2018 site investigation programs in the Clinton Creek Waste Dump. As can be noted from this figure, only limited curves include materials finer than #200 sieve (0.074 mm).

Figure 2 illustrates the selected complete grain-size distribution curves that were utilized in hydraulic conductivity calculations. Three additional upper-bound, average, and lower-bound grain-size distribution curves, corresponding to 40%, 25%, and 15% fines, respectively, were also considered. These synthetic curves were extended to beyond 0.074 mm grain-size by prorating the hydrometer data for finer than #200 sieve as follows. Six available hydrometer data shown in Figure 1 were used to come up with average slopes of the grain-size distribution curve finer than #200 sieve. It should be noted that one of the 2018 grain-size distribution curves are excluded from this practice. These average slopes were used to extend the rest of the grain-size distribution curves beyond 0.074 mm grain size. Thus, the portion of these curves shown in dashed-line in Figure 2 are from extrapolating the fine-grained end of the particle distribution.

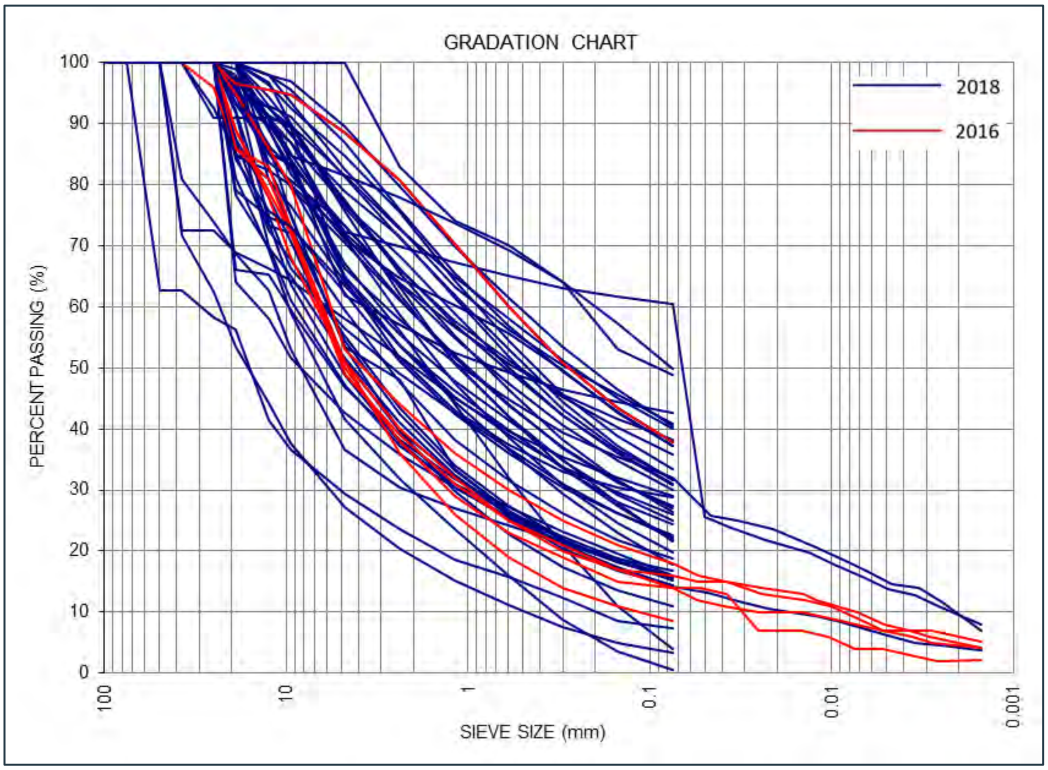

Figure 1: *Grain-size distribution curves from 2016 and 2018 site investigation programs in the Clinton Creek Waste Rock dump.*

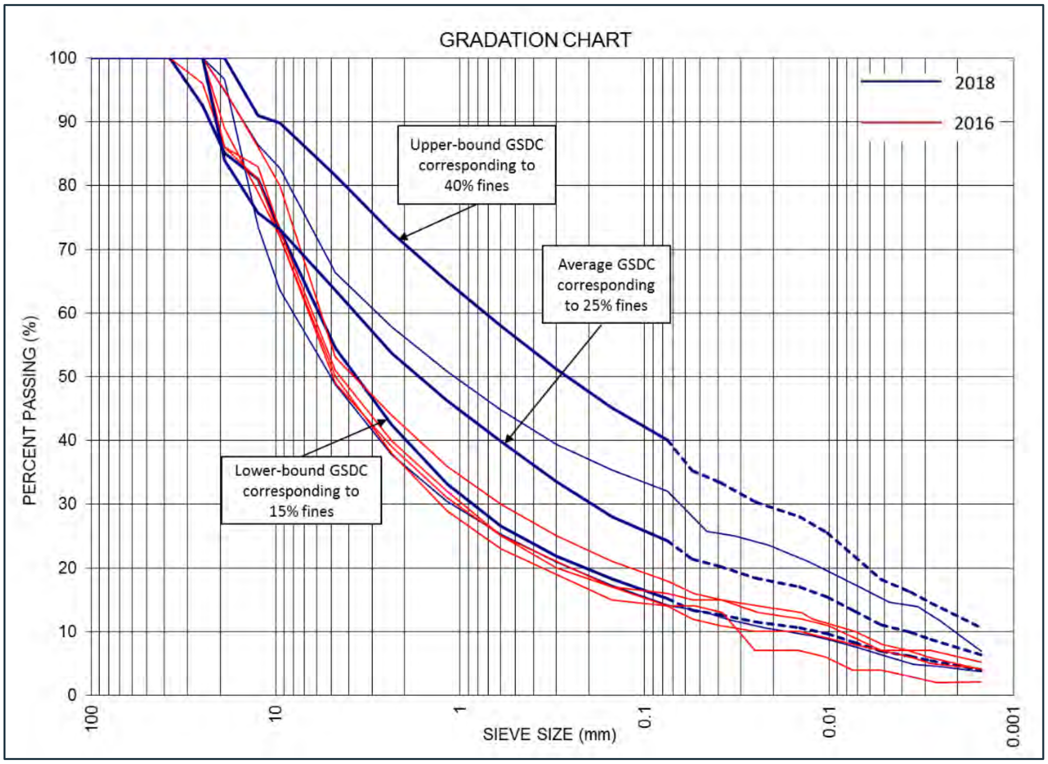

Figure 2: *Selected complete grain-size distribution curves (sieving and sedimentation) used in the hydraulic conductivity calculations.*

Figure 3 shows the calculated hydraulic conductivities using Chapuis (2004) and Kozeny-Carman (Chapuis and Aubertin, 2003) methods. The calculations were conducted on all available complete grain-size distribution curves (which include sieving and hydrometer shown in Figure 2. As can be noted from this figure, the predicted hydraulic conductivity falls within a range of $5 \times 10^{-5}\, m/s$ and $10^{-6}\, m/s$.

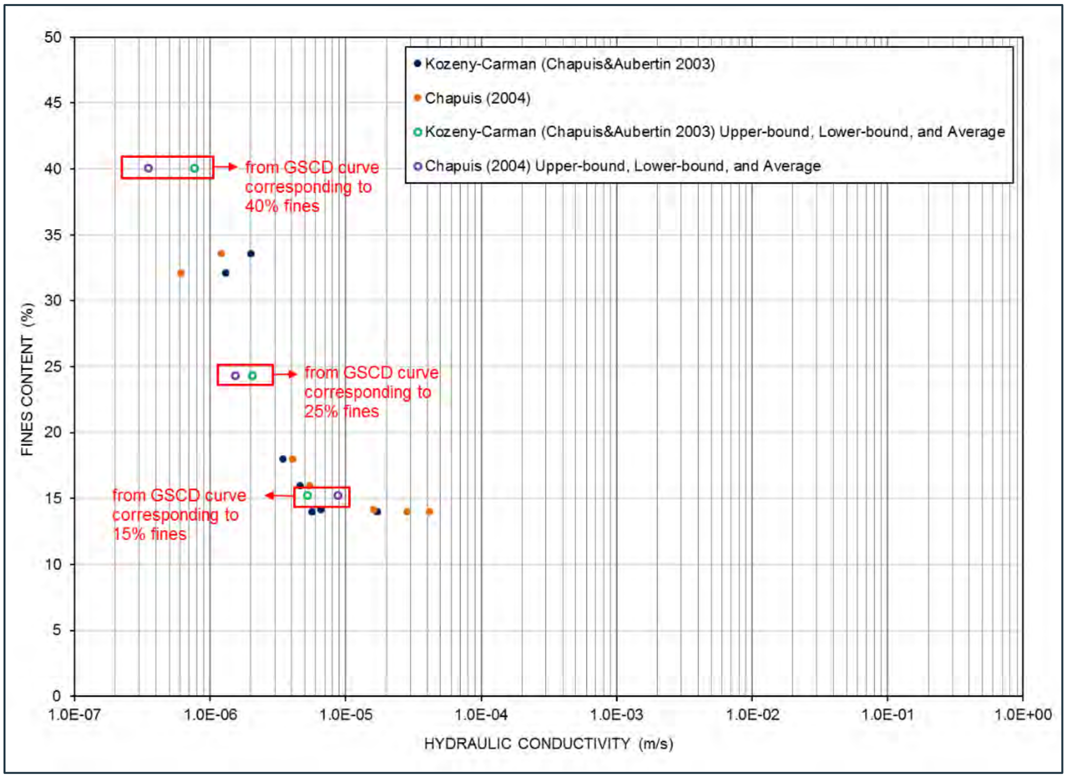

Figure 3: *Hydraulic Conductivity vs Fines Content from the Clinton Creek Waste Rock Dump.*

It should be noted that the models used are for water at 20^oC, and therefore a temperature correction must be made to account for an in-situ groundwater temperature using equations below to account for kinematic viscosity correction (Chapuis, 2012):

$$K(T^oC) = K(20^oC)\frac{\upsilon(20^oC)}{\upsilon(T)}$$

$$\frac{\boldsymbol{\upsilon}(\mathbf{20^oC})}{\boldsymbol{\upsilon}(\mathbf{T})} = \mathbf{10}^{\frac{\mathbf{1.37023(T-20)+0.000836(T-20)^2}}{\mathbf{109+T}}}$$ Dorsey (1968)

$$\frac{\upsilon(20^oC)}{\upsilon(T)} = exp\left(\frac{509.53}{20 + 123.15} - \frac{509.53}{T + 123.15}\right)$$ Vogel-Tammann-Fulcher (VTF) equation

Where υ is the kinematic viscosity of water, and T is in ^{0}C. Assuming the in-situ groundwater temperature of around 2^oC to 4^oC, the correction factor results in the range of 0.60 to 0.64. By applying the temperature correction factor, the hydraulic conductivity falls within a range of $0.3 \times 10^{-5}\, m/s$ and $0.6 \times 10^{-6}\, m/s$, which is a negligible change. It should be also noted that the correction for water density is not considered because it can be neglected in comparison with the correction for water temperature (Chapuis, 2012).

3.0 Hydraulic Conductivity Estimated from Hudgeon Lake

An estimate of the hydraulic conductivity through the waste dumps can be arrived at by considering that during the winter months the lake level should be relatively constant, as there are no run-off inflows into the lake. Some snow may collect on portions of the ice surface.

As illustrated in Figure 4, there are two piezometers installed on the lake shore relatively near the spillway. There is an older vented piezometer located at a depth of about 0.5 m of water. In August 2018 Wood installed a non-vented VW-tip at about the 5 m depth. This 5 m deep tip requires barometric pressure corrections from the site weather station, whereas the vented tip does not. As these tips measure total water pressure, it does not matter if some water is ice, the pressure measured at the lake bottom will be the same.

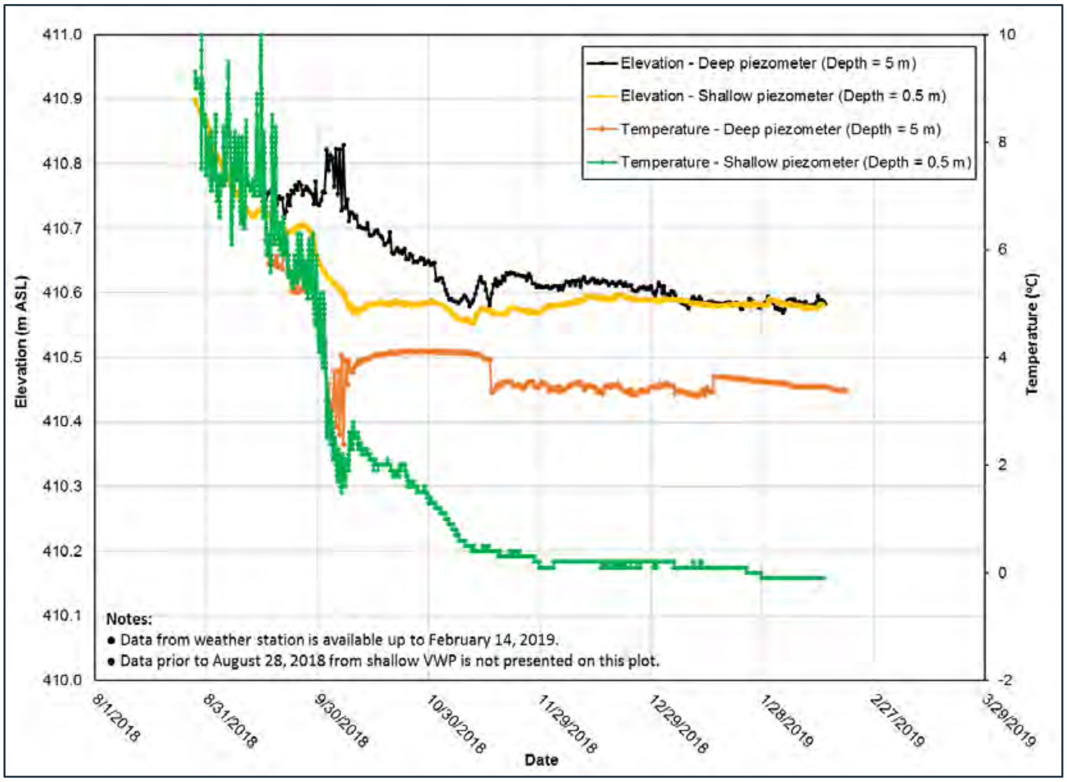

Figure 4: *Plots of water level elevation vs time and temperature vs time from two piezometers installed on the lake shore relatively near the spillway since August 2018.*

If a reliable water level vs time is obtained during winter months and can be largely attributed to seepage into the dump fills, a hydraulic conductivity can be determined as follows:

- Say the lake drawdown is 10 mm/month, or 0.01 m/month, based on current data.
- The lake area is $719{,}000\ m^2$. So, the total flow out of lake in winter is $Q = 2.8 \times 10^{-3}\ m^3/s$ assumed all into dump fills (that is no open water flow in spillway)
- The total area of waste dump across valley wall is $A = 10{,}400\ m^2$.
- A reasonable hydraulic gradient in consideration of overall landslide dam and spillway of $i = 0.06$.
- So, hydraulic conductivity $K = \frac{Q}{iA} = 4.4 \times 10^{-6}\ m/s$.

Rounded to $5 \times 10^{-6}\ m/s$ this is a reasonable fit with the range of hydraulic conductivities from the grain-size distribution curves. It is acknowledged that the agreement may be fortuitous. Back analysis is complicated by the amount of water flowing under the ice in the embayment (and shallow ford) between the lake and the first gabion basket at the beginning of the spillway. If this embayment froze completely in the later winter, then all flow out of the system is more likely infiltration into the dump. For a given winter, infiltration could be more of a transient response, rather than steady state as implicit in the simple use of Darcy's Law. Historically, the shallow piezometer at 0.5 m can freeze in the winter and therefore reliable data is not available. For this reason, the deeper piezometer was installed, and better estimates may be achieved prior to breakup in 2019.

4.0 Seepage Defects and Internal Erosion

This section quantifies the potential for internal erosion caused by the water flow and seepage that could occur in the Clinton Creek Waste Dump. Seepage and internal erosion, as well as the damages and consequences that such erosion might produce, are important phenomenon that are necessarily considered in design.

4.1 Criteria

As cited in Casagrande (1968), Lane (1935) defined the susceptibility to soil erosion through a weighted creep ratio C_w in terms of the horizontal and vertical paths of the water flow, the type of soil and the water head between the upstream and downstream water levels of a hydraulic structure:

$$C_w = \frac{\sum L_H + 3\sum L_V}{3H}$$

Where,

C_w = weighted creep ratio,

L_H = horizontal or flat contact distances,

L_V = vertical or steep contact distances, for our purposes this is zero,

H = head between upstream and downstream.

The value of $C_w = 6.0$ is recommended by Lane (1935) to avoid soil erosion and piping in medium sand materials. Assuming very little vertical flow, the average hydraulic gradient of $1/17$ to $1/18$ is required to prevent piping in a conservatively selected medium sand.

As cited by Rivard (1981), Casagrande indicated at a lecture in Saskatoon in 1979 that he designed dams on previous foundations with an average hydraulic gradient of $1/20$. This average hydraulic gradient was utilized in design of the Gardiner Dam, Tarbella and Columbia projects, where the gradient was defined as the water head divided by embankment length. Rivard (1981) recommended that the average hydraulic gradient to prevent piping should be about $1/15$ to $1/20$. He stated if the foundation soil is very susceptible to piping, the average hydraulic gradient should not exceed $1/25$; and if the soils are resistant to piping, the average hydraulic gradient should not exceed $1/10$.

4.2 Internal Stability

In this section the internal instability, which refers to the movement of finer particles through a network of coarser material, is evaluated.

The internal stability is governed by two main parameters. The first one is the grain size distribution curve, which represents a geometric criterion and is used to determine susceptibility to internal instability. The second parameter is a critical hydraulic gradient at which the onset of internal instability initiates, which establishes a hydromechanical relation. The later was described in the previous section.

Three different methods were utilized in this memorandum to evaluate the susceptibility to internal instability. Upper-bound, average, and lower-bound grain-size distribution curves corresponding to 15%, 25% and 40% fines, respectively, were evaluated using these three methods:

- Method of Kezdi (1969)
  - In this method the grain-size distribution curve is divided into two parts: fine part and coarse part. He defined an instability degree as $I_r = \frac{D_{15\,(coarse)}}{D_{85\,(fine)}}$, which must be lower than 4 to avoid internal instability of fine particles in the void space of larger particles. The Kezdi (1969) method of $I_r < 4$ is equivalent to the following: if a portion of the grain-size distribution curve has a slope lower than 24.9% per log cycle, it will be unable to retain its particles finer than the grain size at which such slope occurs (Chapuis, 1992).
- Method of Sherard (1979)
  - The method of Sherard (1979) is similar to that of Kezdi (1969). In this method the grain-size distribution curve is also divided into two parts: fine and coarse. In this method an instability degree must be lower than 5 to avoid internal instability of fine particles. Sherard (1979) method of $I_r < 5$ is equivalent to the following: if a portion of the grain-size distribution curve has a slope lower than 21.5% per log cycle, it will be unable to retain its particles finer than the grain size at which such slope occurs (Chapuis, 1992).
- Method of Kenney and Lau (1985, 1986)
  - This method is only valid for small size particles ($\leq D_{20}$). In this method the percentage of particles having a size between d and $4d$ must represent at least the percentage of particles smaller than d. This is equivalent to the following: at a particle size D_y ($y \leq D_{20}$), the slope per log cycle of the gradation curve must be higher than $1.66y$ to have internal stability. The equation gives following slopes: at D_5, $slope >$ 8.3%; at D_{10}, $slope >$ 16.6%; at D_{15}, $slope >$ 24.9%; and at D_{20}, $slope >$ 33.2% (Chapuis, 1992).

Figure 5 shows the susceptibility to internal instability of the upper-bound grain size distribution curve corresponding to 40% fines using Kezdi (1969), Sherard (1979), and Kenney and Lau (1985, 1986) methods. As illustrated in this Figure, based on the methods of Sherard (1979) and Kezdi (1969), the soil is unable to stabilize its own particles finer than 0.3 mm and 1.3 mm, respectively. Applying Kenney and Lau (1985, 1986) criterion shows internal instability of small size particles ($\leq D_{20}$).

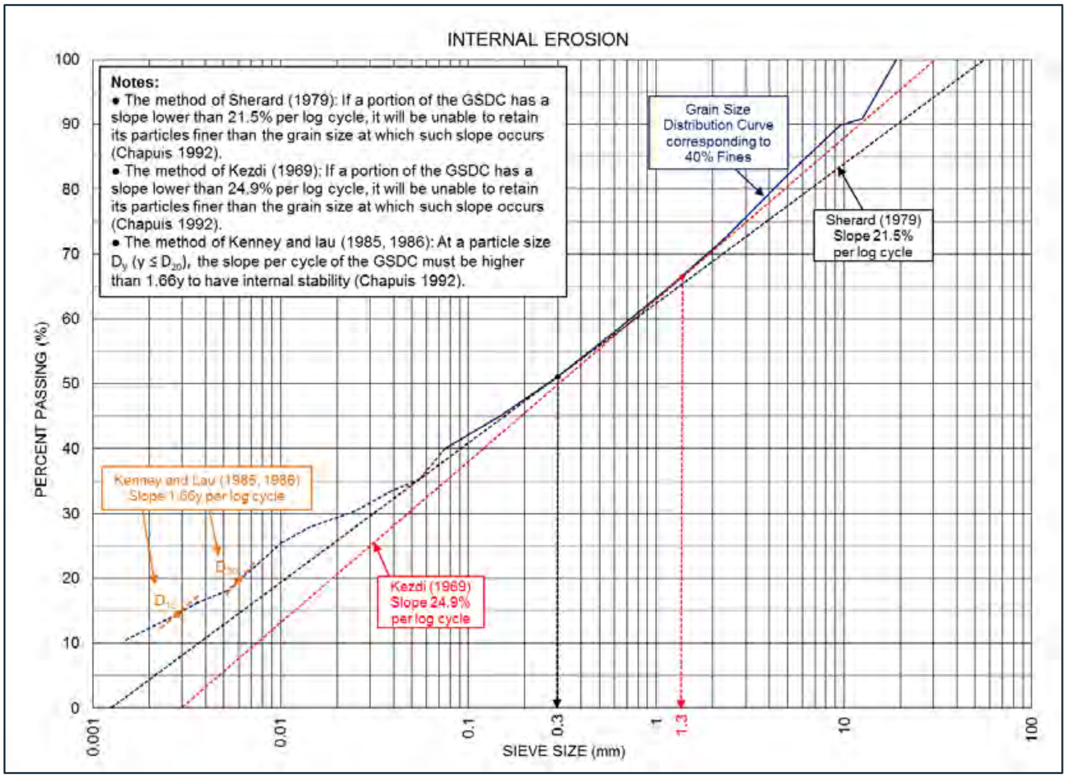

Figure 5: *Evaluation the susceptibility to internal instability of the upper-bound grain size distribution curve corresponding to 40% fines using three different methods outlined in this memorandum.*

Figure 6 shows the susceptibility to internal instability of the average grain size distribution curve corresponding to 25% fines using three aforementioned methods. Based on the methods of Sherard (1979) and Kezdi (1969), the soil is unable to stabilize its own particles finer than 0.7 mm and 1.6 mm, respectively. Applying Kenney and Lau (1985, 1986) criterion shows internal instability of small size particles ($\leq D_{20}$).

Figure 7 shows the susceptibility to internal instability of the lower-bound grain size distribution curve corresponding to 15% fines. Based on the methods of Sherard (1979) and Kezdi (1969), the soil is unable to stabilize its own particles finer than 0.6 mm and 1.2 mm, respectively. Applying Kenney and Lau (1985, 1986) criterion shows internal instability of small size particles ($\leq D_{20}$).

In summary these methods indicate that there is an issue with internal stability.

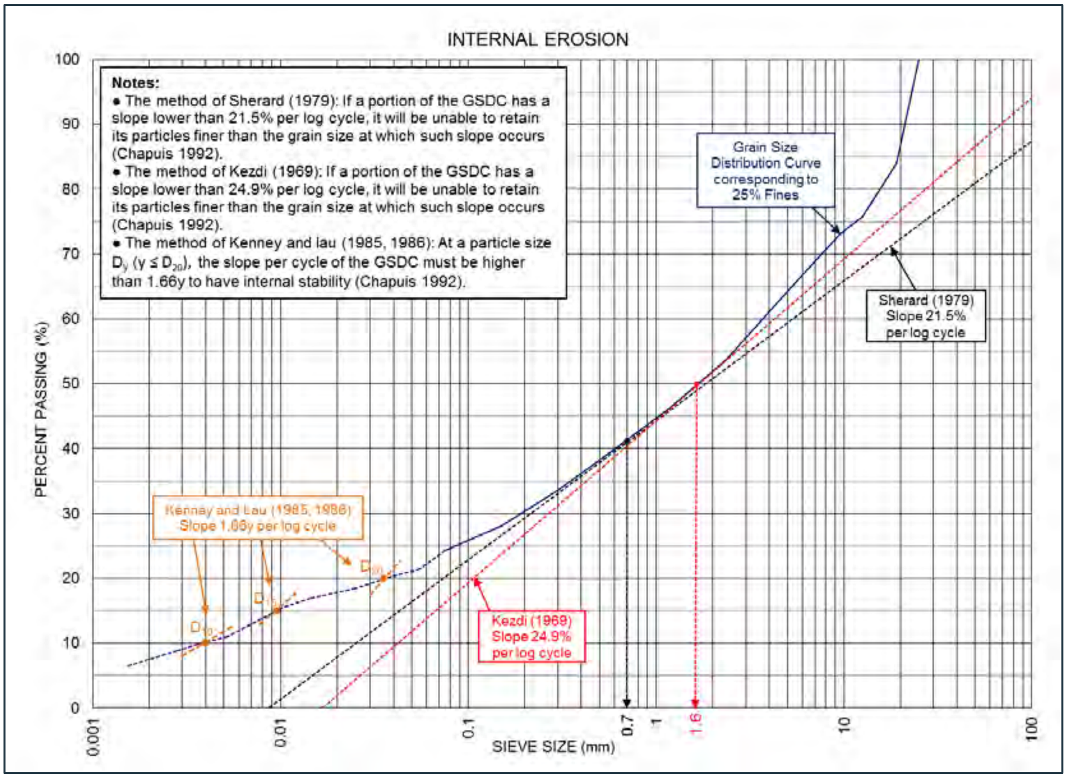

Figure 6: *Evaluation the susceptibility to internal instability of the average grain size distribution curve corresponding to 25% fines using three different methods outlined in this memorandum.*

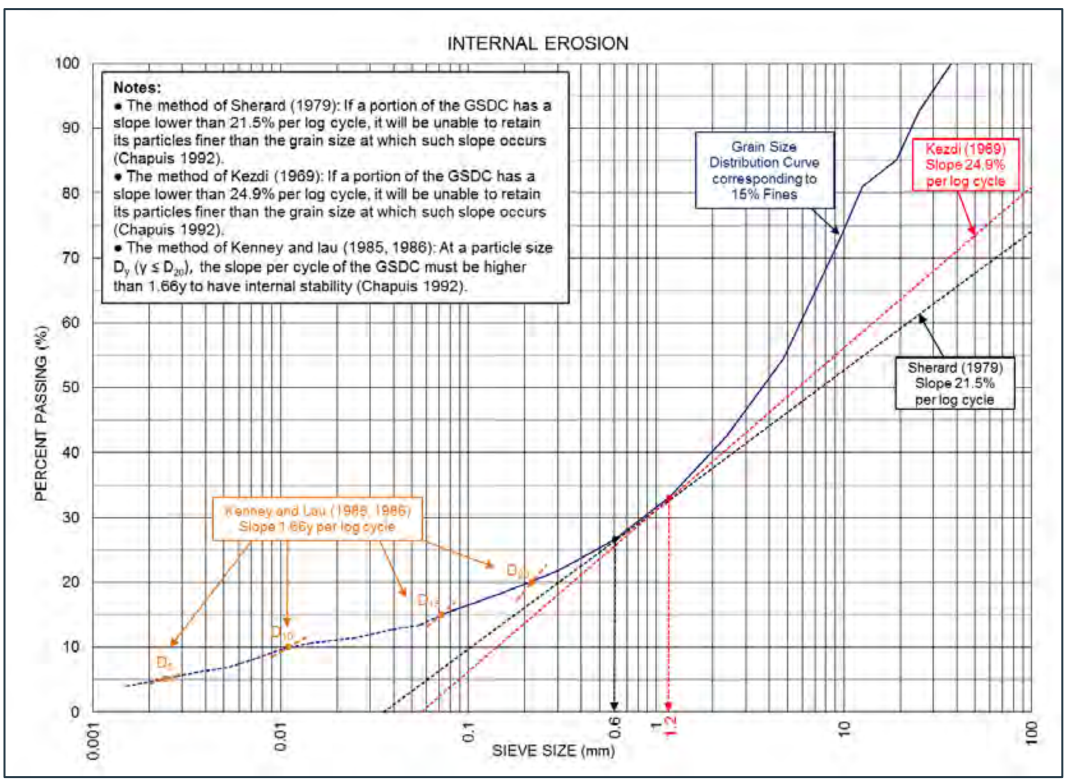

Figure 7: *Evaluation the susceptibility to internal instability of the lower-bound grain size distribution curve corresponding to 15% fines using three different methods outlined in this memorandum.*

4.3 Clinton Creek site Seepage Gradients

Based on the considerations of internal stability, it can be expected that there could be an internal erosion problem, depending on seepage gradients.

4.3.1 Overall Waste in Clinton Creek valley

The gradients imposed on the overall landslide dam from the lake to the east toe of the dumps, where the outlet widens out, are 1/20 or less. Exit gradients from BH18-9 to the dump toe are from 1/35 to 1/40. These hydraulic gradients are considered low risk.

It is noted that no seepage has been observed exiting at the dump / alluvium contacts. It appears that seepage is downwards into the alluvial materials inside the dumps, or to the bed of the spillway channel. Alternatively, seepage may not have reached steady state and the free water table is still advancing down gradient.

4.3.2 Hudgeon Lake Spillway

The hydraulic gradients from the lake to the toe of the spillway chute at DS4 (now eroded out) is about 1/10. Local gradients between the spillway pools, or through the gabions, are in the vicinity of 1/5. These hydraulic gradients indicate a sensitivity to internal erosion.

Figure 8 records an observed localized flow and probably internal erosion during the fall 2018 field program. Seepage is exiting downstream of the DS4 gabions along the north wall, apparently under the chute. This seepage likely undermined the chute. Localized seepage, and likely internal erosion, along a construction joint is visible as a horizontal layer where seepage exits.

Figure 8: *Observed localized flow and probably internal erosion during the Fall 2019 field program.*

4.3.3 Wolverine Creek

Hydraulic gradients through the tailings plug creating a small pond in the Wolverine Creek valley appear low. The north lobe landslide dam has not yet been assessed.

4.3.4 Porcupine Creek

As outflow from the Porcupine Creek watershed flows through the dumps, or possibly the original creek alluvial bed, then there is no immediate internal erosion concern to answer to. The potential failure mechanism is plugging of the subterranean flow channel leading eventually to the formation of a larger lake.

One source of fines for plugging is the thawing of ice rich colluvium with time, predicting a fines load. Balancing against that process is the likelihood that the hydraulic gradients that would develop will keep fines mobile; that is, continue to flush the flow pathways. There is some expectation that the current balance between internal erosion and fines deposition will remain so.

4.4 Landslide Dams

Schuster (2000) provides a summary of the safe performance of landslide dams built on pre-existing landslides. Schuster summarized 167 Large Dams (Definition > 15 m height) worldwide of which only (4) four failed due to problems associated with pre-existing landslides. Schuster summarizes his findings as:

1. Seepage through landslide abutments has been the most common source of failures. When carried to the extreme, seepage could possibly lead to piping, however no case was found in which this occurred because of a dam having been constructed on a landslide. (Wood interprets this conclusion as being due to the width (i.e. distance from the dammed lake to the downstream end of the deposit) of a landslide dam).
2. There is no clear indication as to which landslide types perform best or worst from a stability perspective when used as a foundation or abutment, with slides in shales probably performing the poorest.
3. In contrast, all landslide types seem to be subject to seepage problems unless appropriate preventative or remedial measures are taken.
4. While all aspects of the physical characteristic of the individual landslide should be considered carefully, "of particular importance is the permeability of the landslide mass."

While Schuster concerned himself with dams built on landslides that have partially or completely blocked a valley his remarks provide useful guidance. It is to be noted that landslides that block valleys usually fail by overtopping, followed by surface erosion of an outlet, followed by a breach. If a spillway is provided before overtopping occurs, then the structure could be called a landslide dam. As in the case of Clinton and Wolverine Creeks these spillways typically require on-going maintenance. Generally, these landslide dams require additional design modifications to function.

Schuster's conclusion #1 is interpreted to mean that while seepage through a landslide dam could in principle lead to piping or internal erosion failures there have been no failures attributed to this mechanism in the failures reported. This provides qualitative support for the conclusion that the Clinton Creek dumps will not suffer from internal erosion.

5.0 Rapid Drawdown Analysis

The CC2 and CC3 options raise the question of allowable lake drawdown rate as to not initiate dump failures into Hudgeon Lake. Rapid drawdown could trigger static liquefaction failures, initiation of which is not monitorable, as such failures can develop very quickly and pose a significant safety threat. The currently recommended cut-back for slopes are based on maintaining an internal piezometric level equal to the lake level.

Analyses are reported in Appendix I for the CC3 option formulated a simple model and dump and alluvial adopting hydraulic conductivities from $5\text{x}10^{-5}\ m/s$ and $10^{-6}\ m/s$. This model assumed the lake was instantaneously dropped 30 m, from elevation 412 m a.s.l, which represents a 2-D section through the deepest part of the lake. Reasonable unsaturated flow parameters were adopted. The model considered a no-flow boundary more-or-less in the middle of the dump. No recharge due to precipitation was included but is not considered significant at this time. These analyses (see Appendix I, Figures 9 and 10) indicate that drawdown governed by hydraulic conductivity of 10^{-6} m/s could pose significant delays in executing a no-lake option whereas a dump with significant $5\text{x}10^{-5}$ layers would drain rapidly.

From a design perspective, the basic question is can dump removal excavations proceed on the premise that the piezometric head within the dump will drop at the same rate as the lake is lowered, therefore not being a consideration in operational scheduling. This model indicates that this factor needs to be carried further into consideration in design and scheduling,

The key factor is the permeability of the dump and potential for sub-horizontal drainage layers. Based on fines content distribution and sample descriptions, there is no readily identifiable drainage zones.

6.0 Conclusions

The main findings in this memorandum are:

- Clinton Creek
  - The overall dump with a Lake at 412 m a.s.l (i.e. the current condition, and Option CC1) can be considered safe against internal erosion.
  - Re-designs of the spillway on the same vertical alignment and concept fails acceptable internal erosion criteria, and preventative measures should be built into the design
  - A regime channel at a uniform 3% grade means an internal erosion hydraulic gradient of 1/33 which is acceptable. Locally steep hydraulic gradients greater than 1/20 must be avoided.
  - Analysis of hydraulic conductivity of waste dumps indicates that drawdown rate will be an uncertainty factor in the design of the dumps for safety during operations and scheduling of operations.
- Wolverine Creek
  - At this time Wood has not considered the in-valley seepage issues for closure designs in Wolverine Creek valley.
- Porcupine Creek
  - The current status of flow from Porcupine Creek watershed, with assumed flow under the dumps, may be sustainable, but with time flow might plug. Performance projections considering the long-term impact of a major seismic event further complicate decision making. It is not realistic to expect confident predictions one way or the other.

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Appendix I
Clinton Creek Mine Waste Dump Drawdown Predictions Memo

Memo

To: VE52705D

From: Yaming Chen | **Reviewer:** Ed McRoberts

cc: Ed McRoberts | **Wood File No.:** VE52705D.100.3

Date: 18 March 2019

Re: **CLINTON CREEK MINE WASTE DUMP DRAWDOWN PREDICTIONS**

1.0 Introduction

This memorandum presents the methodology and results for Clinton Creek Mine waste rock dump (WRD) drawdown predictions, in support of geotechnical analysis for slope stability and water management.

2.0 Methodology

The industry-standard software SEEP/W within the GeoStudio 2016 version 8.16.2.14053 package is adopted for developing a two-dimensional groundwater flow model with a domain along the central line of the existing WRD cross-section, which extends from the pit lake to the center of the dump.

The conceptual groundwater flow system is illustrated in **Figure 1**. The WRD is 30 m high and 450 m long, and the basal drain (representing the top bedrock beneath the dump) is 10 m thick and 480 m long. The materials in both units are assumed to be homogenous and isotropic.

The numerical groundwater model is developed based on the conceptual model, and it is carried out in two steps as following:

1) Step 1: steady-state flow simulation for calculation of hydraulic heads at the existing conditions with the pit lake elevation at 412 meters above sea level (masl); and
2) Step 2: transient flow simulation for predictions of drawdown vs. time using the solutions of the hydraulic heads from Step 1 as initial heads, by assuming the pit lake elevation drops instantaneously from 412 to 382 masl (***Scenario 1: Full Pit Drawdown***) and from 412 to 402 masl (***Scenario 2: 10m Pit Drawdown***).

The model is run with variably-saturated groundwater flow for seven simulations, including five for **Scenario 1 (a, b, c, d, e)** and two for **Scenario 2 (a, b)**. For saturated flow, the hydraulic conductivities (Ks, m/s) and the volumetric water contents (m^3/m^3) used in the simulations are listed in **Table 1**. For the unsaturated flow, the data point functions for relative hydraulic conductivity (Kr, m/s) vs. pore-water pressure / matric suction (kPa) and for volumetric water content (m^3/m^3) vs. pore-water pressure / metric suction (kPa) available for sand in the SEEP/W database are used to represent the materials within the WRD and the basal drain.

For Step 1 steady-state flow simulations, a constant head at 412 masl is assigned along the WRD slope and the toe of the basal drain to represent the full pit lake reservoir at the existing conditions. For Step 2 transient flow simulations in Scenario 1 (a, b, c, d, e), a constant head at 382 masl is assigned at the toe of the WRD and the toe of the basal drain to represent the pit lake drops instantaneously from the existing full reservoir conditions to the pit bottom (to represent the full pit drawdown), together with a potential seepage boundary being applied along the WRD slope to allow passive drainage of water out of the dump towards the pit. For the Step 2 transient flow simulations in Scenario 2 (a, b), a constant head at 402 masl is assigned along the WRD slope from the elevation 402 masl to the toe of the dump and the toe of the basal drain (to represent the 10 m pit drawdown), while a potential seepage boundary is applied along the slope at between 402 to 412 masl. The bottom of the model domain and the upper water divide on the left-hand side of the domain are assumed as no flow boundaries.

The transient flow simulations are run with a minimum pressure head difference of 0.005 m as a convergence criterion, an initial timestep of 30 days, and the outputs at 30 timesteps in 100 years of duration. The steady-state flow simulations are run with the same tight convergence criterion.

Table 1: Hydraulic Parameters for Saturated Groundwater Flow

Model Scenario	Saturated Hydraulic Conductivity Ks (m/s)	Saturated Volumetric Water Content (m3/m3)	Residual Volumetric Water Content (m3/m3)	Pit Dewatering
1a	1 x 10^{-6} m/s for WRD and Basal Drain	0.38	0.16	Full Drawdown
1b	1 x 10^{-5} m/s for WRD and Basal Drain	0.38	0.16	
1c	1 x 10^{-5} m/s for WRD; 1 x 10^{-6} m/s for Basal Drain	0.38	0.16	
1d	1 x 10^{-6} m/s for WRD; 1 x 10^{-5} m/s for Basal Drain	0.38	0.16	
1e	5 x 10^{-5} m/s for WRD; 1 x 10^{-5} m/s for Basal Drain	0.38	0.16	
2a	1 x 10^{-5} m/s for WRD and Basal Drain	0.38	0.16	10 m Drawdown
2b	5 x 10^{-5} m/s for WRD; 1 x 10^{-5} m/s for Basal Drain	0.38	0.16	

3.0 Results

The predicted drawdowns vs. time in the WRD in the scenarios are shown in **Figures 2 to 8**. The predicted hydraulic heads vs. time at the WRD base (below the crest) in the scenarios are shown in **Figures 9 and 10**.

The results indicate that the predicted drawdowns and hydraulic heads vary significantly with the permeability of the materials in the WRD and the basal drain (the bedrock beneath). In another word, the time to be required for dewatering the WRD highly depends on the permeability of the materials in the units. For the case where the saturated hydraulic conductivity is 5 x 10^{-5} m/s for the WRD and 1 x 10^{-5} m/s for the basal drain, the model predicts that it would take 40-50 years (in the full pit drawdown scenario) or 4-5 years (in the 10 m pit drawdown scenario), for the WRD to be dewatered.

It should be mentioned that the model predictions should also more or less be affected by other hydraulic parameters such as volumetric water contents and recharge, which are not investigated during the simulations.

Overall, the results show that dewatering the WRD under natural drainage conditions would take years. Therefore, other options such as active dewatering of the dump by using pumping wells may be needed.

4.0 Closure

We trust that the results presented above will satisfy the needs for the project. Please let us know for questions.

Sincerely,

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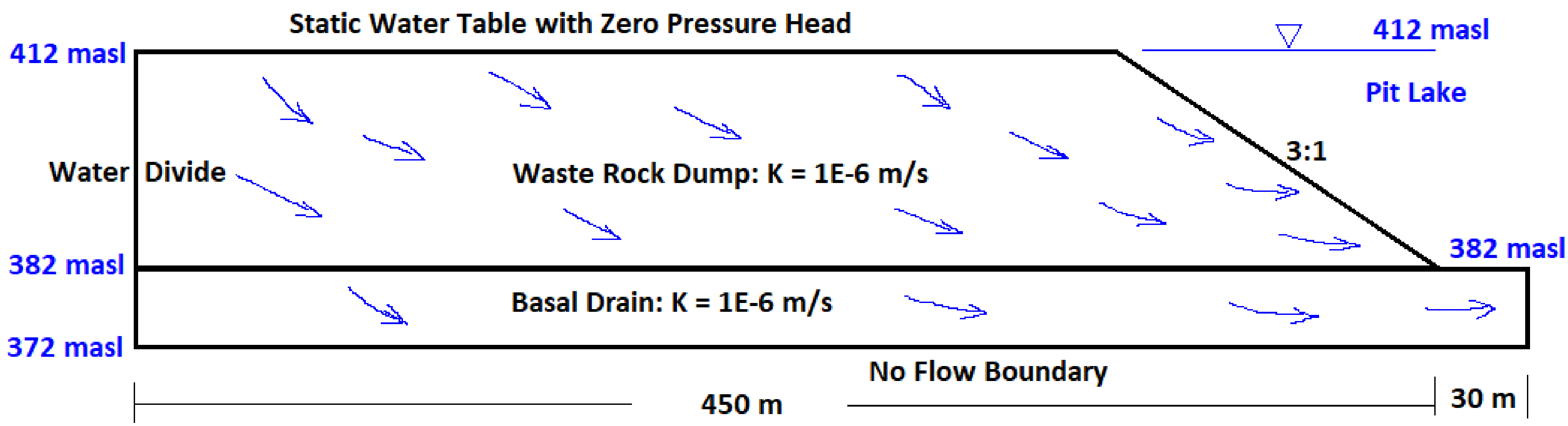

Figure 1: **Conceptual Groundwater Flow System**

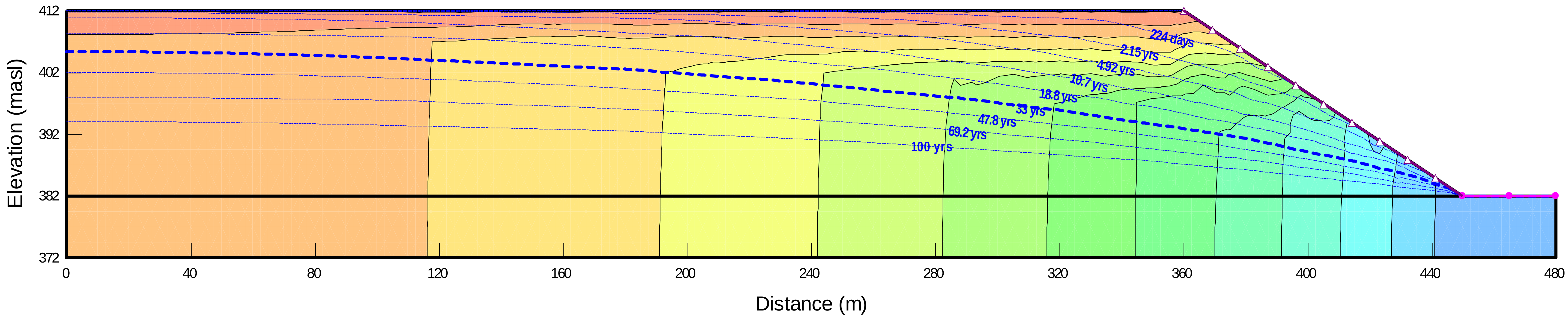

Figure 2: Predicted Drawdown vs. Time in Scenario 1a: Full Pit Drawdown (Ks = 1 x 10^{-6} m/s for WRD and Basal Drain)

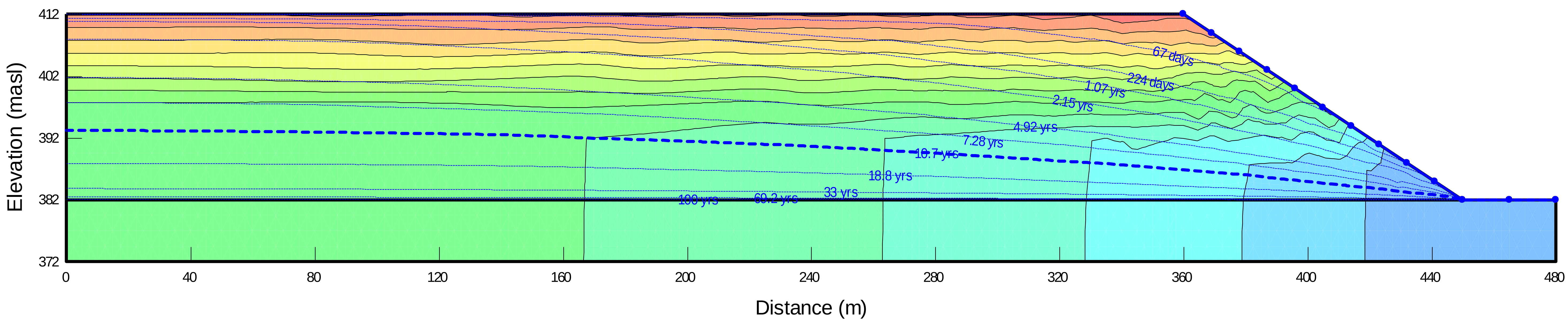

Figure 3: Predicted Drawdown vs. Time in Scenario 1b: Full Pit Drawdown (Ks = 1 x 10^{-5} m/s for WRD and Basal Drain)

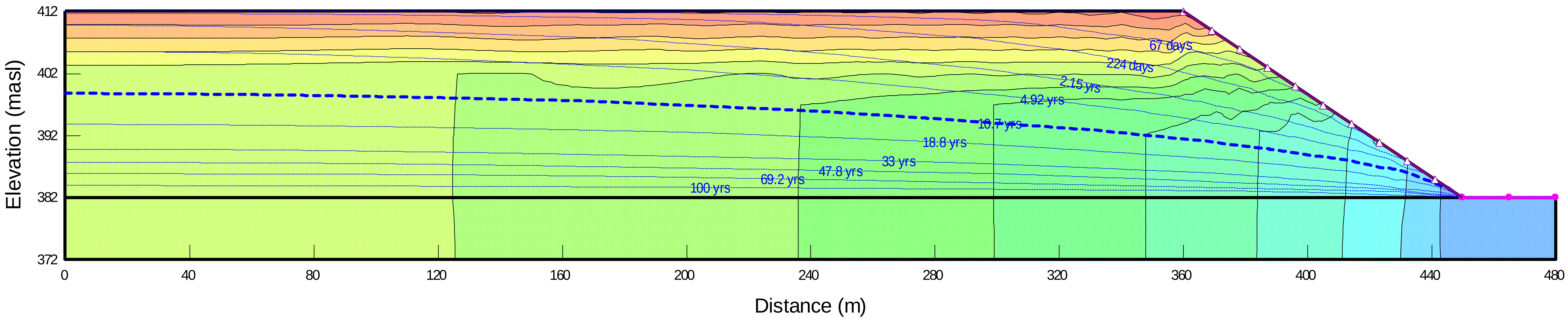

Figure 4: **Predicted Drawdown vs. Time in Scenario 1c: Full Pit Drawdown (Ks = 1 x 10^{-5} m/s for WRD, 1 x 10^{-6} m/s for Basal Drain)**

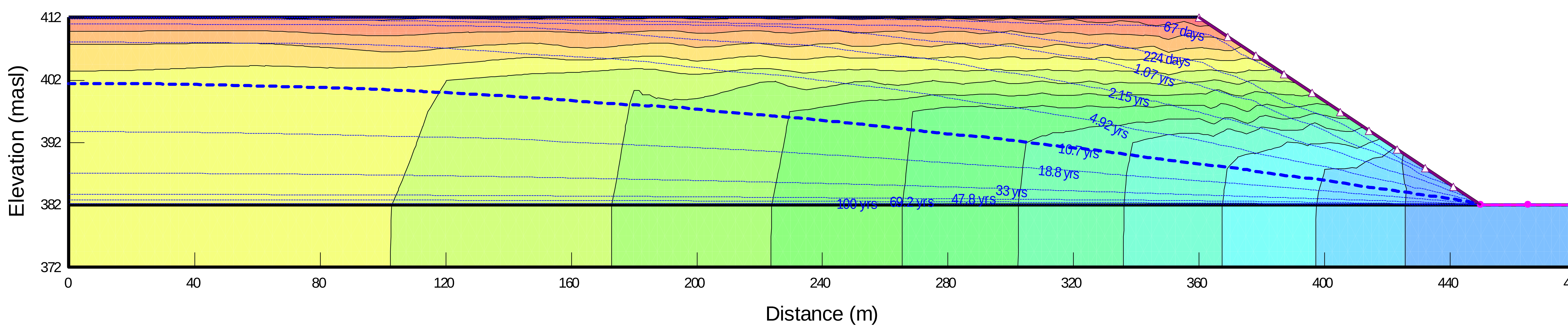

Figure 5: **Predicted Drawdown vs. Time in Scenario 1d: Full Pit Drawdown (Ks = 1 x 10^{-6} m/s for WRD, 1 x 10^{-5} m/s for Basal Drain)**

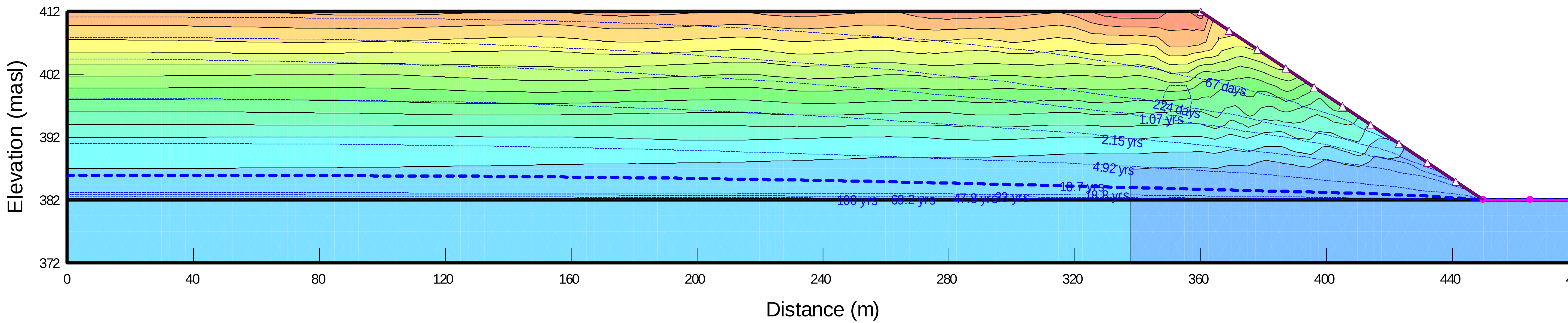

Figure 6: **Predicted Drawdown vs. Time in Scenario 1e: Full Pit Drawdown (Ks = 5 x 10^{-5} m/s for WRD, 1 x 10^{-5} m/s for Basal Drain)**

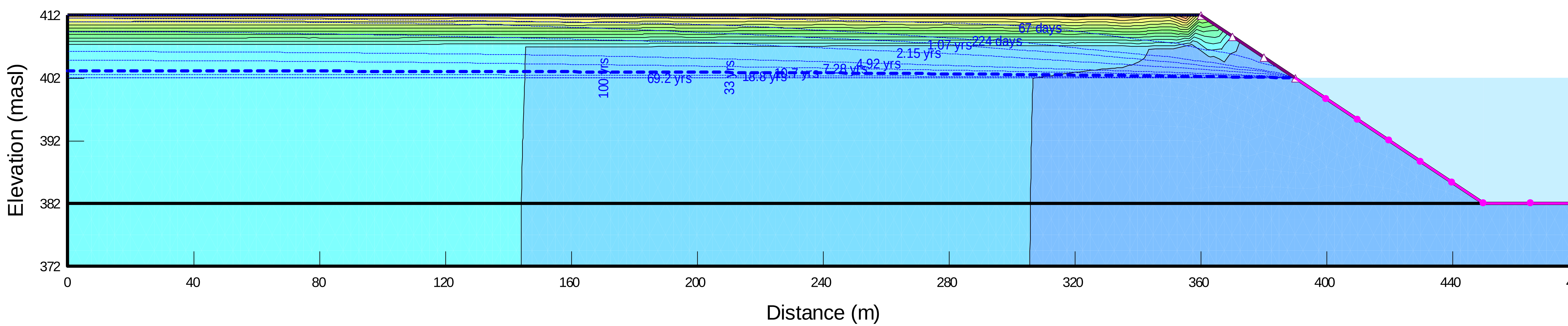

Figure 7: **Predicted Drawdown vs. Time in Scenario 2a: 10m Pit Drawdown (Ks = 1 x 10^{-5} m/s for WRD and Basal Drain)**

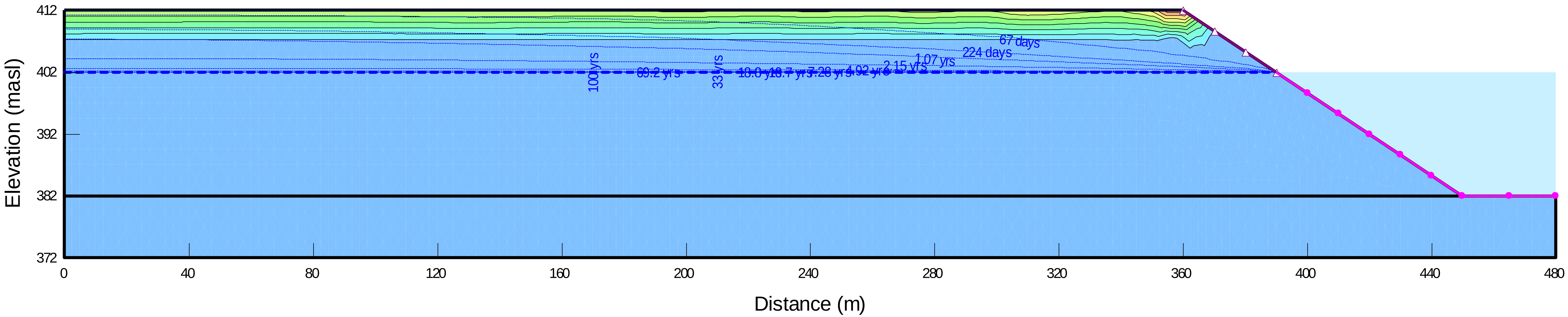

Figure 8: **Predicted Drawdown vs. Time in Scenario 2b: 10m Pit Drawdown (Ks = 5 x 10^{-5} m/s for WRD, 1 x 10^{-5} m/s for Basal Drain)**

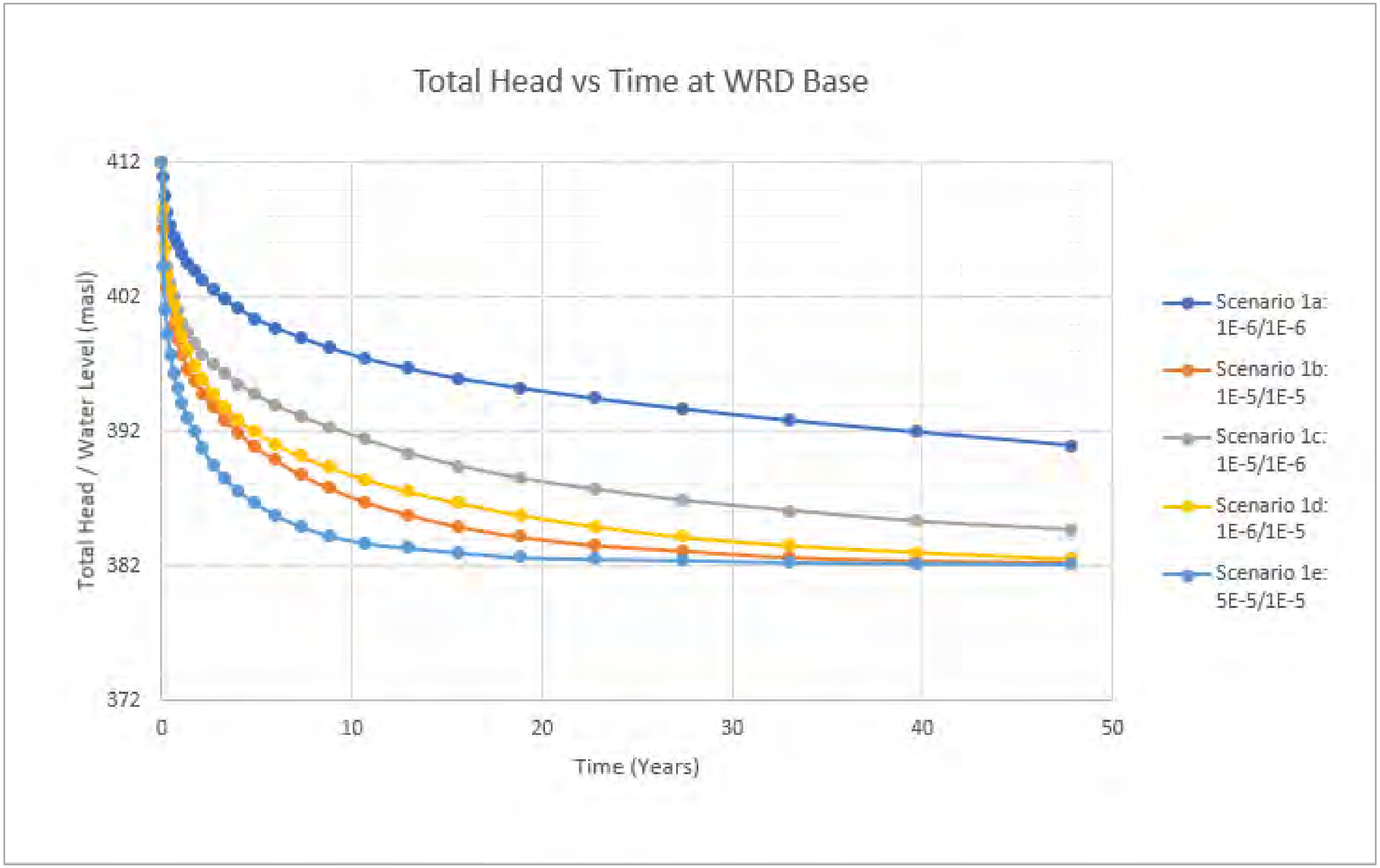

Figure 9: **Predicted Total Head vs. Time at WRD Base in Scenario1 (a, b, c, d, e): Full Pit Drawdown (Note: values e.g. 5E-5/1E-5 in the legend stands for saturated hydraulic conductivities in m/s of WRD/Basal Drain)**

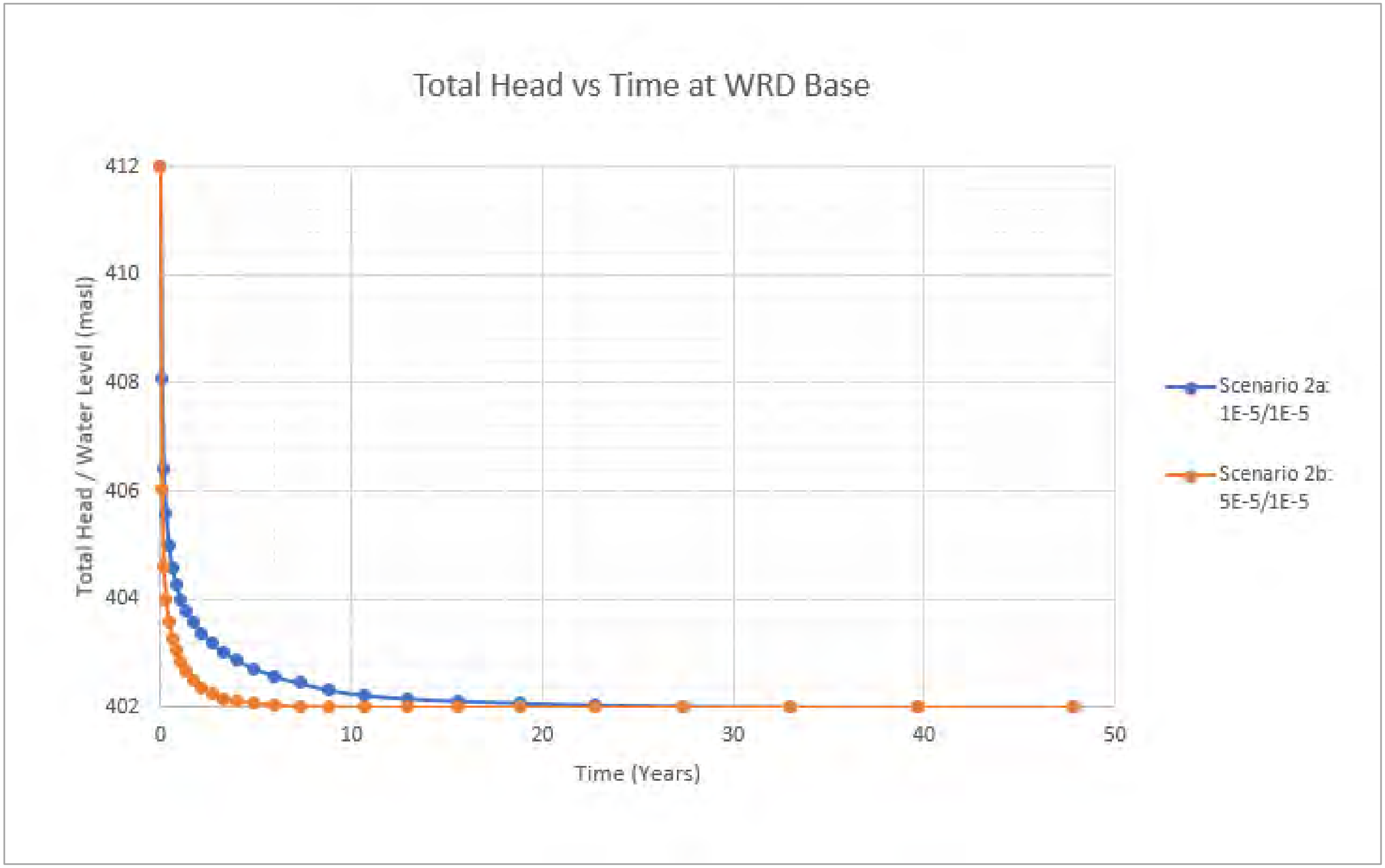

Figure 10: **Predicted Total Head vs. Time at WRD Base in Scenario 2 (a, b): 10m Pit Drawdown**
(Note: values e.g. 5E-5/1E-5 in the legend stands for saturated hydraulic conductivities in m/s of WRD/Basal Drain)

Appendix D
Geothermal Analyses for Spillway and Dump

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Memo

To:	VE52705D		
From:	Alexandre Tchekhovski, PhD, PEng	**Reviewer:**	Ed McRoberts, PhD, PEng
cc:	Karen Hincks, MSc, PGeo	**Wood File No.:**	VE52705D
Date:	9 August 2019		
Re:	**Geothermal Analyses for Spillway and Dump**		

1.0 Introduction

- The memo has been issued by Wood Environment & Infrastructure, a Division of Wood Canada Limited (Wood) to the Government of Yukon in regard to the Clinton Creek Asbestos Mine, located approximately 75 kilometers northwest of Dawson, YK. The purpose of geothermal analyses was to determine the thaw settlement rate for ice material which was found in several boreholes during performing the 2018 fill program. Analysis herein follows from previous geothermal work reported in Amec Foster Wheeler's Geotechnical Design Gaps (March 2018) Appendix "Geothermal Analysis Waste Dumps & Tailings Deposits". Geothermal analyses in the present memo are based on collected an appropriate field, laboratory and published information on permafrost, hydrological and climate conditions of the site.

2.0 Scope of Work

The scope of the work for the present memo has considered the following tasks

- Analysis of published data on temperature conditions of the Hudgeon Lake and spillway channel;
- Limited analysis of soil and thermal conditions from Wood's 2016 and 2018 field investigations;
- Using available data to obtain longitudinal topography of the spillway channel;
- Using a 2D geothermal model of the schematic stratigraphy with various water (ice) content under the spillway, to determine the relative impact of heat conduction from the surface water of the spillway and heat convection from the Hudgeon Lake to the settlement rate of thawing soils (both, the spillway and Hudgeon Lake will consider seasonal fluctuations of the water temperature);
- Using a 2D geothermal model of the schematic stratigraphy with various water (ice) content under the spillway and various boundary conditions to determine the relative impact of only heat conduction from the ground surface of the spillway and the Hudgeon Lake to the settlement rate of thawing soils (the Hudgeon Lake will consider seasonal fluctuations of the water temperature on the left-hand side of the model grid);
- Using a 1D geothermal model with a representative geothermal gradient in the dump to predict the settlement rate of thawing soil with various water (ice) content; and
- Analysis of the geothermal modeling.

3.0 Methodology

The thaw settlement rate (S=mm/year) was determined based on an analysis of soil temperature at the spillway and dump areas by the following formula:

$$S = U_{th} * R/100; \qquad (1)$$

Where:

U_{th} – thaw amount per year, mm;

R – strain rate, %.

The thaw amount was calculated based on results of temperature prediction. For the current study, 1 and 2-dimensional versions of SIMTEMP software (developed in-house by Ames Foster Wheeler) were used for soil temperature prediction. The program uses the finite element method to compute a numerical solution for the heat transfer problem. Physical/mathematical algorithms used in the SIMTEMP model have been published and the simulation process has been verified against well-known analytical solutions and with numerical solutions produced by other commercial/non-commercial geothermal modelling software. Wood has successfully used the SIMPTEMP program for a variety of geothermal applications over the last twenty years.

3.1 Methodology of Temperature Prediction

The two-dimensional equation for transient heat transfer was written as shown below.

$$C(T,x,y)\frac{dT}{dt} = k(T,x,y)\frac{\partial^2 T}{\partial x^2} + k(T,x,y)\frac{\partial^2 T}{\partial y^2}; \qquad (2)$$

Where:

C(T,x,y) – volumetric heat capacity;
k(T,x,y) – thermal conductivity;
T – temperature;
t – time.

Using Goodman's and Kirchoff's substitutes to the left and right portions of Equation 2 respectively, the non-linear equation is transformed to a quasi-linear equation which is written as follows:

$$\frac{dH}{dt} = \frac{\partial^2 F}{\partial x^2} + \frac{\partial^2 F}{\partial y^2}; \qquad (3)$$

Where:

H – enthalpy (Goodman's substitute);
F – temperature flux (Kirchoff's substitute).

Equation 3 is solved numerically using the finite element method in SIMTEMP software.

For heat assessment of water seeping into the waste dump, a convection term (Conv) was added to the left-hand side which is written as follows:

$$Conv = C_w V_w \frac{dT}{dl}; \qquad (4)$$

Where:

C_w – water heat capacity;
V_w – water velocity determined from Darcy's equation;
l – distance between neighbouring nodes of grid.

The finite element grid for the two-dimensional thermal analyses comprised 7738 nodes and 15080 finite elements. Figure 1, Appendix A, shows dimensions of the two-dimensional grid.

Equations 2 through 4 were used for prediction of soil temperature within the spillway area. A one-dimension versions of Equation 2 and 3 were used for prediction of soil temperature within the dump. The one –dimensional grid comprised 131 nodes and 130 finite elements. Figure 2, Appendix A, shows dimensions of the one-dimensional grid.

4.0 Schematic Soil Profile

For the two-dimensional thermal analyses, the schematic soil profile consisted of the 25 m thick layer of colluvium soil. For the majority of the colluvium soil, water (ice) content was assumed to be 18 percent. However, for a layer of the frozen soil extended at a distance of 170 m as shown in Figure 1, the water (ice) content was assumed to be 150 or 50 percent. Figure 1also shows that the thickness of the frozen layer was assumed to be 10 m.

For the one-dimensional thermal analyses, the schematic soil profile consisted of the 10 m thick layer of colluvium soil underlain with bedrock. Water (ice) content of the colluvium soil varied from 25 to 150 percent. Table 1 below provides physical and thermal properties of the schematic soils.

Table 1: Physical and Thermal properties of Identified Soils

Soil	Moisture Content,%	Thermal Cond., W/m/°K		Heat Capacity, MJ/m³/°K		Latent Heat, MJ/m³
		frozen	unfrozen	frozen	unfrozen	
Colluvium	18	1.44	1.35	2.344	2.847	108.522
	25	1.25	1.16	2.052	2.763	83.740
	35	1.38	1.29	2.345	3.098	105.512
	50	1.42	1.25	2.114	3.350	133.978
	150	1.55	0.86	2.721	3.768	200.966
Bedrock	2	2.15	2.15	2.512	2.512	18.422

5.0 Spillway Area

5.1 Boundary and Initial Conditions

Boundary temperatures on the top of the two-dimensional grid for the analyses with incorporated convection and for the analyses with no convection were presented by water temperatures in the spillway which were measured throughout the year months. These data include measured temperatures of water at 6 stations in the Hudgeon Lake and 3 stations along the spillway. Table 2 below provides water temperatures used as boundary conditions on the top of the spillway.

Table 2: Boundary Parameters on Spillway Surface

Data	Application	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Water temp., °C	Analyses with and with no convection	0.5	0.5	0.7	1.2	3.2	11.0	10.0	8.7	6 0	6.0	1.0	0.7
Air temp, °C	Analyses with no convection	-28.2	-25.9	-11.0	-0.6	10.0	17.2	19.7	16.1	9.7	-1.7	-13.4	-24.1
Snow thickness, cm	Analyses with no convection	42	50	47	43	---	---	---	---	---	9	24	38
Snow density, g/cm^3	Analyses with no convection	0.25	0.25	0.30	0.35	---	---	---	---	---	0.20	0.20	0.20

The analyses with no convection also were performed using air temperatures for calculation of boundary temperatures at the spillway surface. An air temperature history, which in details was discussed in the memo of 2018, was used to assess the mean monthly air temperatures. It was shown in the 2018 memo that the summer air temperature was warmed at a rate of 0.0082 °C/year and the winter months were warmed at a rate of 0.0176 °C/year for the analysed period of time (from 1950 to 2016). Applying these annual warming trends to the mean monthly air temperatures of 1950, the mean monthly air temperatures for the modeling were calculated (see Table 2). To obtain the spillway surface temperatures in summer months, n-factor of 1.2 was applied to the mean monthly air temperatures. It was also assumed in the model that due to low solar radiation amount, the snow surface temperature in winter months is equal to the mean monthly air temperatures.

The spillway surface temperatures in winter time ($T_{surface}$) were calculated by the following equation:

$$\frac{39.44(d_{snow}-0.06)(T_{air}-T_{surface})}{H_{snow}\left(14-0.5(T_{air}-T_{surface})\right)} = \frac{(T_{surface}-T_{node})k}{D} \qquad (5)$$

Where:

d_{snow} – snow density, g/cm^3 (see Table 2);

T_{air} – air temperature, °C (see Table 2);

H_{snow} – snow thickness, m (see Table 2);

T_{node} – soil temperature at depth D from spillway surface, °C;

D – distance between $T_{surface}$ and T_{node} , m;

k – soil thermal conductivity, W/m/°C.

On the left-hand side of the grid, mean monthly water temperatures measured in the Hudgeon Lake at various depths were used as boundary temperatures for all analyses. The background information used for assessment of these boundary temperatures was Figure 5 provided in the 2018 memo. These boundary temperatures were applied to the 25 m depth of the grid. Table 3 below provides water temperatures vs depth in the Hudgeon Lake used as boundary conditions on the left-hand side of the grid.

Table 3: Lake Boundary Temperatures (°C) on Left Hand Side of Grid

Depth, m	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
0-5	0.5	0.5	0.5	0.5	3.2	11.0	10.0	8.7	6 0	6.0	0.5	0.5
5-10	1.0	1.0	1.2	2.0	3.0	3.7	3.0	3.5	4.0	4.0	2.0	1.5
10-15	0.7	0.7	0.7	1.2	1.5	2.0	1.7	1.7	1.7	1.7	1.7	1.2
15-25	0.5	0.5	0.5	0.7	1.0	1.2	1.2	1.2	1.2	1.2	1.2	0.7

The zero-heat flux was applied on the left-hand side of the grid below the 25 m depth and over the entire right-hand side of the grid.

The heat flux of 0.043 W/m^2, which corresponds to the geothermal gradient in the bedrock of 0.02 °C/m, was applied on the bottom boundary of the two-dimensional grid.

The convective heat transfer term (Equation 4) was applied to the 25 m depth in the colluvium soil which have temperature above 0°C. The hydraulic conductivity of the colluvium soil was considered to be in the order of 10^{-6}m/sec to 10^{-5}m/sec. However, it was found by comparing results of the analyses with hydraulic conductivity of 10^{-6}m/sec with results of the analyses with no convection, that warming impact of the convection with such hydraulic conductivity is negligible. Thus, the results of the analyses in the present memo are provided only for hydraulic conductivity of 10^{-5}m/sec. For calculation of the water velocity in Equation 4, the hydraulic gradient in the Colluvium/alluvium soils was taken as 6.2 percent through the distance of 255 m from the Hudgeon Lake and the was reduced to 3 percent down to the right-hand side of the grid.

The Initial temperature of the bedrock and colluvium soil was above 0°C which gradually cooled down from 6.4 °C at the 270 m depth following the geothermal gradient of 0.02 °C/m. However, the identified colluvium layer with high water (ice) content (150 or 50 percent) had initially negative temperature of -0.05 °C. This layer was 170 m long (extended from 85 m to 255 m from the Hudgeon Lake) and was 10 m thick (extended from 15 m to 25 m below the spillway).

5.2 Results for Spillway with Convection

Table 4 below shows annual settlement and thaw together with the total settlement and thaw for icy colluvium soil with initial water (ice) content of 150 percent.

Table 4: Settlement and Thaw (mm) of Icy Soils within Spillway Area (150% water (ice) content and settlement strain rate 65%)

Year	Location					
	10 m from upstream edge of icy soil (see Fig. 1)		Middle of icy soil (see Fig. 1)		10 m from downstream edge of icy soil (see Fig. 1)	
	Settlement	Thaw	Settlement	Thaw	Settlement	Thaw
1	0	0*	0	0	0	0
2	130	200	65	100	65	100
3	130	200	65	100	65	100
4	130	200	65	100	65	100
5	130	200	130	200**	130	200
6	130	200	65	100	65	100
7	130	200	65	100	65	100
8	130	200	65	100	65	100
9	130	200	65	100	65	100
10	130	200	65	100	65	100
11	130	200	130	200	65	100
12	195	300	65	100	65	100
13	130	200	65	100	130	200
14	130	200	65	100	65	100
15	130	200	65	100	65	100
16	195	300	65	100	65	100
17	195	300	130	200	65	100
18	130	200	65	100	65	100
19	195	300	65	100	65	100
20	195	300	65	100	65	100
Total	**3055**	**4700**	**1495**	**2300**	**1430**	**2200**

***No** thawing was observed for the first year when boundary conditions of the model were adjusting to the initial conditions.

****Increased** annual thaw for some years are typical for used density of the grid in Y-direction (100 mm between neighboring nodes).

Analysis of Table 4 demonstrates that annual settlement and thaw are considerably greater near the upstream edge of the icy soils where warming influence of seeping water from the Hudgeon Lake is more noticeable. Comparing the total settlement, it can be stated that the amount of settlement near the upstream edge of the icy soil is approximately 3000 mm, while near the middle and the downstream edge of the icy soil, the total settlement is more than twice less (approximately 1450 mm).

Table 5 below shows annual settlement and thaw together with the total settlement and thaw for icy colluvium soil with initial water (ice) content of 50 percent.

Table 5: Settlement and Thaw (mm) of Icy Soils within Spillway Area (50% water (ice) content and settlement strain rate 30%)

Year	Location					
	10 m from upstream edge of icy soil (see Fig. 1)		Middle of icy soil (see Fig. 1)		10 m from downstream edge of icy soil (see Fig. 1)	
	Settlement	Thaw	Settlement	Thaw	Settlement	Thaw
1	30	100*	30	100	30	100
2	60	200	30	100	30	100
3	90	300	60	200	60	200
4	90	300	60	200	60	200
5	90	300	30	100	30	100
6	90	300	60	200	60	200
7	120	400**	60	200	60	200
8	90	300	30	100	30	100
9	120	400	60	200	60	200
10	120	400	60	200	30	100
11	120	400	60	200	60	200
12	120	400	30	100	60	200
13	120	400	60	200	30	100
14	150	500	60	200	30	100
15	120	400	30	100	60	200
16	150	500	60	200	60	200
17	120	400	60	200	60	200
18	120	400	60	200	30	100
19	120	400	30	100	60	200
20	120	400	60	200	30	100
Total	**2160**	**7200**	**990**	**3300**	**930**	**3100**

*****Low thawing was observed for the first 1-2 years when boundary conditions of the model were adjusting to the initial conditions.

******Increased annual thaw for some years are typical for used density of the grid in Y-direction (100 mm between neighboring nodes).

Similar to results of the analyses shown in Table 4, Table 5 demonstrates that annual settlement and thaw for ice soils at water (ice) content of 50 percent are considerably greater near the upstream edge of the icy soils where warming influence of seeping water from the Hudgeon Lake is more noticeable. Comparing the total settlement, it can be stated that the amount of settlement near the upstream edge of the icy soil is approximately 2160 mm, while near the middle and the downstream edge of the icy soil, the total settlement is more than twice less (approximately 950 mm).

It can easily be calculated that the total thaw in Table 5 (water (ice) content 50 percent) is approximately 1.4-1.5 times greater than that in Table 4 (water (ice) content 150 percent). Such considerable difference is due to lower latent heat which is required to thaw frozen soil with lower water (ice) content (see Table 1 above).

5.3 Results for Spillway with no Convection

Following 1-2 years required for adjustment of the boundary conditions to the initial conditions, the results have demonstrated that the thaw rate for frozen material of 150 and 50 percent of water (ice) content was stable for the 20 years. Table 6 below summarizes the results of modeling.

Table 6: Thaw and Settlement Rate (mm/year) of Icy Soils within Spillway Area

Water content, %	Water temperature as boundary temperatures (Table 2)		Air temperature as boundary temperatures (Table 2)	
	Thaw rate, mm/year	Settlement rate, mm/year	Thaw rate, mm/year	Settlement rate, mm/year
150	105	68	100	65
50	158	47	130	39

Table 6 demonstrates that the thaw rate for the soil at water (ice) content of 150 percent is in the range of 100-105 mm/year and the settlement rate is in the range of 65-68 mm/year. The thaw and settlement rates are slightly higher for the scenario when water temperatures in the spillway were applied as the boundary temperatures.

As it was expected, the thaw and settlement rates are higher for the soil at water (ice) content of 50 percent. Again, the thaw and settlement rates are slightly higher for the scenario when water temperatures in the spillway were applied as the boundary temperatures. The insignificant, generally, increase of the thaw rate for frozen soil at 50 percent of water (ice) content can be explained by the fact that the 50 percent frozen soil has lower thermal conductivity than that for the 150 percent frozen soil (compare 1.42 W/m/°K and 1.55 W/m/°K, see Table 1 above). This results that the 50 percent frozen soil warms up slower than the 150 percent frozen soil. Contrary, the 50 percent unfrozen soil has higher thermal conductivity than that for the 150 percent unfrozen soil (compare 1.25 W/m/°K and 0.86 W/m/°K, see Table 1 above). This results that the 50 percent unfrozen soil warms up faster than the 150 percent frozen soil.

6.0 Dump Area

6.1 Boundary and Initial Conditions

The heat flux of 0.102 W/m^2 which corresponds to an approximate gradient of 3°C/40 m as was measured in-situ (see Figure 8 provided in the 2018 memo. This is a good match to measured gradients in the Clinton Creek waste dumps.

The heat flux of 0.043 W/m^2, which corresponds to the geothermal gradient in the bedrock of 0.02 °C/m, was applied on the bottom boundary of the one-dimensional grid.

The initial temperature of the colluvium soil layer, 10 m thick, was -0.05 °C, while the initial temperature of bedrock was above 0 °C, gradually cooling down from 6.4 °C at the 270 m depth to 1.5 °C at the 10 m depth following the geothermal gradient of 0.02 °C/m.

6.2 Results for Dump

Results of the one-dimensional analyses with no influence of warming effect for seeping water from Hudgeon Lake have demonstrated that thawing of icy frozen soils located at depths 15-25 m is very slow process. The modeling has shown that there was no thawing for the first 9 years for the 150 percent frozen soil when the initial conditions were adjusting to the boundary conditions. For the following 11 years, the total thaw was only 100 mm, resulting in settlement rate of approximately 6mm/year.

For the 50 percent frozen soil, adjustment of initial conditions to the boundary conditions took only after 1 year. For the following 19 years, the total thaw was 800 mm, resulting in settlement rate of approximately 12.5 mm/year.

Similar one-dimensional analyses were also conducted for the frozen soils at moisture content of 25 and 35 percent. Results of all undertaken analyses for the dump is summarized in Table 7. It is interesting to analyze the thaw and settlement rate for the 4 identified soils. As it was expected, the thaw rate increases from approximately 9 mm/year to 76 mm/year as the water (ice) content of the frozen soil decreases from 150 to 25 percent. However, the settlement rate does not follow this uniformity, and the maximum settlement rate (12.6 mm/year) corresponds to the water (ice) content of 50 percent, and the minimum settlement rates correspond to the maximum and minimum water (ice) content (150 and 25 percent, respectively).

Table 7: Settlement and Thaw (mm) of Icy Soils for Dump with no Water Seepage

Water (ice) content, %	Adjustment of initial and boundary conditions, year	Total thaw, mm	Total settlement, mm	Thaw rate, mm/year	Settlement rate, mm/year
150	9	100	65	9.1	6.0
50	1	800	240	42.1	12.6
35	4	1000	150	62.5	9.4
25	3	1300	65	76.5	3.8

7.0 Conclusion

The results presented herein have confirmed the main conclusion provided in the 2018 geothermal memo that thermal process (thawing of buried frozen materials) under the various deposits is slow and gradual and likely takes a period of time over 20 years to complete thawing and corresponding settlement of the dump. This conclusion must be taken into account during a design phase of the dump remediation, unless special measures will be implemented to speed up thawing of the buried frozen materials or to stabilize its present spread. An international cold engineering practice has numerous examples of the implementation both of these methods for provision of stable and strong base for structure foundations.

8.0 Closure

Wood trusts that the information presented in this memo satisfies the current needs to provide a remediation design for the Clinton Creek dump. If there are questions or requests for additional information, please contact the undersigned at your convenience.

Sincerely,

Wood Environment & Infrastructure Solutions
a Division of Wood Canada Limited

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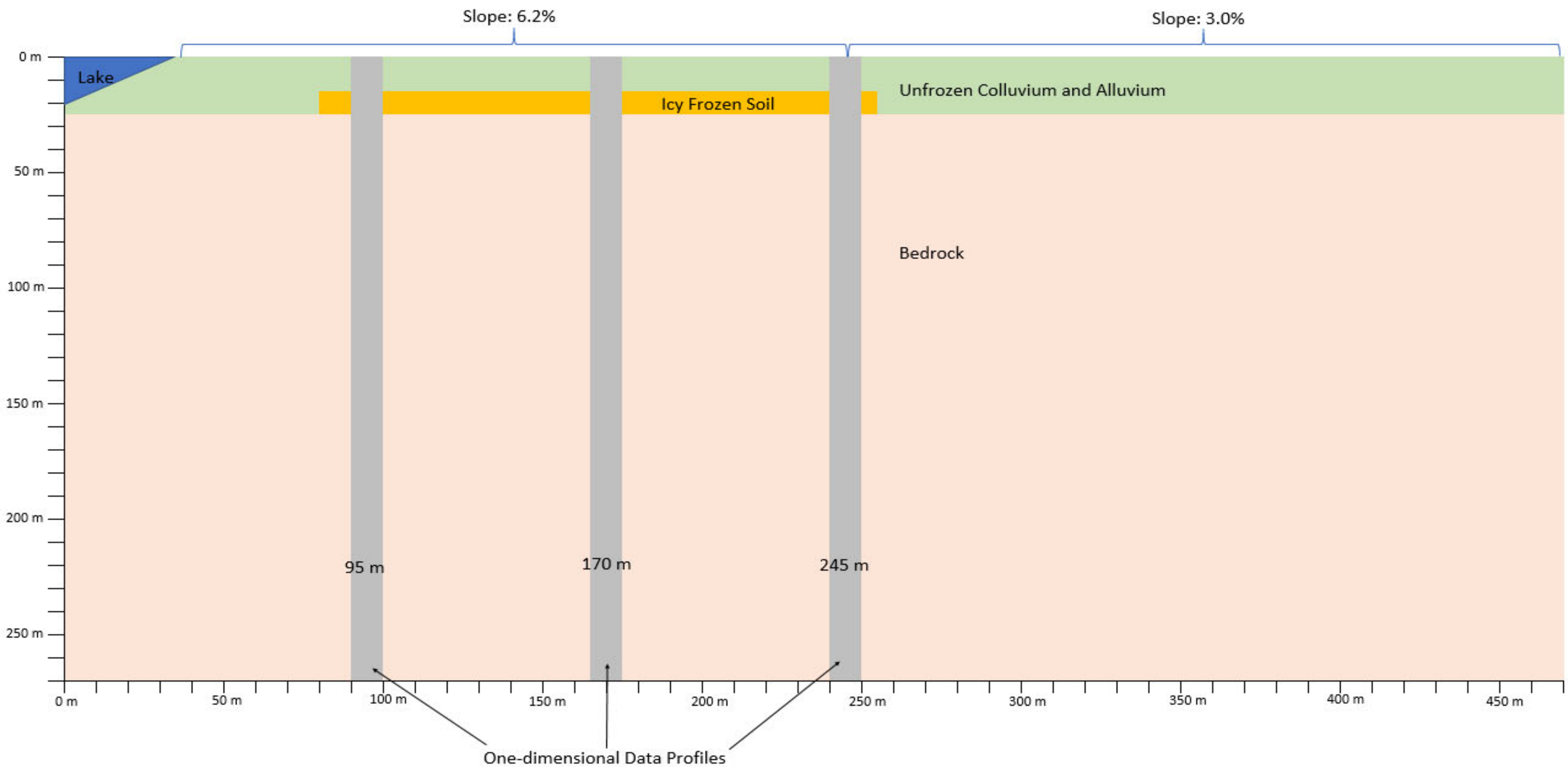
AT/rm

Attachment
Figures

Appendix A

Figures 1 & 2

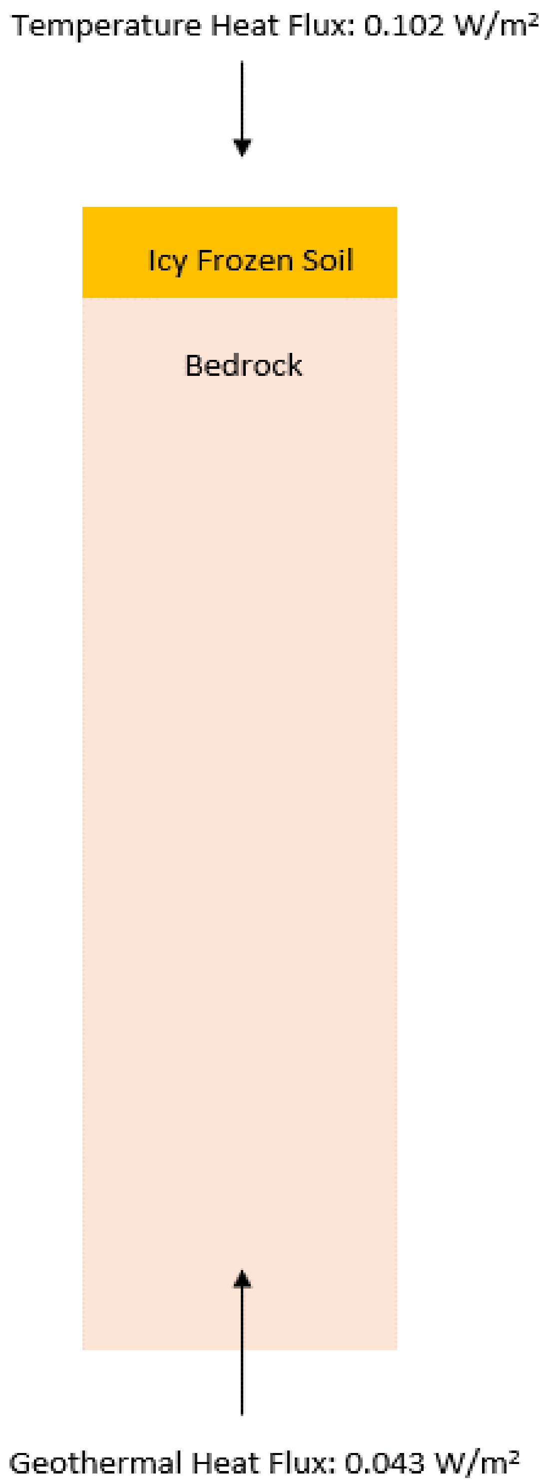


wood.

Wood Environment &
Infrastructure Solutions

CLIENT: Government of Yukon	PROJECT: Geothermal Analysis for Spillway and Dump
	TITLE: Cross Section for Two-Dimensional Thermal Analysis

DATE:	JOB No.:	FILE:	FIGURE No.:	REV.
March 2019	VE52705	VE52705D.100.4_Figure1_and_2_Rev 0.xlsx	1	0



Wood Environment & Infrastructure Solutions	PROJECT:	**Geothermal Analysis for Spillway and Dump**			
	TITLE:	**Cross Section for One-Dimensional Thermal Analysis**			
CLIENT: **Government of Yukon**	DATE: March 2019	JOB No.: VE52705	FILE: VE52705D.100.4_Figure1_and_2_Rev 0.xlsx	FIGURE No.: 2	REV. 0

Appendix E
Assessment of Liquefaction Triggering and Post Liquefaction Strength

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Memo

To:	File		
From:	Ed McRoberts PhD, PEng	**Reviewer**	Blair Gohl PhD, PEng (BC)
		Wood File No.:	VE52705D.100.2
Date:	26 August 2019		
Re:	**Assessment of Liquefaction Triggering and Post Liquefaction Strength**		

1.0 Introduction

1.1 Scope

This memo summarizes the methodology used to determine the potential for triggering liquefaction and post liquefaction strength to be adopted for use in analysis.

1.2 Reference Documents

The details of waste dump and tailings properties relevant to liquefaction potential are found in Wood (2019) Geological & Geotechnical Site Characterization and Model. A brief summary of insitu data is provided herein.

Appendix I to Wood (2019) provides a Design Basis Memorandum (DBM) - Geotechnical Criteria & Issues Memorandum which provides the seismic ground motions determined for use at the Clinton Creek mine site.

2.0 Insitu Data Summary

The tailings deposits were investigated by both LPT/SPT and CPT. A summary of the SPT N1(60) and CPT Qtn,cs test results are summarized in Figure 1. The SPT data is plotted relative to original ground and the CPT above refusal which is basically considered as the tailings / original ground interface. The Clinton Creek Waste Dump, forming the landslide dam and Hudgeon Lake, were only investigated by LPT/SPT as summarized in Figure 2.

Tailings N1(60) values over about 15 can generally be viewed as non-liquefiable and dilatant (frictional). Robertson (2010) considers that a Qtn,cs over 70 delineates a boundary between liquefiable and non-liquefiable. (Note: Robertson methodology predicts a Undrained Strength Ratio (USR) or s_{us}/σ'_v of about 0.25 at a Qtn,cs = 70 which is not a fully dilatant condition).

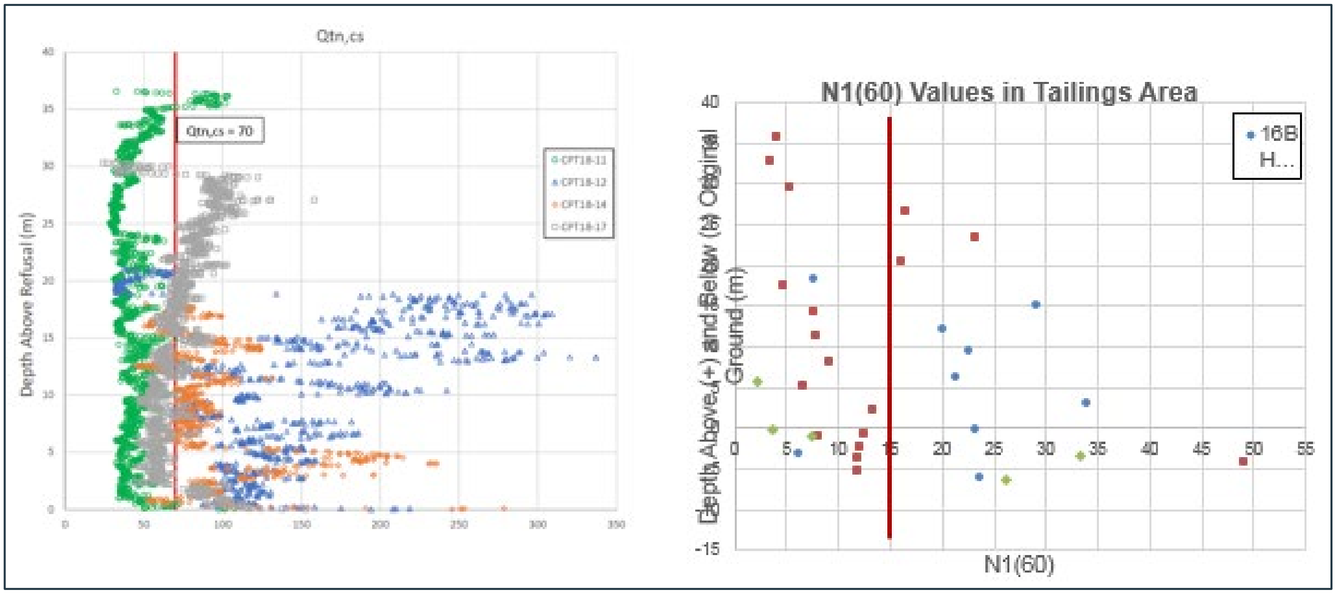
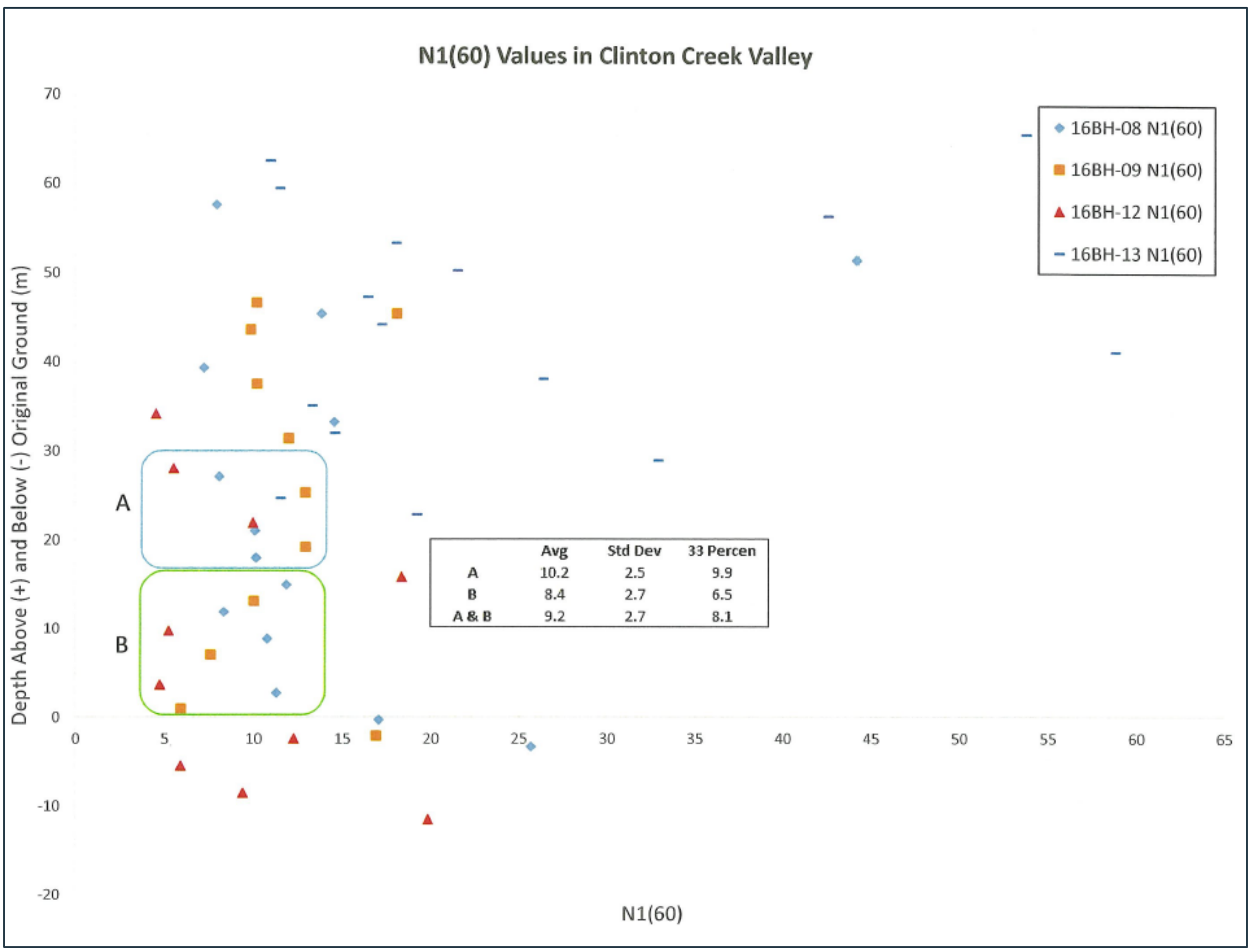


Figure 1: SPT & CPT Data Summary for Tailings in Wolverine Creek Valley



	Avg	Std Dev	33 Percen
A	10.2	2.5	9.9
B	8.4	2.7	6.5
A & B	9.2	2.7	8.1

Figure 2: SPT Data for Clinton Creek Valley Waste Landside Dam Waste Dumps

SPT interpretation for the Clinton Creek dumps focus on the looser range of data, in the anticipation that higher data could represent frozen zones, or interaction with coarse rock fragments. An average value of N1(60) = 8 has been assigned to the loose zone and is used for design purposes. Note that a lower bound N1(60) = 5, and a 33 Percentile value in the lower 10 m Zone B of Figure 2 = 6.5.

The LPT/SPT data of the tailings exhibits an average value of N1(60) = 8 for the loose zone, with a lower bound N1(60) = 5. A CPT Qtn,cs = 40 is considered representative of the upper tailings deposits. Data sets in the lower slope component of the tailings might be considered to be dominated by non-liquefiable tailings state; whereas the upper mass of the tailings is definitely liquefiable, if saturated or close to saturation. It is possible that the higher data in the lower slope could be due to the tailings being frozen, however, they will in the long term be assumed to thaw. Alternatively, efforts made during mining operations to stabilize the lower slope by benching and compaction may be reflected by the higher SPT and CPT results from some test locations in the lower slopes.

There was no CPT data obtained in the Clinton Creek Waste Dump due to damage concerns with boulder content. The SPT data required correction to transform LPT tests to the standard SPT values. It was found that the CPT methodology predction of the SPT state agreed with the adjustment of LPT to SPT in the tailings deposits, validating the adjustments used.

The CPT data formulated in terms of Qtn,cs embodies an internal correction for fine content to a "clean sand" equivalent within the methodology and no longer requires a fines correction. However, for the SPT data, a correction factor can be applied to account for fines content. A different correction for the Cyclic Stress Method (CSM) approach versus determination of the undrained strength ratio (USR) is recommended.

The fines content range (passing the #200 sieve) for waste dump materials is from 5% to 40% with a reasonable average being 30% and a sensible low fines content of 15%. Tailings fines contents range from 2% to 12% with an average of 8%.

3.0 Triggering

Triggering of liquefaction can occur due to the dynamic effect of cyclic ground motions due to earthquakes. Liquefaction can also be triggered due to so-called static loading conditions to differentiate this type of trigger with seismic events. While considerations of triggering may be very different, it is clear that the end result, that is that the liquefied shear strength that can be developed post triggering, does not depend on the initiation mechanism.

3.1 Cyclic Stress Method – Failed Waste Dumps

The CSM predicts the triggering of liquefaction based on interpretations of case records supplemented by laboratory testing. Herein we rely on the updates to the method as found in Idriss and Boulanger (2014).

Analyses have been undertaken using the following parameters:

- Design Magnitude of 6.2
- Peak acceleration at 2% in 50 years (1/2750) = 0.27
- Average fines content Fines content 30% correction ΔN = 5.5
- Design N1(60) equivalent clean sand of 8+ 5.5 = 13.5

The analysis for the Section DS3 on an East – West axis from the waste dumps into Hudgeon Lake has predicted a FOS_{liq} of between 0.54 near the dump toe to 0.45 at a 60 m crest depth. The general target for the CSM FOS_{liq} is typically 1.2 to 1.3 to reliably conclude that liquefaction will not be triggered.

It can be noted that adoption of a higher magnitude M of 7 and a 33-percentile level design SPT level would further reduce the predicted FOS_{liq}.

The analysis indicates that seismic liquefaction is a realistic threat, based on the current data base.

3.2 Tailings

The original (pre-construction) Wolverine Creek valley slope is about 200 m high. Prefailure ground slopes were in the order of 15 degrees in the upper slope and 20 degrees in the mid to lower slope. The break in slope occurs about 470 m a.s.l, or about 2/3 of the way down the slope. The shear stress to total stress ratio on a slope can be approximated by sin (slope angle) or 0.26 to 0.34. As the downstream slope of the upper loose tailings is in the order of 30 degrees, this may increase the insitu shear stresses along the inclined slope.

In consideration of the earthquake design motion (EQDM) of 0.27g at the 2%/ 50 years (1/2750), the peak horizontal loading will essentially equal the insitu shear stress, without the follow-on effect of cyclic loading embed in the CSM. Olson and Stark (2003) consider the yield strength ratio required to trigger liquefaction. For a N1(60) between 5 and 8, the average yield ratio to trigger is 0.24 to 0.27. Considering that the yield ratio is close to the static stress ratio, then the additional loading from even a smaller event indicates the susceptibility to liquefaction.

The significant issue for the upper slope is whether the tailings are essentially dry or if a relatively thin saturated zone could develop in the future, resulting in liquefaction being triggered.

3.3 Static Liquefaction

Static liquefaction as opposed to seismic (or dynamic) liquefaction is also a considerable threat to loose tailings. Triggering modes can include rapid construction, shear straining induced in loose granular deposits, and rising water tables in apparently stable slopes.

A rising water table may have impacted the original dump slopes as water was squeezed out of organic and muskeg type deposits in the Clinton Creek valley. A rising water table from Hudgeon Lake infilling may have impacted the significant slope failures facing Hudgeon Lake.

The loose tailings in the Wolverine Tailings Dump are also vulnerable to a rising water table. In fact, previous Amec Foster Wheeler reports have speculated on liquefaction being a mechanism in the original failures of the south and north lobes. The fact that additional failures have not occurred may point to unsaturated conditions in the tailings, which is not inconsistent with the available piezometeric data.

In the long term, if the water table rises, even by a few meters, in the existing tailings deposits, static liquefaction could be triggered.

The implications of a rapid drawdown event have not been analyzed. If pore pressures within the dump do not drop in response to a falling lake level, the dump contents may become unstable. This could also trigger a static liquefaction event.

3.4 Impact of Dump Operations on Waste Dump Liquefaction Susceptibility.

Generally speaking, waste dumps formed from strong rocks perform well. It is expected that this is the case for the quartz-muscovite schist or quartzitic rocks from the Porcupine and Snowshoe Pits. However, the argillite rock demonstrates considerable sensitivity to weathering and crushing. Dawson et al (1998) report on the mechanisms for flow sliding in the BC coal mines. Rocks in the coal measures, such as carbonaceous shales, can be similar to argillite. Dawson et al (1998) report that when dumps are operated such that shear straining is induced, the initially coarser rocks are crushed and ground-up creating fines, and a shear structure that, if saturated, will statically liquefy, generating an undrained strength ratio much less than the expected frictional response. Continued dumping at one location with a deposition mode that creates local internal shearing (rather than over the edge dumping) is one mechanism. At Clinton Creek, the initial dump movements may have been due to permafrost effects, but the subsequent failures then created a liquefaction susceptibility in the argillite dominated dumps.

4.0 Undrained Liquefied Shear Strength

Current practice formulates the post liquefaction strength by normalizing the back-calculated strength of a liquefaction flow with the initial vertical stress prior to the failure. This results in an USR or s_{us}/σ'_v.

The post liquefaction strength of contractant granular soils is considered to be independent of the triggering mechanism. The assessment of post liquefaction strength is typically based on case records of flow failures. There are three (3) methods available in the guidance literature that are presented In Appendix I:

1. **Olsen and Stark (2003)** For this method the USR is correlated with N1(60). This method considers that there is no reliable correlation with fines content correction, and only the stress level and energy corrections should be used.
2. **Idriss and Boulanger (2014)** For this method the USR is correlated with N1(60) cs. A correction is applied to the N1(60) based on fines content. This is not the same fines correction as used for the CSM.
3. **Robertson (2010)** For this method the USR is correlated with a parameter derived from CPT referred to as Qtn,cs. This method only uses cases records for which CPT data is available and the equivalent of a fines correction is made in the processing of the CPT data.

Table 1 summarizes the USR predictions based on the three methodologies. A summary of the design values selected for this phase of design are also given.

Table 1: Summary of Undrained Strength Ratio USR

Method	Waste Dump		Tailings	
	State	USR		
Oslen and Stark	N1(60) = 5 N1(60) = 8	0.07+/-0.03 0.09+/-0.03	N1(60) = 5 N1(60) = 8	0.07+/-0.03 0.09+/-0.03
Idriss and Boulanger	N1(60) cs = 8 N1(60) cs = 11	0.08 0.10	N1(60) cs = 6 N1(60) cs = 9	0.06 0.09
Robertson	No CPT		Qtn,cs = 40	0.05
DESIGN	**USR = 0.10**		**USR = 0.08**	

5.0 Movement Analysis

Seismic movement analyses are required for Closure Concepts WC2 and WC3, where the possibility of no liquefaction triggering is considered. The methodology of Bray and Tvarasarou (2007) is adopted.

Seismic movement analyses are not considered for the liquefied scenarios for WC2 and WC3, or for all CC options.

6.0 Conclusions

The conclusions are:

1. The failed dumps that have formed the landslide dams on both Clinton Creek, and probably Porcupine Creek, are saturated at the base and will likely liquefy when subject to the design seismic loading. They are also considered susceptible to static liquefaction during a rapid drawdown event.
2. The tailings piles on Wolverine Creek are susceptible to both triggering from static loading with a rising water table, and seismic loading.
3. Undrained strength ratio for use in post-liquefaction design remediation are provided.

Sincerely,

Wood Environment & Infrastructure Solutions
a Division of Wood Canada Limited

Prepared by:

Ed McRoberts, PhD, PEng
Principal Geotechnical Engineer
27 Aug 19

Reviewed by:

Blair Gohl, PhD, PEng (BC)
Principal Geotechnical Engineer

7.0 References

Bray J and Travasarou T (2007). *Simplified Procedures for Estimating Earthquake-Induced Deviatoric Slope Displacements.* JGGE, ASCE, April 2007.

Olsen S and Stark T (2003). *Yield Strength Ratio and Liquefaction Analysis of Slopes and Embankments*. JGGE ASCE August 2003.

Dawson R, Morgenstern N, and Stokes A. (1998). *Liquefaction flow slides in rocky Mountain Coal Mine Waste Dumps*. CGJ 35, 1998.

Idriss I. and Boulanger R. (2014). *CPT and SPT Liquefaction Triggering Procedures*. Report UCD/CGN-14/01. University of California at Davis.

Robertson P. (2010) *Evaluation of Flow Liquefaction and Liquefied Strength Using the Cone Penetration Test.* JGGE ASCE June 2010

Appendix I

Models Used in Determining Triggering & Post Liquification Strength

Models Used In Determining Triggering & Post Liquefaction Strength

Appendix I
Assessment of Liquefaction Triggering and Post Liquefaction Strength

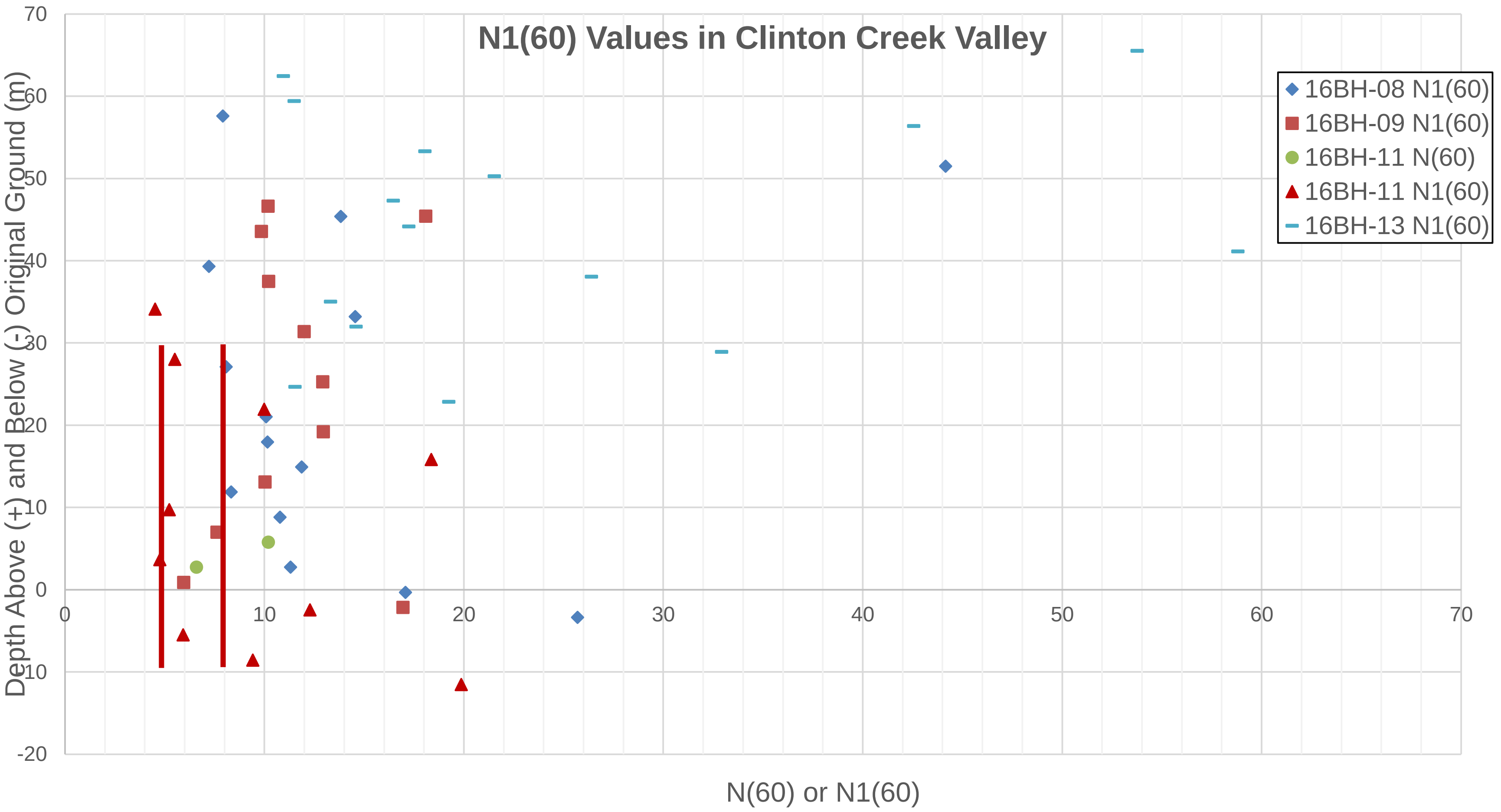
SEISMIC PARAMETERS

The significant seismic data for the site are:

The horizontal site acceleration of 0.27g at a 2%/50year hazard level to be used.

The NRCan deggregation predicts a M = 6.2 at 24 km. At a higher period magnitudes of up to 7 to 8 from more distant event can occur.

This data has been collected in 2016 in the Clinton Creek unstable waste dumps that formed the landslide dam on Clinton Creek. It is also considered to be a realistic model for the failed dumps in Porcupine Creek valley.



N1(60) avg in loose = 8

N1(60) lower bound = 5

Different methodologies can apply "corrections" to the N1,60 by adding a ΔN1,60 to obtain a N1,60-cs. "cs" designates clean sand equivalent.

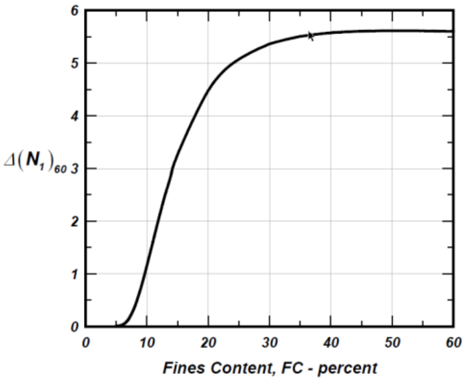
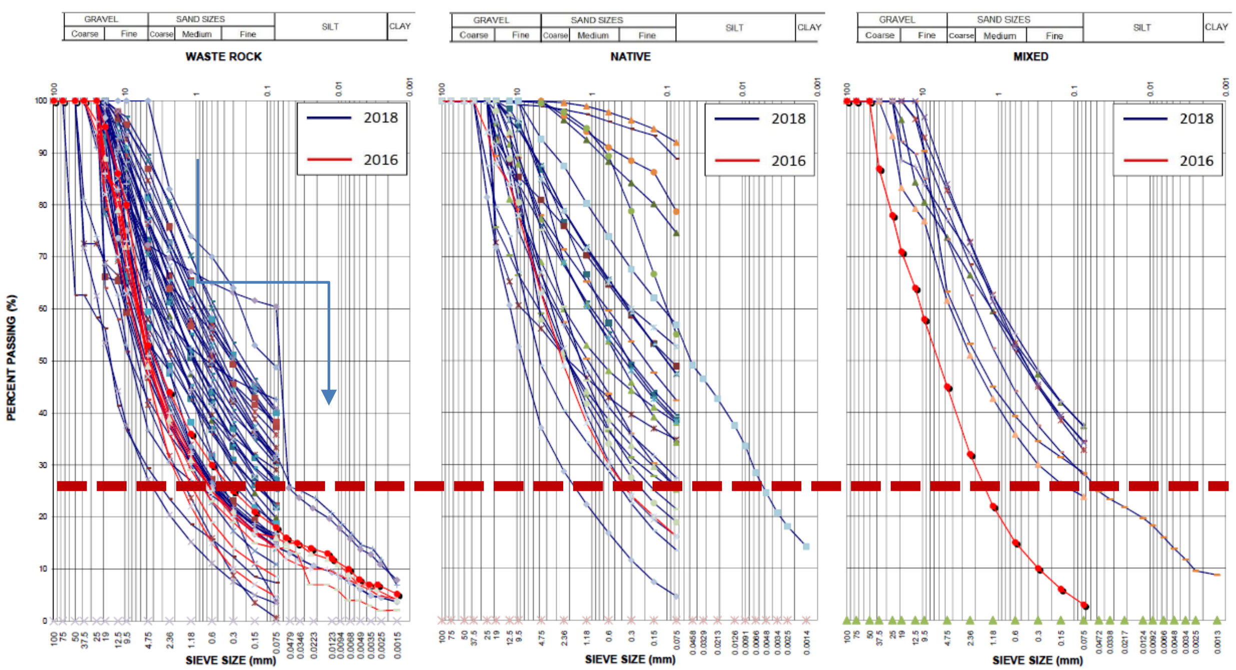


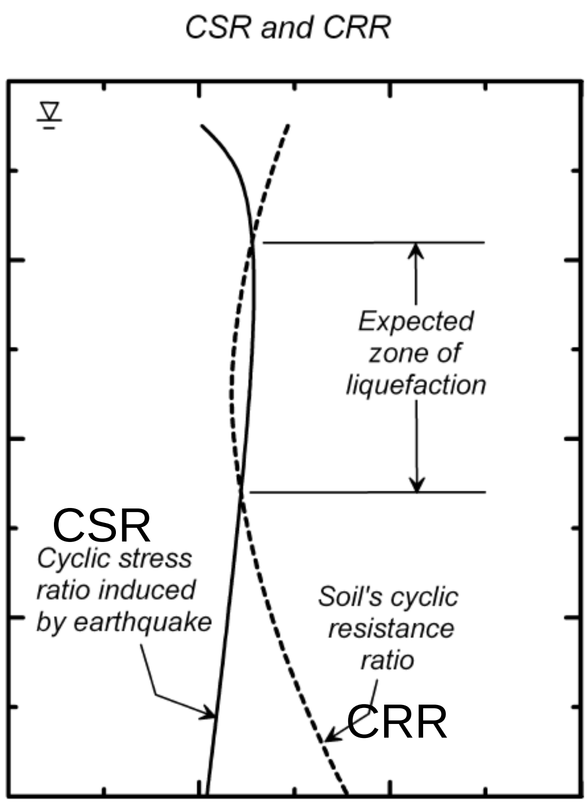
Fig. 19: Variation of $\Delta(N_1)_{60}$ with fines content.

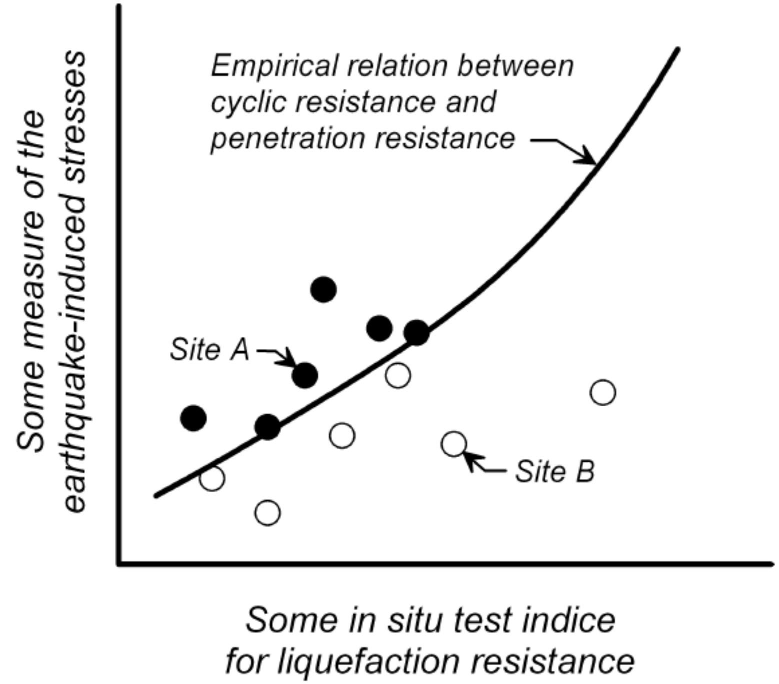
Fines content 30% correction ΔN = 5.5; N1(60)CS =8+5.5 = 13.5

Fines 15%, ΔN = 3: N1(60)CS = 5+3 = 8



Triggering Analysis Basics





Liquefaction is predicted if the CSR > CRR

Or: **CRR > FS_{liq} *(CSR)** to be "safe" against getting a liquefaction event

So basically predicting the "margin" between a site liquefying or not liquefying.

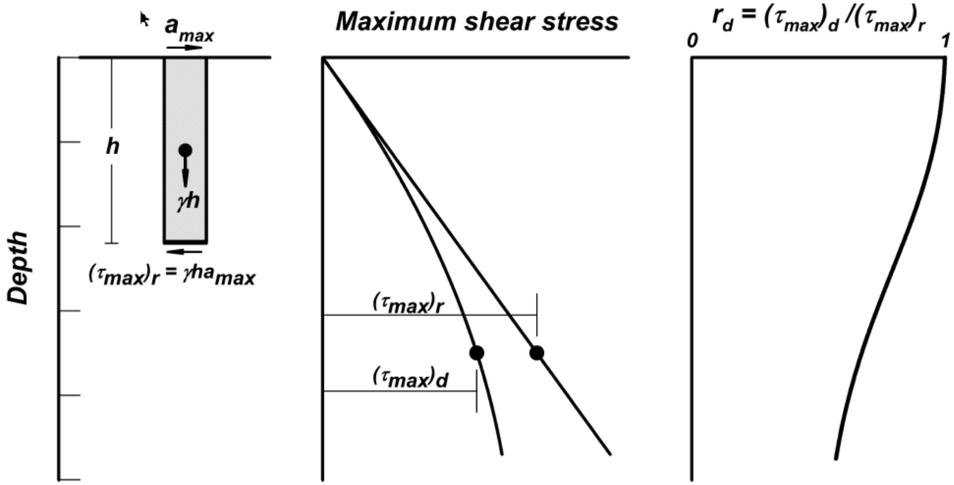


Figure 50. Schematic for determining maximum shear stress, $\tau_{\max}$, and the stress reduction coefficient, r_d.

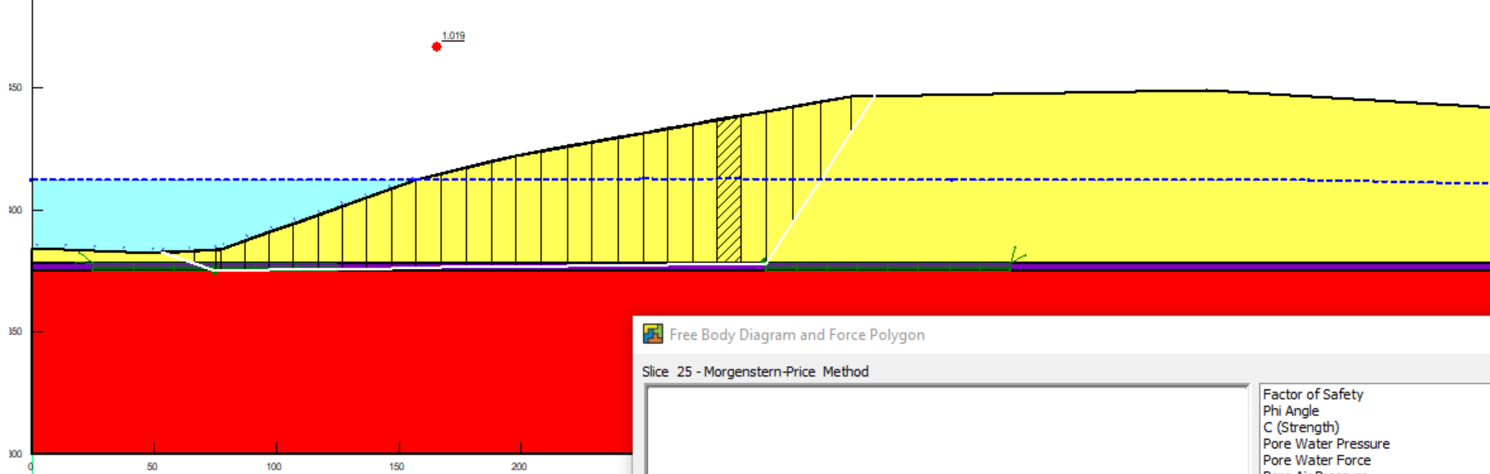
The **CSR** is the loading on the soil element in question.

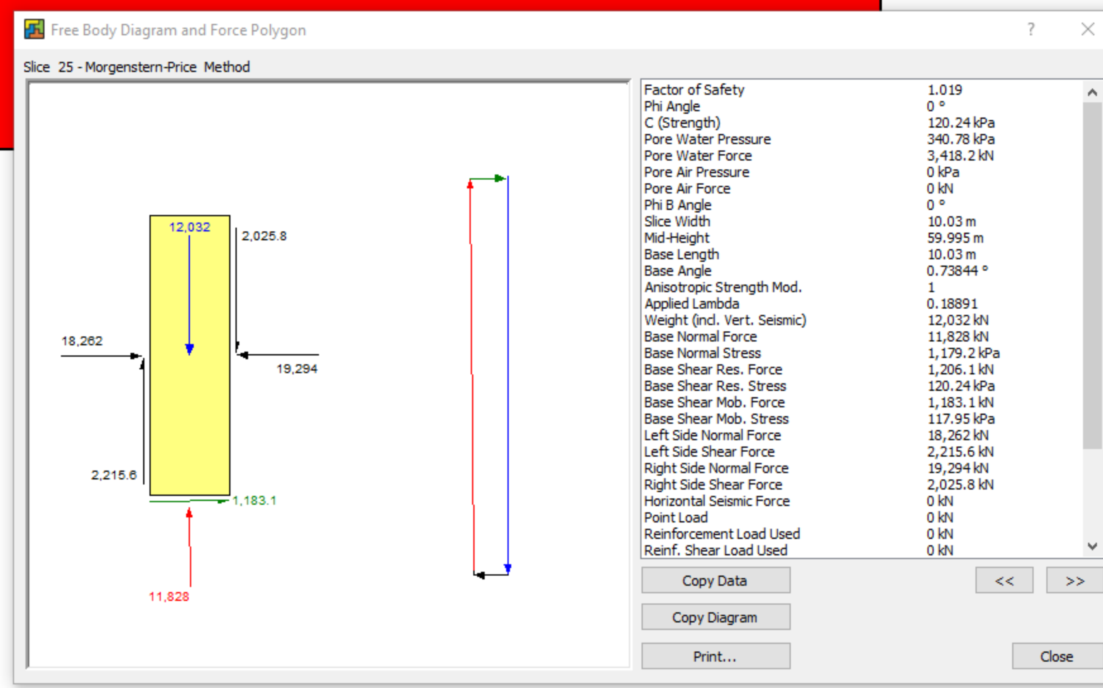
The loading term derives from the a_{max} or peak ground acceleration at the ground surface. For Clinton Creek $a_{max}/g = 0.27$ for Site Class C

$$\mathrm{CSR} = 0.65\frac{\tau_{\max}}{\sigma'_{vc}} = 0.65\frac{\sigma_{vc}}{\sigma'_{vc}}\frac{a_{\max}}{g}r_d$$

However, the r_d factor is derived for a FLAT surface, or at most gently sloping ground (spreading case). There is no guidance on this parameter for a significant slope.

Section DS-3 Hudgeon Lake as example





For slice 25:
Total stress = 1203 kPa
Effective stress = 861 kPa

CSR = $0.245r_d$

$$\mathrm{CSR} = 0.65\frac{\tau_{\max}}{\sigma'_{vc}} = 0.65\frac{\sigma_{vc}}{\sigma'_{vc}}\frac{a_{\max}}{g}r_d$$

CRR

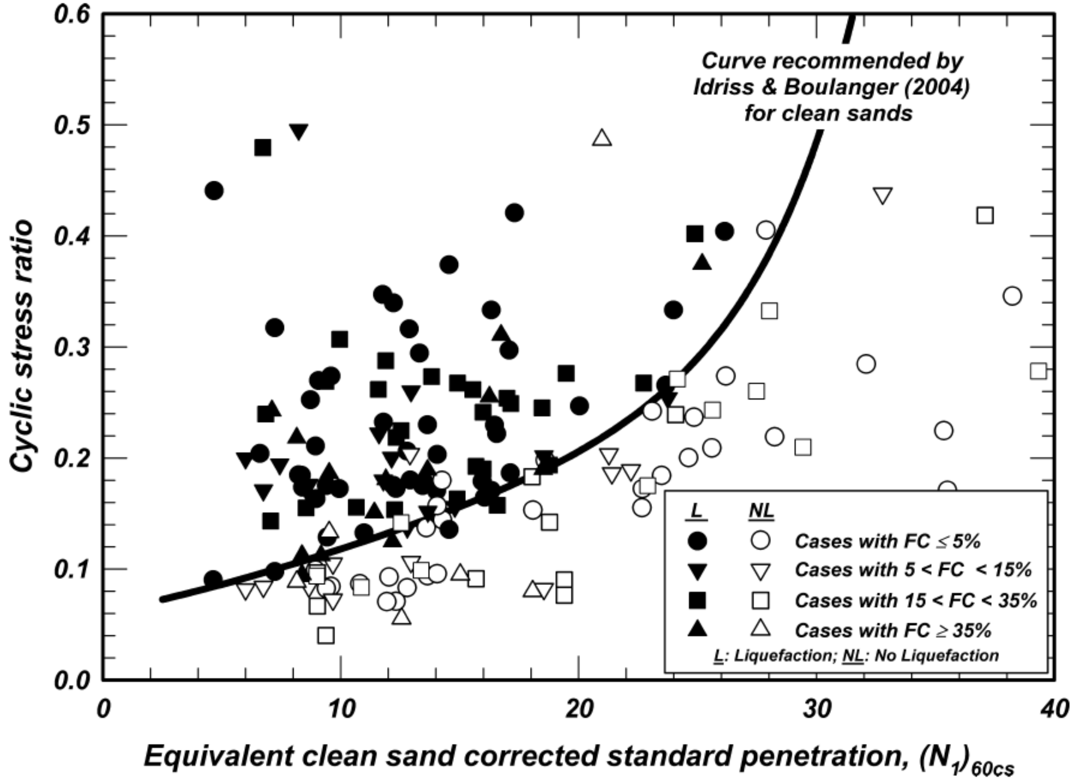


Figure 75. SPT case histories of liquefaction in cohesionless soils with various fines contents plotted versus their equivalent clean sand $(N_1)_{60cs}$ values for M = 7.5 and $\sigma'_{vc} = 1$ atm.

CRR M6.2 = CRR M7.5 x Kσ x Kα x MSF

Therefore:

- at N1(60)cs of 13.5 then CRR M6.2 = 0.14
- at N1(60)cs of 8 then CRR 6.2 = 0.11

MSF and **Kσ** Factors

1. **MSF:** Idriss and Boulanger (2008) MSF factor. These authors specifically note that Ambraseys and Arango methodology was different than current procedures and should not be used. Recommendation for MSF for 6.2 is = 1.41 in I&B (2008) not the 1.63 cited in spreadsheet.

Idriss and Boulanger (2014) update MSF in Figure 2.6 . For a N1,60cs = 13.5 a MSF = 1.1 is now recommended.

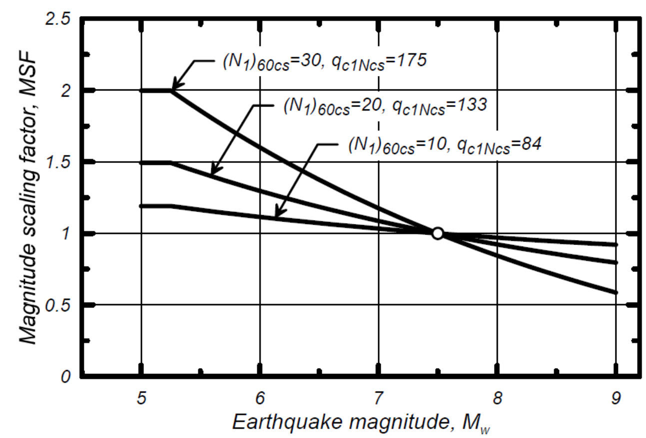
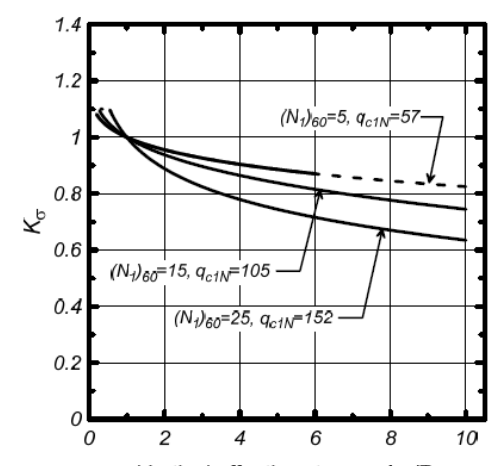


Figure 2.6. Variation in the MSF relationship with q_{c1Ncs} and with $(N_1)_{60cs}$ for cohesionless soils

Kσ: For effective stress of 800kPa and N1(60)cs = 13.5 use 0.8



Kα

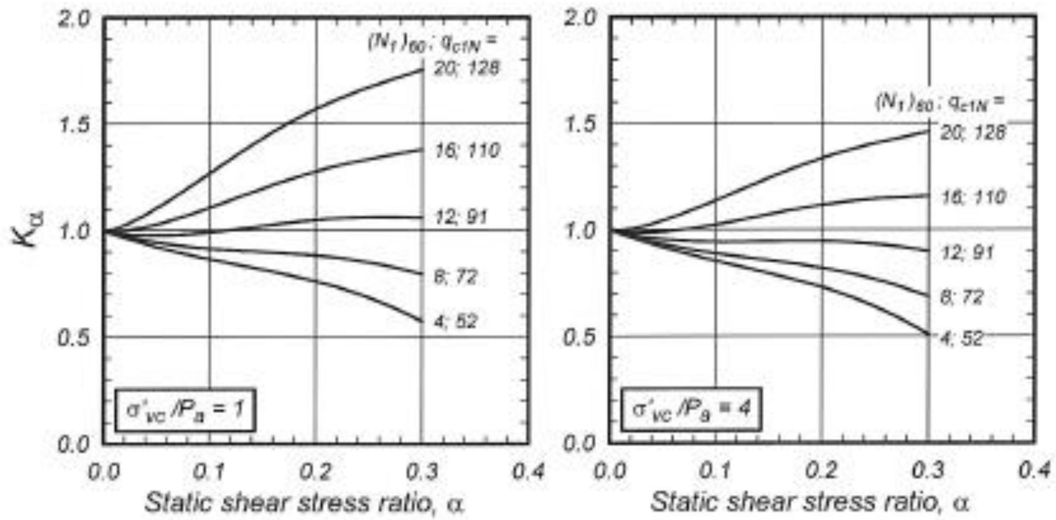


Figure 65. Variations of K_α with SPT and CPT penetration resistances at effective overburden stresses of 1 and 4 atm.

Kα correction not equal to 1.0 per spreadsheet for sloping ground. Static stress ratio for The DS-3 waste dump slope is about 0.15. a 60 M high slope element is "off the chart" for effective stress of 860 kPa. Adopting Kα correction of 0.9 is not unreasonable.

.

For selected element in DS-3 at 60 m depth facing Hudgeon Lake

FS_{liq} = CRR/CSR
CSR = 0.245r_d
CRR = 0.14 x 0.8 x 0.9 x1.1 = 0.11

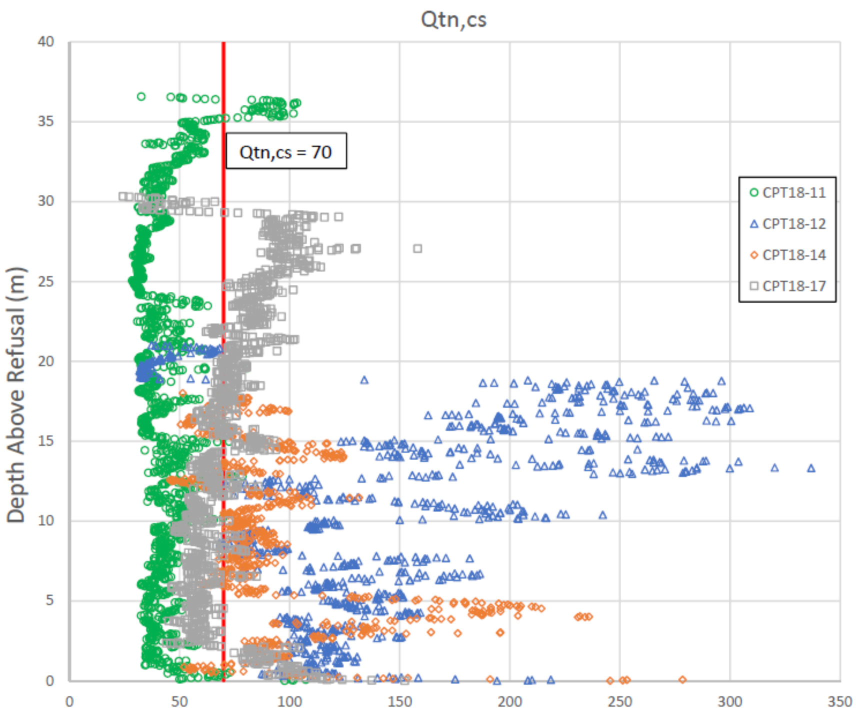
FS_{liq} = CRR/CSR = 0.11/ 0.245r_d = 0.45/r_d for avg loose SPTand 30% fines
FS_{liq} = 0.35/r_d for lower bound SPT and 15% fines

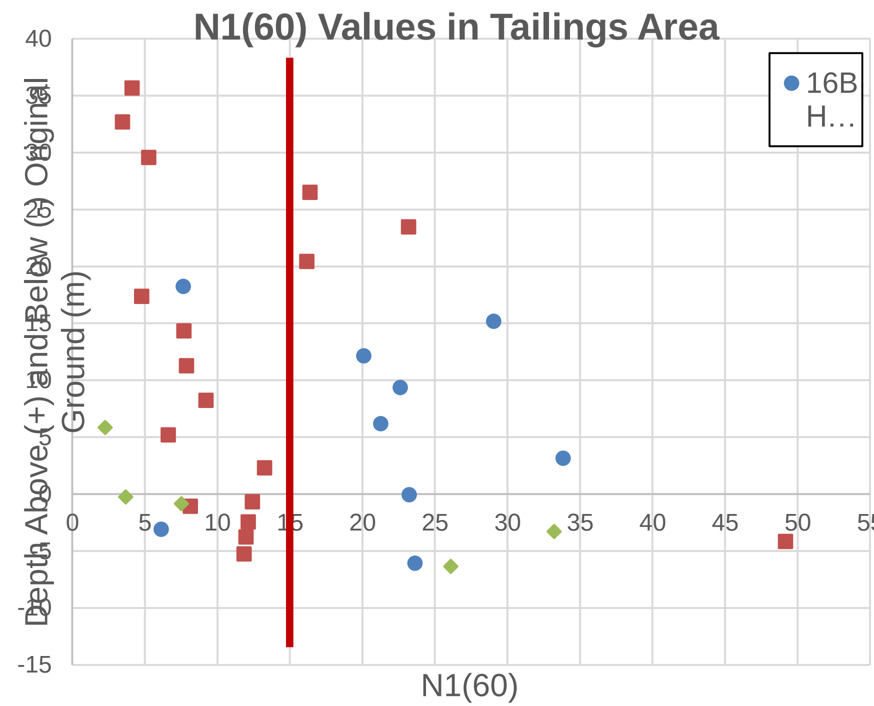
The issue of what "rd" parameter to use for an component of sloping ground is not well stated in the CSM literature. If the slope was moving as a rigid block this parameter is unity. For flat ground the r_d could be 0.5 or less.

The range in FS_{liq} could be from 0.35 to 0.90, given variability in operative sand state, fines content and the dynamic modification in driving stress given the stiffness of the slope interacting with the foundation.

A more detailed 2D analysis might increase the margin against liquefaction for dump slopes, but would unlikely result in the necessary margins to eliminate the risk of triggering due to seismic action.

TAILINGS DEPOSITS IN WOLVERINE CREEK VALLEY





Both SPT N1(60) and CPT qtn,cs data are available for the tailings. Sand state Qtn,cs = 70 and N1(60) are considered susceptible liquefaction. Consistently loose tailings is found in the upper slope, versus higher indicated state in the lower slope.

CONSIDERATION OF POST LIQUEFACTION STRENGTH

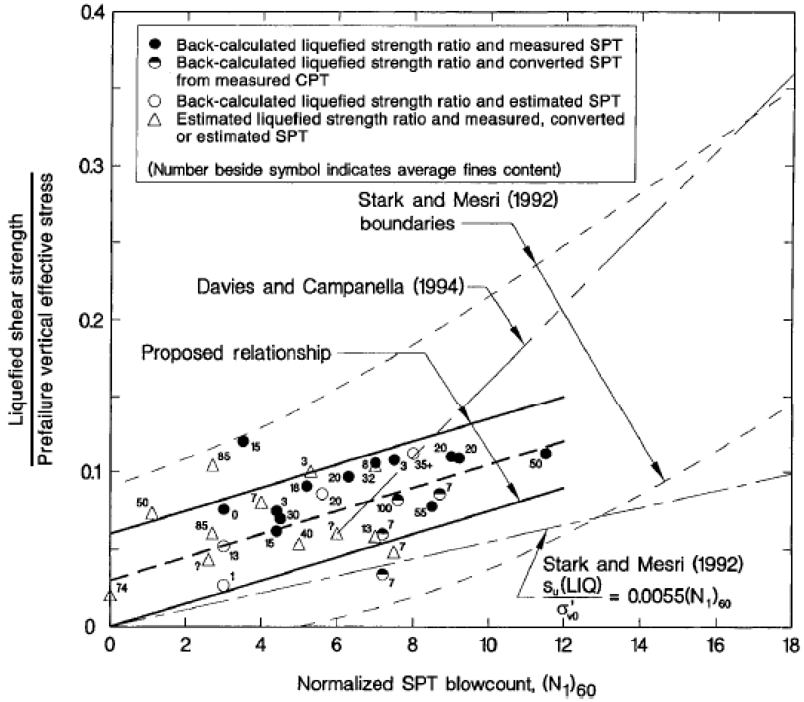
Three methods that have been summarized are based on

Olson and Stark (200

Idriss and Boulanger (2014)

Robertson (2010)

Fig. 6. A comparison of liquefied strength ratio relationships based on normalized SPT blowcount.



The Olson and Stark approach does NOT apply fines corrections.

No fines content adjustment was adopted in this study. In Figs. 5 and 6, the fines content of the liquefied soil is provided next to each data point. The data reveal no trend in liquefied strength ratio with respect to fines content. The authors anticipate that although soils with higher fines contents should exhibit lower values of penetration resistance (as a result of greater soil compressibility and smaller hydraulic conductivity), these soils are more likely to maintain an undrained condition during flow. The combination of these factors may, in effect, offset each other, resulting in no apparent difference in values of liquefied strength ratio for cases of clean sands and sands with higher fines contents. Therefore, this study recommends no fines content adjustment for estimating liquefied strength ratio from the proposed relationships.

$$\frac{s_u(\mathrm{LIQ})}{\sigma'_{vo}} = 0.03 + 0.0075[(N_1)_{60}] \pm 0.03$$

for $(N_1) \lesssim 12$

TAILINGS & WASTE DUMP

N1,60 = 5;USR = 0.07 +/- 0.03
N1,60 = 8;USR = 0.09 +/- 0.03

The Idriss and Boulanger (2014) methodology provides a fines correction (but as previously noted not the same one as is used for triggering analysis). For a tailings average fines of 8% the Δ N = 1. For waste dump 30% fines Δ N = 3

Table 2. Values of $\Delta(N_1)_{60\text{-}Sr}$ recommended by Seed (1987)

Fines Content, FC (% passing No. 200 sieve)	$\Delta(N_1)_{60\text{-}Sr}$
10	1
25	2
50	4
75	5

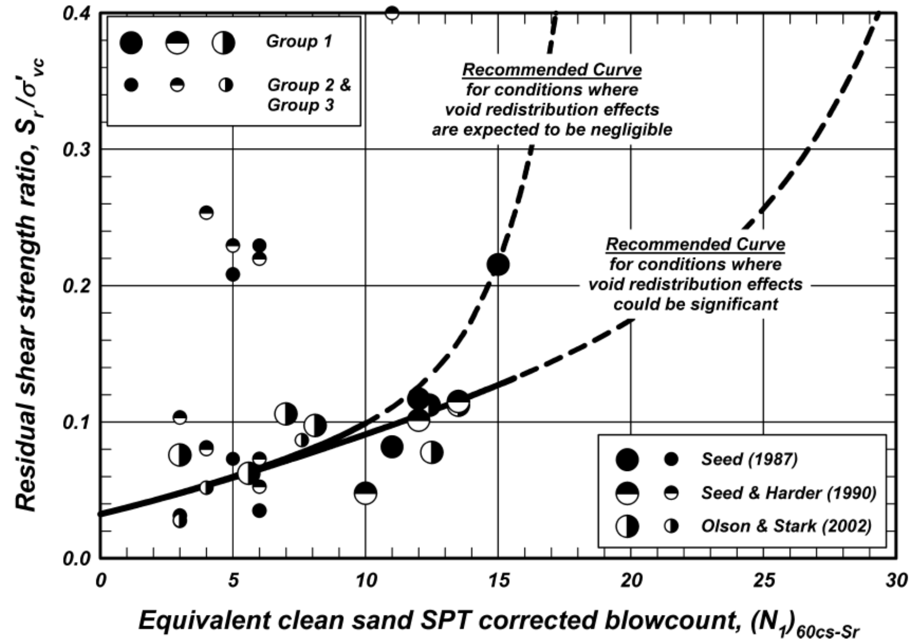


Figure 89. Normalized residual shear strength ratio of liquefied sand versus equivalent clean-sand SPT-corrected blow count for $\sigma'_{vc} < 400$ kPa, using the case histories published by Seed (1987), Seed and Harder (1990), and Olson and Stark (2002).

TAILINGS

For a Idriss and Boulanger version of Liquefaction back-analysis a N1,60 of 5 to 8 corrects to 6 to 9.

The resulting prediction is USR = 0.06 to 0.09

WASTE DUMPS

For a Idriss and Boulanger version of Liquefaction back-analysis a N1,60 of 5 to 8 corrects to 8 to 11.

The resulting prediction is USR = 0.08 to 0.10

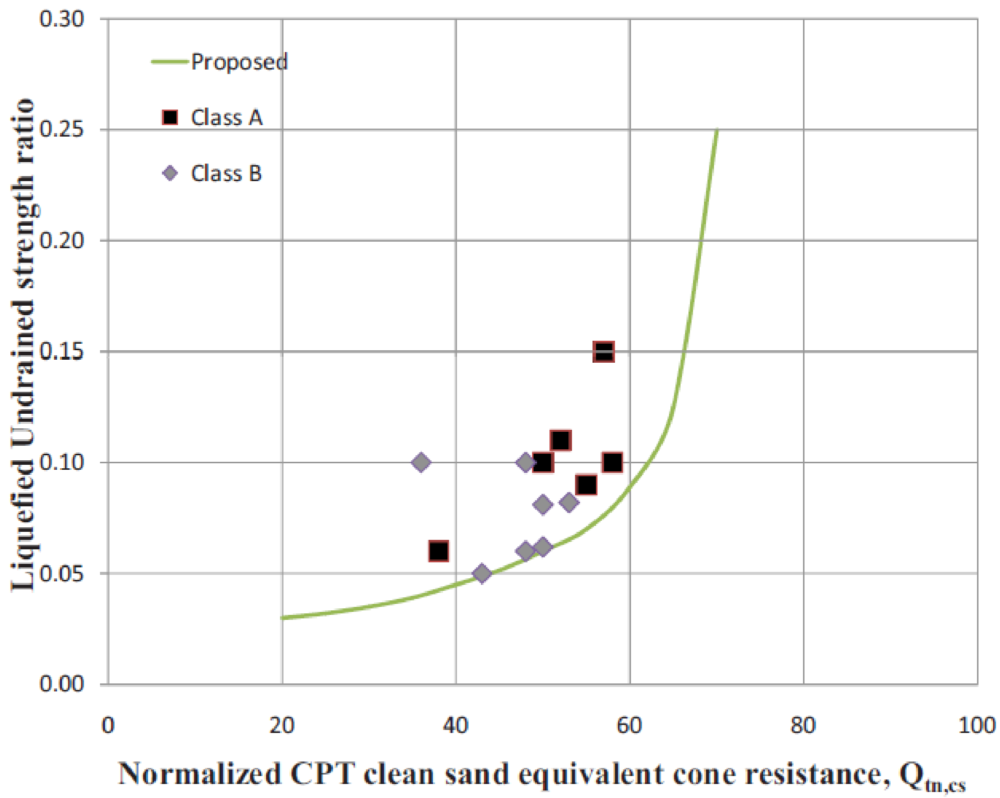


Fig. 7. Liquefied shear strength ratio and normalized CPT clean sand equivalent penetration resistance from Classes A and B flow liquefaction failure case histories

TAILINGS

For a Qtn,cs of 40 the USR = 0.05

Appendix F
Stability Analysis

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Memo

To:	File		
From:	Karen Hincks, MSc, PGeo (NT)	**Reviewer:**	Ed McRoberts, PhD, PEng
cc:	Surinder Garewal, MEng, PEng (BC)	**Wood File No.:**	VE52705D.100.4
Date:	26 August 2019		
Re:	**Clinton Creek Waste Dump and Porcupine Pit Geotechnical Assessment**		

1.0 Introduction

This memo addresses the geotechnical aspects pertaining to the proposed remediation measures at the site of the Clinton Creek Waste Dump at the Clinton Creek mine, located 80 km northwest of Dawson City, Yukon. Please refer to the Geological & Geotechnical Site Characterization and Model for additional information on the background of the site and the history of the Clinton Creek Waste Rock Dump (Wood, 2019a).

In 2016, the Project Partners selected three closure concepts for the Clinton Creek side to be advanced to the 10% design phase. This memorandum addresses the slope stability modelling that was completed in order to support the 10% design of the three closure concepts. This includes back-analysis of the slopes into Hudgeon Lake, in order to determine the parameters to be carried forward in design of the slopes for the closure concepts.

For additional information on assumptions made for strength, liquefaction susceptibility and geology, this memorandum should be read in conjunction with the Geological & Geotechnical Site Characterization and Model and the Design Basis Memorandum (DBM) presented in Appendix I of the Wood Geotechnical Studies Status Report.

2.0 Overall Approach

The overall approach for stability analysis is to determine the slope configurations required for both static and seismic stability, as appropriate. This is accomplished based on the cross-sections developed by Wood for the Clinton Creek Waste Dump. The basic slope model selected for analysis reflects Wood's understanding of how these failures developed, are likely to re-initiate, or be triggered by seismic action. This model is typically a non-circular model comprising an active driving wedge with horizontal to sub-horizontal block sliding on a presumptive weak layer, with the sub-horizonal sliding block essentially following the near bedrock surface. Where appropriate, there is an active wedge at the toe. Circular models were not considered appropriate for this modelling. In the case of Section DS3 into Hudgeon Lake, the stabilizing effect of the lake are accounted for. As the lake is drawn down for Options CC2 and CC3, it is assumed for stability purposes that the dump piezometric level will reflect the lake level during the draw down process. If this is not achieved, then flatter slopes will be required up to some point, at which liquefaction may be triggered by rapid drawdown; a finite element model or FLAC model would be required to resolve this situation.

Following the DBM (Appendix A of this report), the analyses have proceeded as follows:

1. A back analysis was completed on Section DS3 to determine the strength parameters that, along with current piezometric levels, result in a factor of safety (FoS) slightly greater than unity to the slope crest, where cracking has been noted.
2. Then the overall slope angle required for a FoS of 1.2 was determined for the static case and FoS slightly greater than unity for considerations of seismic triggering using an undrained liquefied strength ratio (see Appendix E of this report for the liquefaction assessment). This slope angle was usually near to the slope angle determined for the static case.

Closure options CC2 and CC3 involve creek widening and/or lowering the creek bed. Option CC3 also involves a significant shift of the creek location to the south to avoid ice rich colluvium and to seat the creek channel on the original valley bottom. These channel closure concepts, as well as provision of a safe setback from the re-aligned creek edge, will require excavation and removal of existing waste dump materials and re-shaping of the slopes to achieve FoS acceptable for closure. The purpose of this exercise is to satisfy the requirements of mitigating a catastrophic failure of the dumps into the channel that could lead to another blockage. It will not be practical to shift CC1 to the south to the same amount, given the entry embayment and overall configuration.

The critical strength parameters determined in the back analyses were then used to determine the required slopes for the closure concepts. The slopes were determined by analyzing sections DS4, DS2 (north-south, per Wood, 2019) and DS3 (east-west), with the goal of obtaining a FoS of 1.2 for the determined strength parameters and a FoS of 1.0 for a case with a layer at the base of the waste rock with a mobilized liquefied strength ratio (c/p') of 0.1.

The stability analyses were carried out using SlopeW software (Geostudio 2016) incorporating subsurface data obtained from the 2018 field program and a previous field program conducted in 2016. Additional details for each of the analyzed sections are presented in the following sections.

3.0 Waste Dump Closure Concepts

Three closure concepts have been considered by the Project Partners for the Clinton Creek Waste Dump, as outlined below.

3.1 Option CC1. Water Passage and Catastrophic Failure Mitigation (LCCA Option D3, I2)

This option requires conducting sufficient work on the waste rock pile to mitigate a catastrophic failure of the pile and construct a water conveyance channel to provide water passage from Hudgeon Lake to Clinton Creek.

This option involves keeping Hudgeon Lake at its existing level of approximately elevation 412 masl. Input from the Wood hydrotechnical team indicated that a channel width of 32 m was selected for the water conveyance channel. From a geotechnical perspective, it was decided that the channel should be widened to the south, into the waste rock, rather than over steepening the north slope, which already has some stability issues. This is reflected in the cross-section used for analysis. Also, it has been assumed in this option that the creek bed has not been lowered.

Although specific hydraulic engineering structures will be required, this memorandum focuses only on the required work on the waste rock piles to mitigate a potential catastrophic failure.

2.2 Option CC2. Water Passage, Catastrophic Failure Mitigation and Lowering Lake (LCCA Option E3)

This option requires conducting sufficient work on the waste rock pile to mitigate a catastrophic failure and construct a water conveyance channel to provide water passage from Hudgeon Lake to Clinton Creek, and lower Hudgeon Lake as a part of this concept.

This option involves lowering Hudgeon Lake by some distance below its existing level, assumed for stability to be 10 m, ie to elevation 402 masl. Input from the Wood hydrotechnical team indicated that a channel width of 32 m was selected for the water conveyance channel, along with a corresponding lowering of the creek bed. From a geotechnical perspective it was decided that the channel should be widened to the south, into the waste rock, rather than over steepening the north slope, which already has some stability issues, including thawing permafrost. The creek was assigned an overall gradient of 3%, per the hydrotechnical engineers, which would essentially provide a "regime channel". These decisions are reflected in the cross section presented in the stability analysis.

3.3 Option CC3. Water Passage with Reduction of the Lake Level, Eliminating the Dam, and Mitigating Catastrophic Failure (LCCA Option F)

This option requires conducting sufficient work on the waste rock pile to prevent it from acting like a dam (as defined by the Canadian Dam Association) on Clinton Creek and to mitigate a catastrophic failure of the waste rock pile and construct a water conveyance channel to provide water passage through the site.

This option involves completely draining Hudgeon Lake. Creek restoration work would consist of constructing a 32 m wide channel at the original location of the creek, that is the location prior to mining activity. This would result in the creek channel shifting south of its present location.

4.0 Results

Results for the three options are presented on a cross-section basis in the following order DS3, DS4, and DS2. The three options are labelled by lake elevation and modelled as 412 masl, 402 masl, and No-Lake. The analysis began with DS3 to obtain insights in to the back-analysis approach and the relative impact of liquefaction versus static stability. The results of stability analyses undertaken are reported in Appendices I, II, and III to this memorandum.

4.1 DS3 – East-West Section at Hudgeon Lake

The back-analyses assumed that the slopes had a current FoS of 1.0 back to the extent of visible cracking noted in project photographic archives. This was taken to be near the crest of the slope, close to borehole BH18-02. Slope stability analyses were then run to determine what slopes would be required in order to achieve a 20% increase in the FoS for the overall slope (static condition), and also obtain a FoS of 1.0 in the case of the mobilization of liquefied strengths in a layer at the base of the waste, with an assumed mobilized strength of a c/p' of 0.1.

Per the site characterization report (Wood, 2019), a 2 m thick layer of a low to medium plastic silt and clay unit was found at about elevations 376 to 378 masl in BH18-01 and BH18-02 and is incorporated into the DS3 model as shown in Appendix I Slide 3. Back-analysis determined a FoS of 1.00 for a slope failure between the shoreline and BH18-01 with a mobilized undrained strength ratio of 0.19. Appendix I Slide 4 indicates that a mobilized undrained strength ratio of 0.14 is required to drive the overall failure back to the existing slope crest, consistent with cracking patterns. An undrained strength ratio of 0.14 potentially represents a remoulded strength ratio of a low plasticity silt clay. It was then assumed that an undrained liquefied strength ratio of 0.1, representing seismically liquefied waste dump fill, was present at the same elevation (or just above the silt and clay). The FoS for the existing slope was predicted to be 0.85 using the liquefied strength ratio.

Table 4.1 provides a summary of the analyses for section DS3 presented in Appendix I.

Table 4.1: Summary of Stability Analyses Results for Section DS3

Lake Elevation	Slope	FoS (static)	FoS (Liquefaction)
412	4H:1V (current)	Back Analysis Overall 1.02 for c/p' = 0.14	0.85
	6H:1V	1.21 for c/p' = 0.14	1.00
402	4H:1V (current)	Back Analysis Overall 1.05 for c/p' = 0.14	0.87
	6H:1V	1.22 for c/p' = 0.14	1.01
No lake	4H:1V (current)	Back Analysis Overall 1.17 for c/p' = 0.14	0.98
	6H:1V	1.31 for c/p' =0.14	1.07 (Crest) 1.02 (Toe)

Based on these analyses, a 6H:1V overall cut slope angle was selected for all closure concept options towards Hudgeon Lake.

4.2 DS4 – North-South section at west end of the spillway

Section DS4 is unique in that BH18-03 encountered about 8 m of ice rich permafrost interpreted to be colluvium. Per the DBM (Appendix A of this report), this icy ground has been assigned a frozen strength of 50 kPa. In the closure landscape it is assumed that due to thermal effects this permafrost will eventually thaw.

Analyses of the waste rock dump in this section revealed that the slopes in the existing configuration are at a FoS of 1.01 (static condition) and less than 1.0 in the case of failure being controlled by liquified strength parameters, which, as per DS3, were modelled using a presumptive seismically liquefied layer of waste material assumed at essentially the same depth of a possible silt clay layer. The presence of a frozen colluvium layer at the toe (north end) is responsible for these lower factors of safety.

Table 4.2 provides a summary of the analyses presented in Appendix II.

Table 4.2: Summary of Stability Analyses Results for Section DS4

Lake Elevation	Slope	FoS (static)	FoS (Liquefaction)
412	4.7H:1V (current)	1.01 (Permafrost present[1])	
	6.5H:1V	1.16 (Permafrost present[1]) 1.35	0.83 (all waste below water liquefies) 1.2 (thin layers in waste liquefy)
402	6.5H:1V	1.21 (Permafrost present[1]) 1.48	0.78 (all waste below water liquefies) 1.2 (thin layers in waste liquefy)
No lake	6.5H:1V	1.21 (Permafrost present[1]) 1.17	0.95

Note(s)

1. Colluvium below the waste rock has been assumed to be ice-rich (frozen state) with cohesion of 50 kPa and no friction

Flattening of the dump slopes to 6.5H:1V increases their FoS to meet acceptable levels for both the static and liquefiable cases.

4.3 DS2 – North-South section at east end of the spillway

Analyses of the waste rock dump in this section revealed that the slopes in the existing configuration have acceptable factors of safety. This is because of a lower phreatic surface being present in the dump, since this section is the furthest away from the lake and, therefore, least affected by the lake water levels. Any rise in the phreatic surface causes a reduction in the slope safety factor. The failure is controlled by potential liquefaction of the saturated layer at the base of the waste rock in the event of a rise in phreatic surface.

Table 4.3 provides a summary of the analyses presented in Appendix III.

Table 4.3: Summary of Stability Analyses Results for Section DS2

Lake Elevation	Slope	FoS (static)	FoS (Liquefaction in waste rock)
412	4H:1V (current)	2.36	1.05
	6.5H:1V	1.33 (Frozen Colluvium) 1.42 (Thawed Colluvium)	0.38 (all waste below water liquifies) >1.09 (thin layers in waste liquefy)
402	6.5H:1V	1.39 (Frozen Colluvium) 1.39 (Thawed Colluvium)	0.79 (all waste below water liquifies) (Thawed Colluvium) >1.1 (thin layers in waste liquefy) (Thawed Colluvium)
No lake	6.5H:1V	1.13 (Thawed Colluvium)	1.11 (thin layer in waste rock liquefies)

The results of the analyses indicate that overall flattening of the dump slopes to 6.5H: 1V or greater increases their factors of safety to meet both the static and liquefaction case requirements (with the exception of the very conservative case where all waste material below the water table liquifies).

An additional analysis was performed using benching of the dump slopes. Benching of the slopes resulted in a slight steepening of the slope (from 6.5H:1V to 5.6H:1V) with a corresponding very slight lowering of the FoS. The obtained FoS by benching were found to be acceptable for both the static and liquefied cases.

4.4 Porcupine Pit Backfill Stability

The closure options outlined in the above sections involve stabilization of the waste dumps by means of flattening their existing slopes. Flattening of the dump slopes will result in removal of significant amounts of waste materials from existing locations and transporting and storage in the Porcupine Pit.

Outer (exposed) areas of the stored waste materials, as well as the north end of the in-pit wall, will be covered/encapsulated with a 50 m wide “shell” of compacted waste material, to provide a protective cover to the waste materials. Stability analyses conducted on the overall waste materials in the pit indicated stable slopes both in static and seismic conditions (acceptable FoS). Select stability analyses for the Porcupine Pit shell are presented in Appendix IV. Stability analysis for the Porcupine Pit shell were undertaken using Entry/Exit analysis, assuming a phreatic surface at 375 masl, consistent with the current water level in the Porcupine Pit.

Stability of the existing pit slope walls has not been addressed as part of this study. It is recommended to conduct an assessment of the pit wall stability and develop a safe work plan prior to any work being undertaken within the pit perimeter.

5.0 References

Wood (2019). Geological & Geotechnical Characterization and Model, Clinton Creek Remediation Project, Rev. 2. Project # VE52705C. August 2019.

Sincerely,

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Reviewed by:

Ed McRoberts, PhD, PEng
Principal Geotechnical Engineer

Appendix I

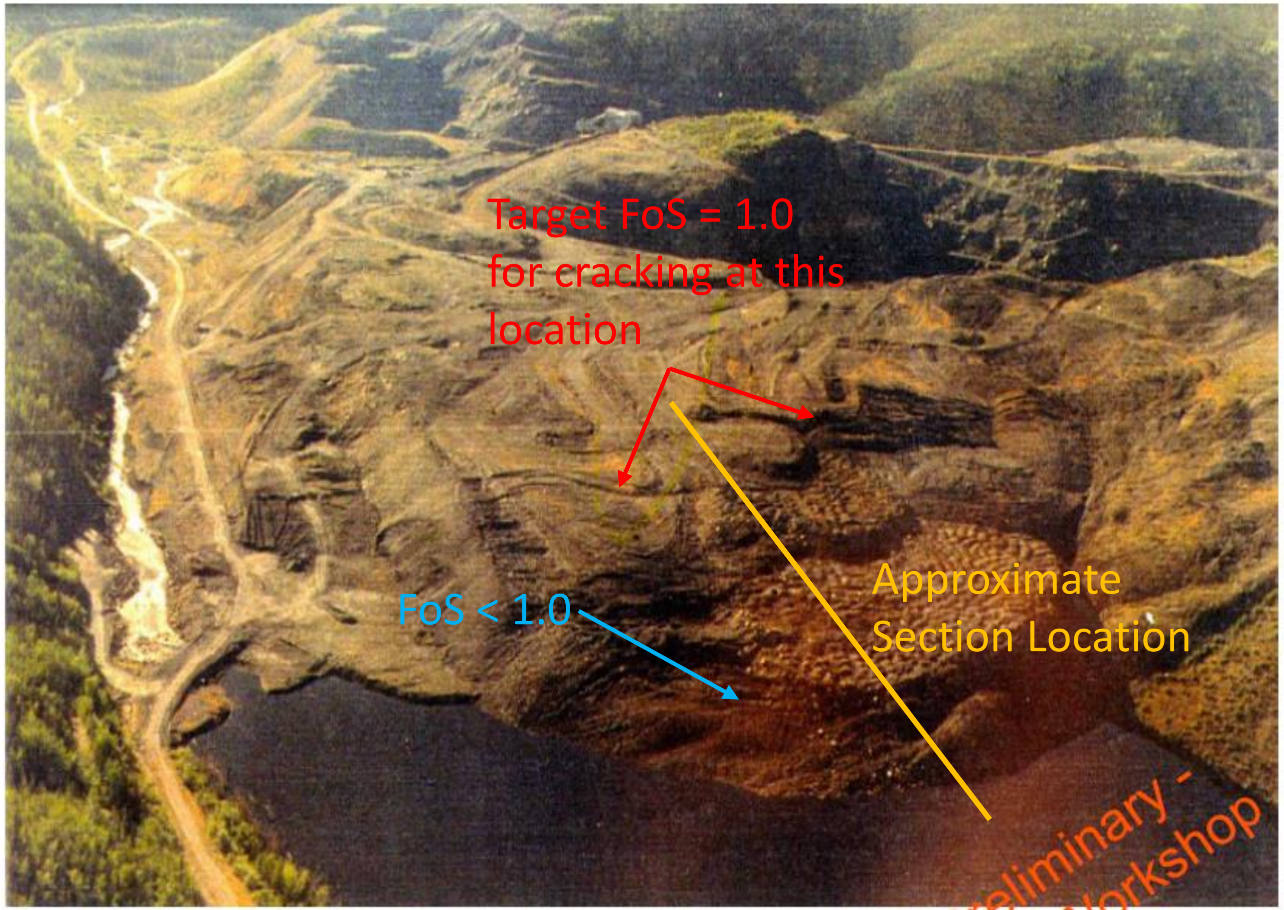
DS3 Slope Stability Summary

DS3 - Slope Stability Summary

1985

Water El.

411.5m



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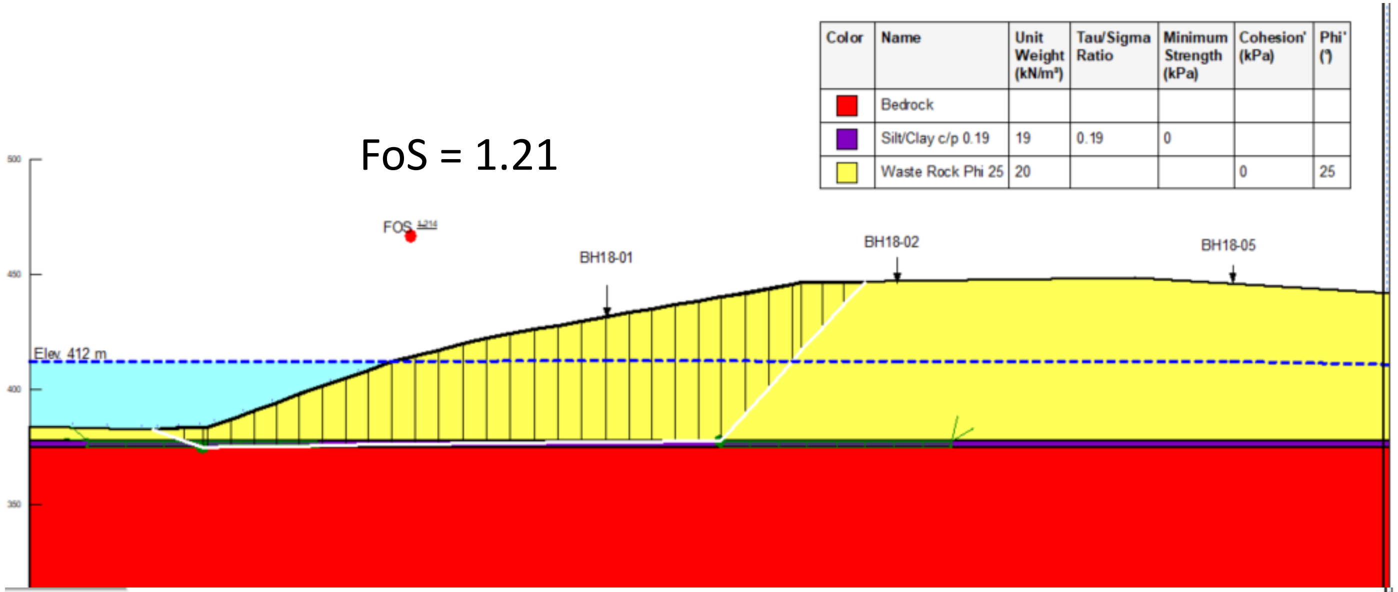
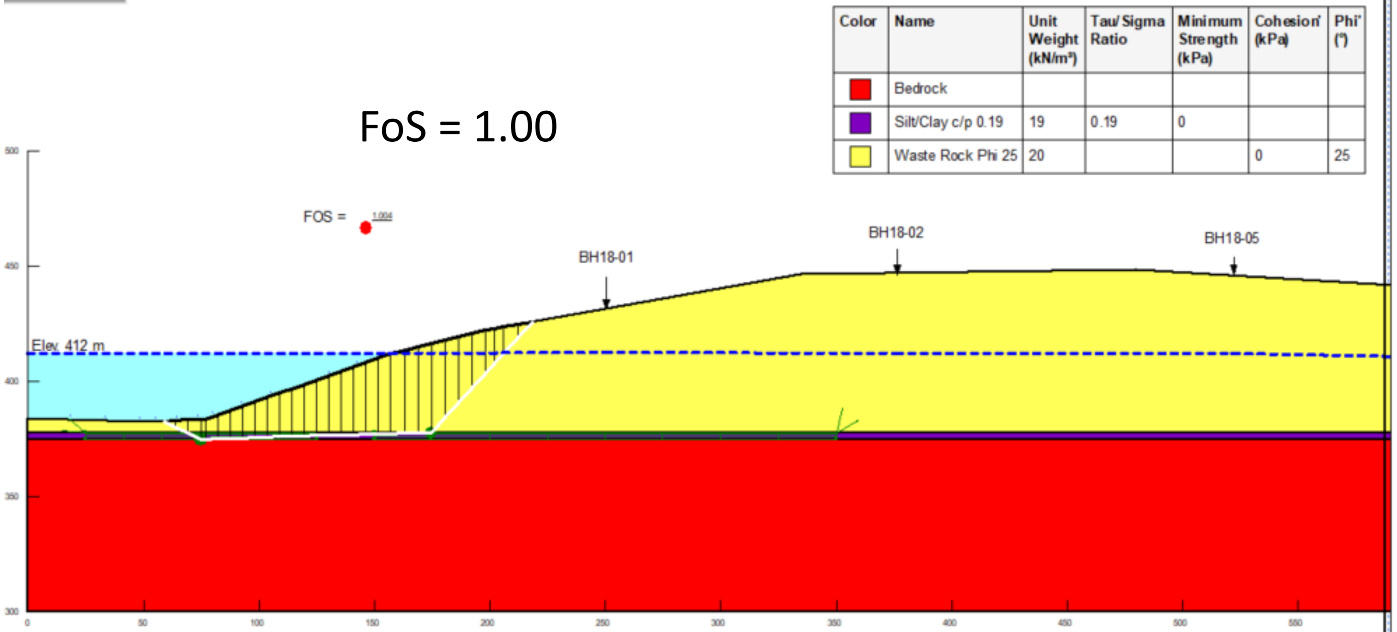
CC1 - Lake at El. 412 m

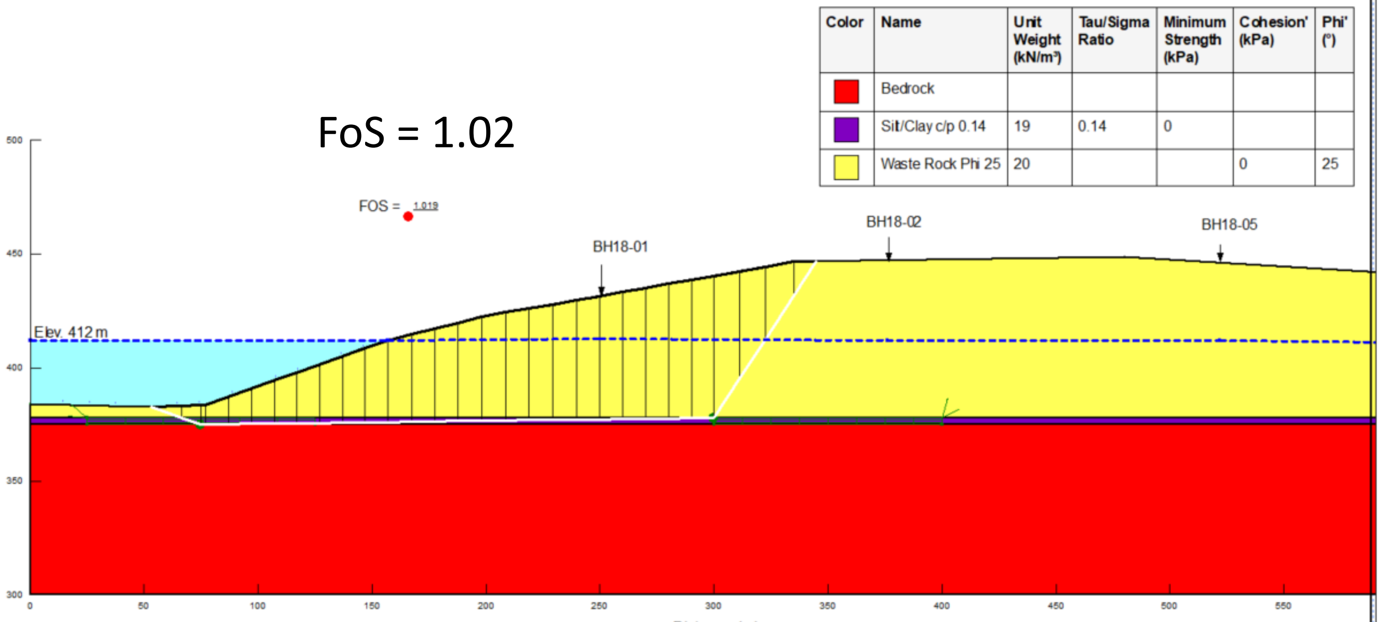
Base Case – Current Slope Geometry

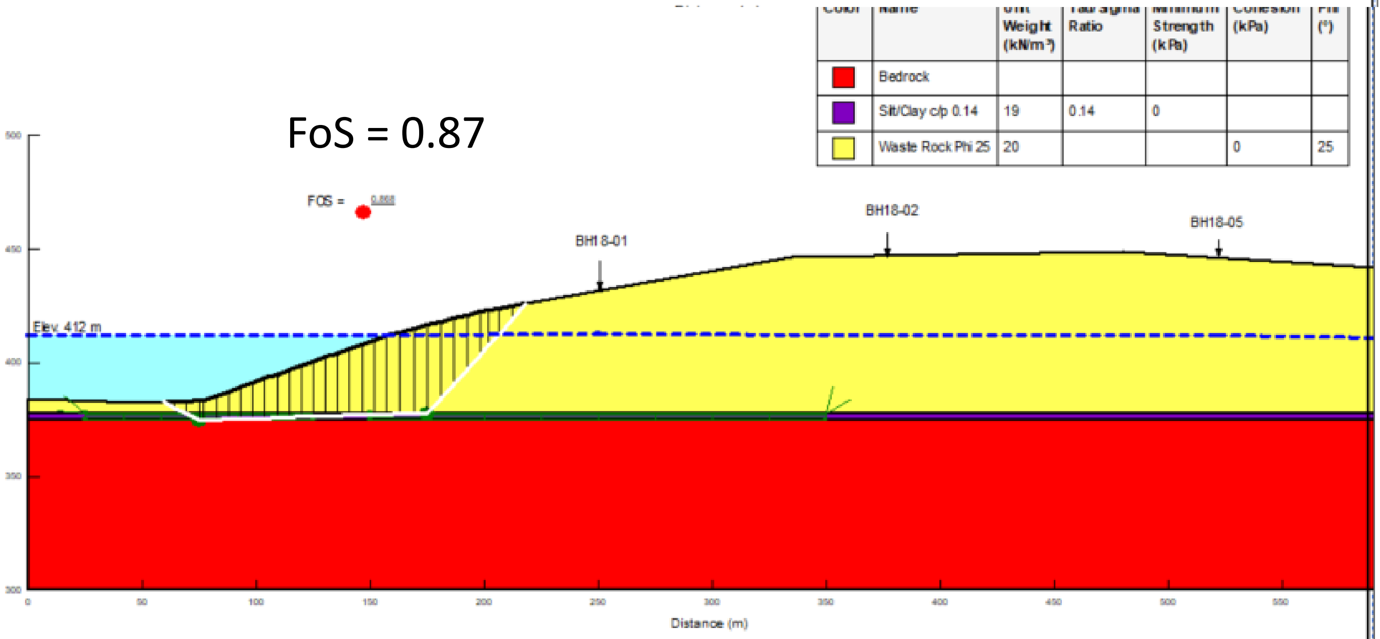




- Current Slope Geometry (nominally 4H:1V)
- Using c/p' = 0.19 in order to get FoS > 1.00 for the current slope (looking for the minimum FoS)
- Results in FoS > 1.2 at the crest of the slope

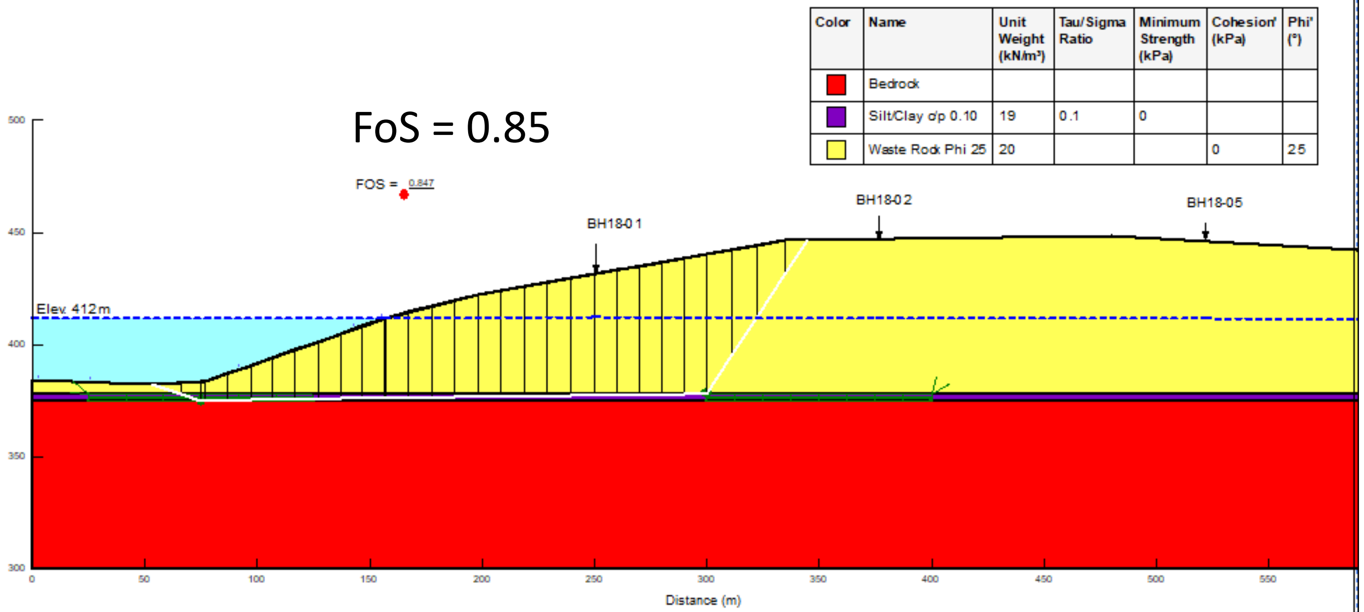
Base Case – Current Slope Geometry

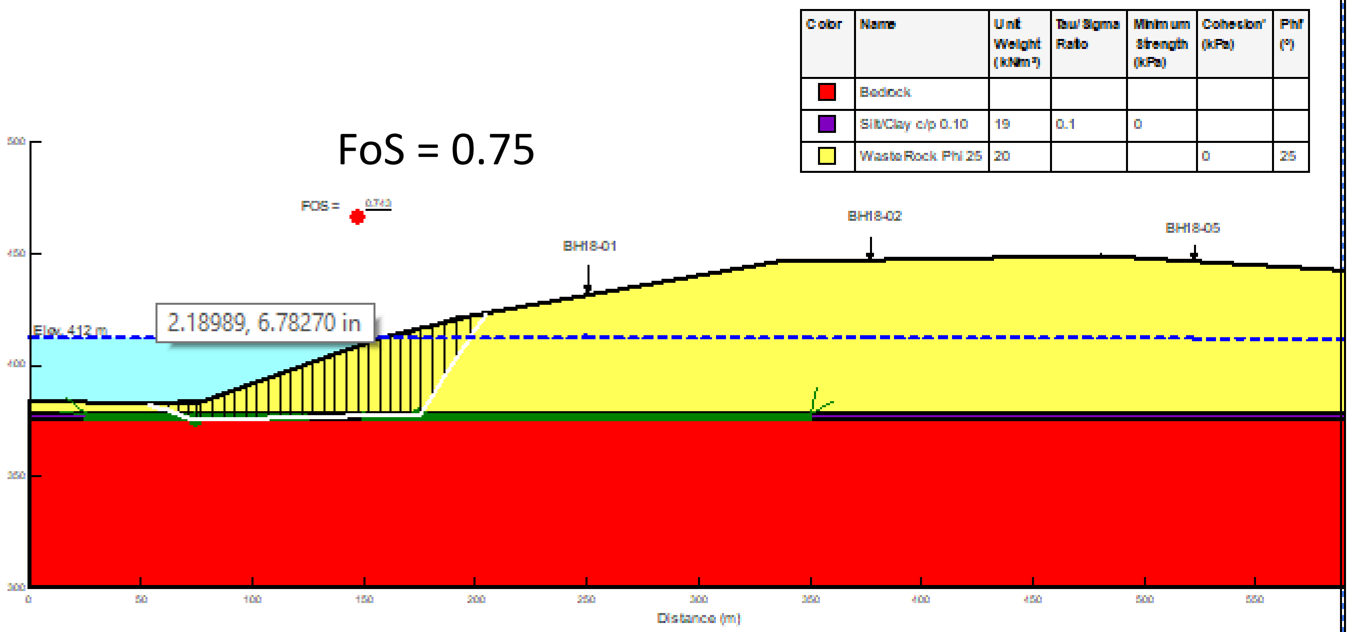




- Current Slope Geometry (nominally 4H:1V)
- Using c/p' = 0.14 in order to get FoS > 1.00 at the crest of the slope
- Results in FoS < 1.00 near Hudgeon Lake

Base Case – Current Slope Geometry

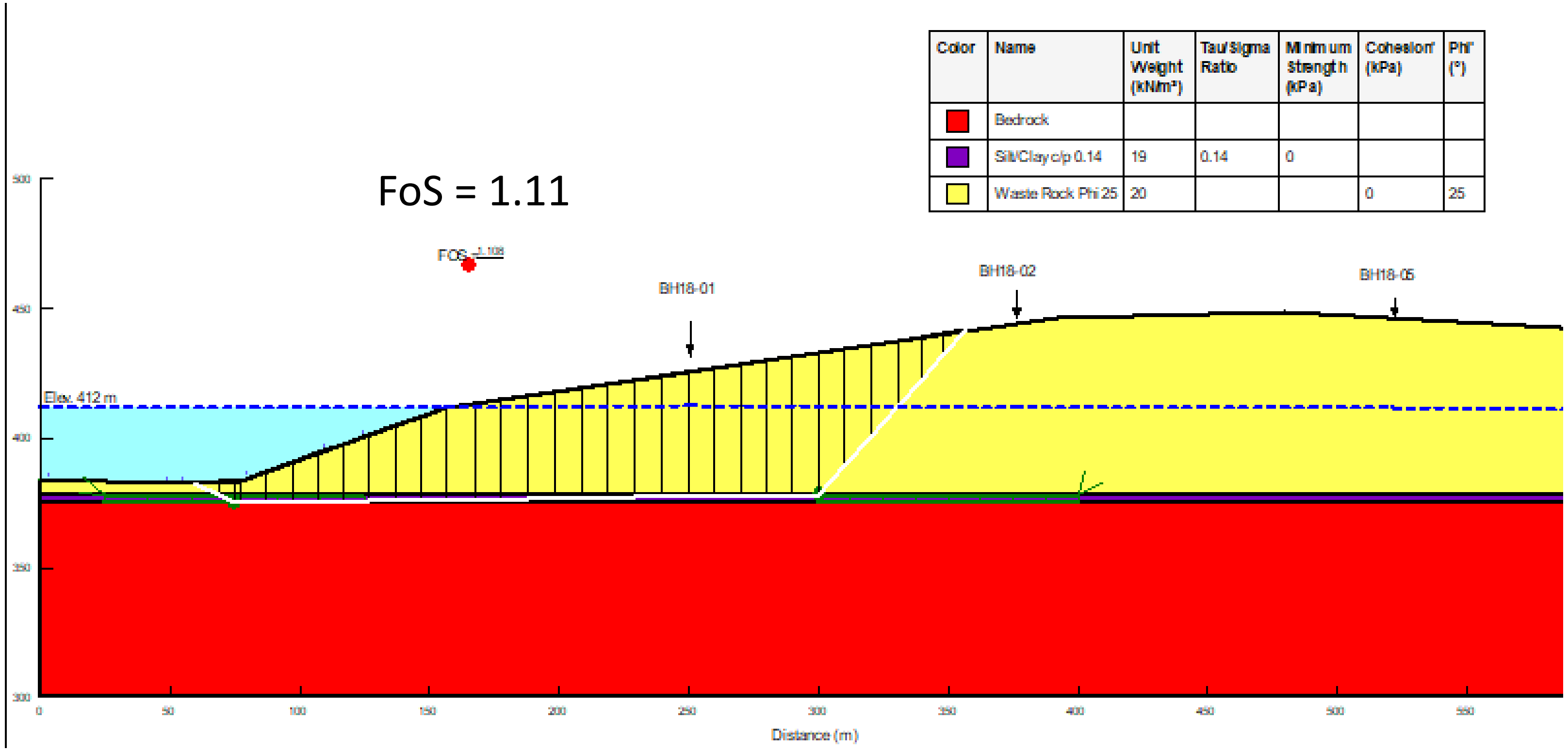




- Current Slope Geometry (nominally 4H:1V)
- Using c/p' = 0.10 to test the stability under liquefaction conditions
- Results in FoS < 1.00 for both the overall case and the case near the toe.
- Slope is not expected to stand up under liquefaction

5H:1V Overall Slope

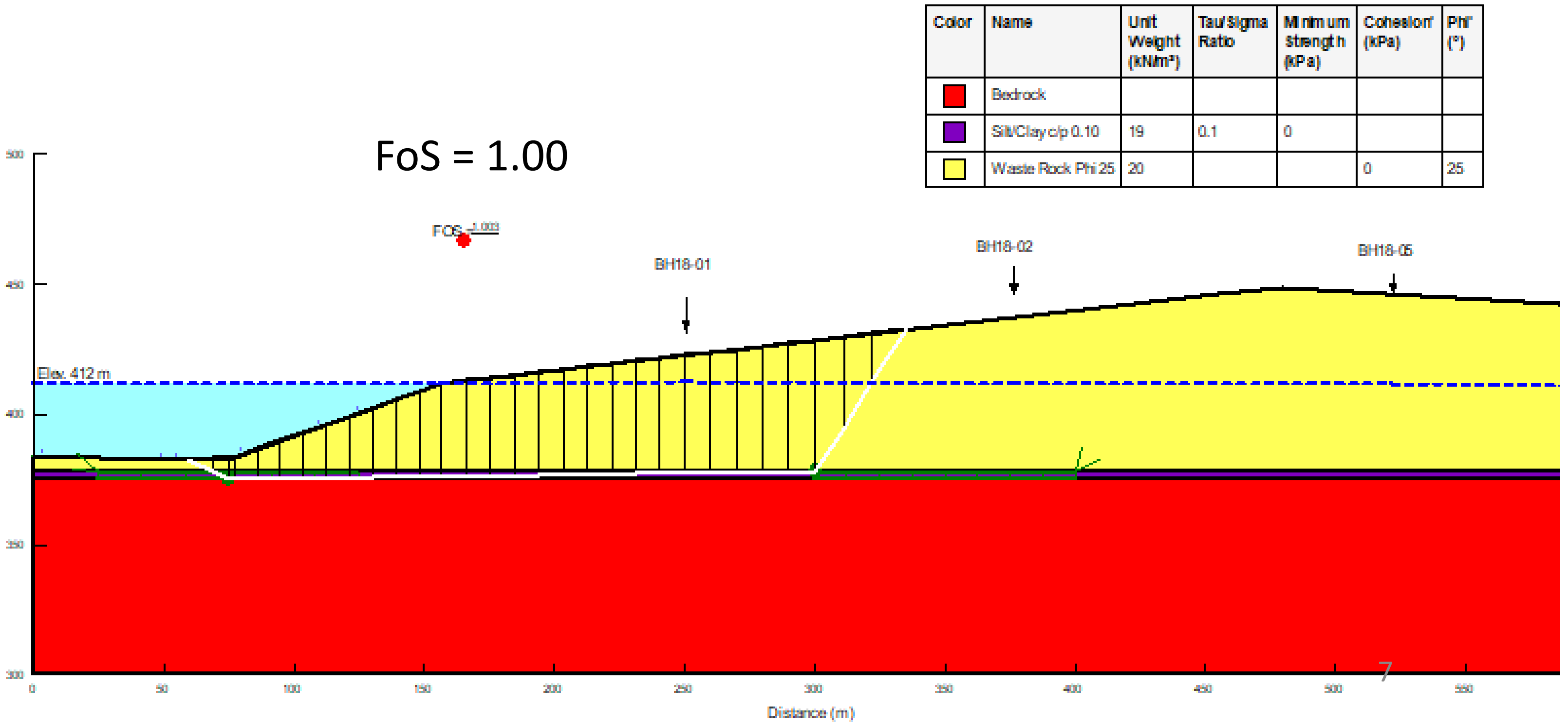
c/p’	Fos Overall	FoS at Toe	% Improvement	
0.19	1.34	1.01	10.7	0.6
0.14	1.11	0.87	9.8	0
0.10	0.93	0.74	9.4	0



6H:1V Overall Slope

c/p’	FoS Overall	FoS at Toe	% Improvement	
0.19	1.45	1.13	20	13
0.14	1.21	0.97	19	11
0.10	1.003	0.83	18	12

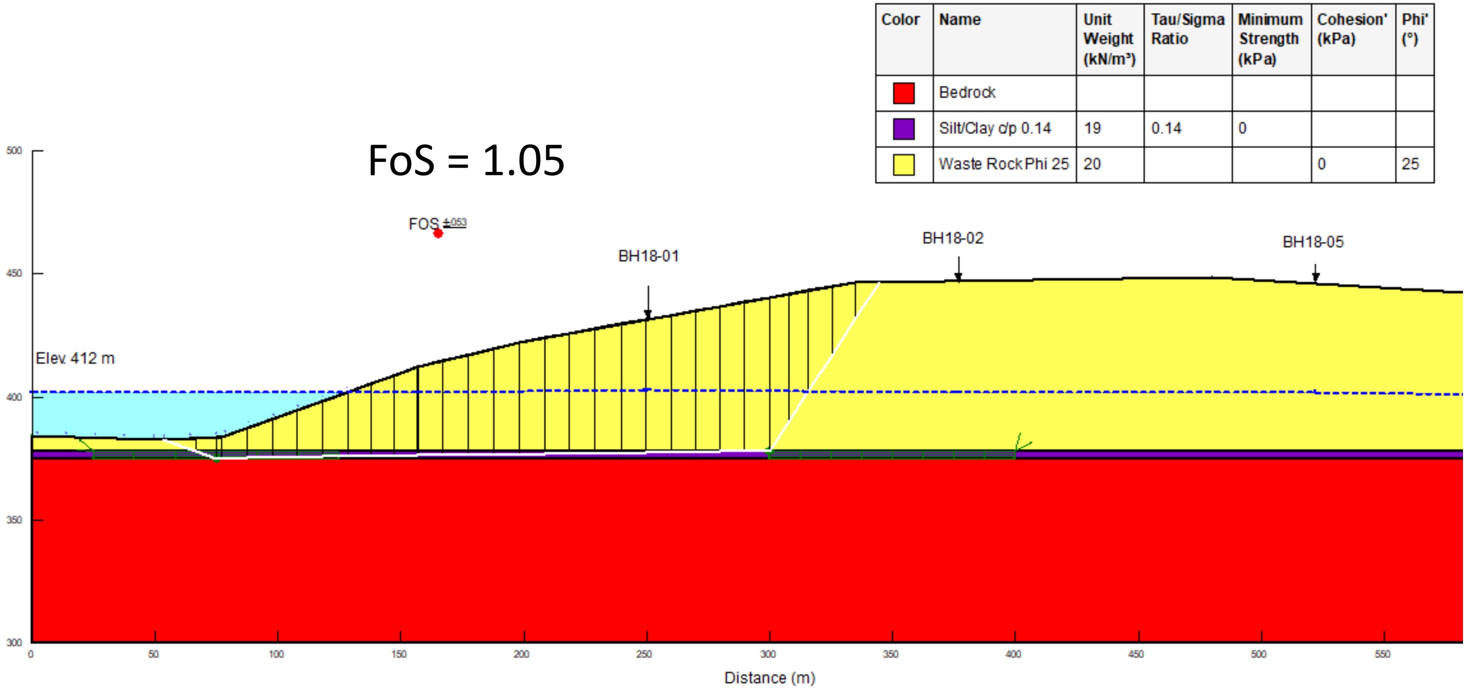
- Meets 20% increase in FoS for c/p’ = 0.14 in base of waste material layer
- Meets FoS > 1.001 for liquefaction case of c/p’ = 0.10 in base of waste material
- FoS at toe remains < 1.0



CC2 - Lake at El. 402 m

Current Geometry (4H:1V)

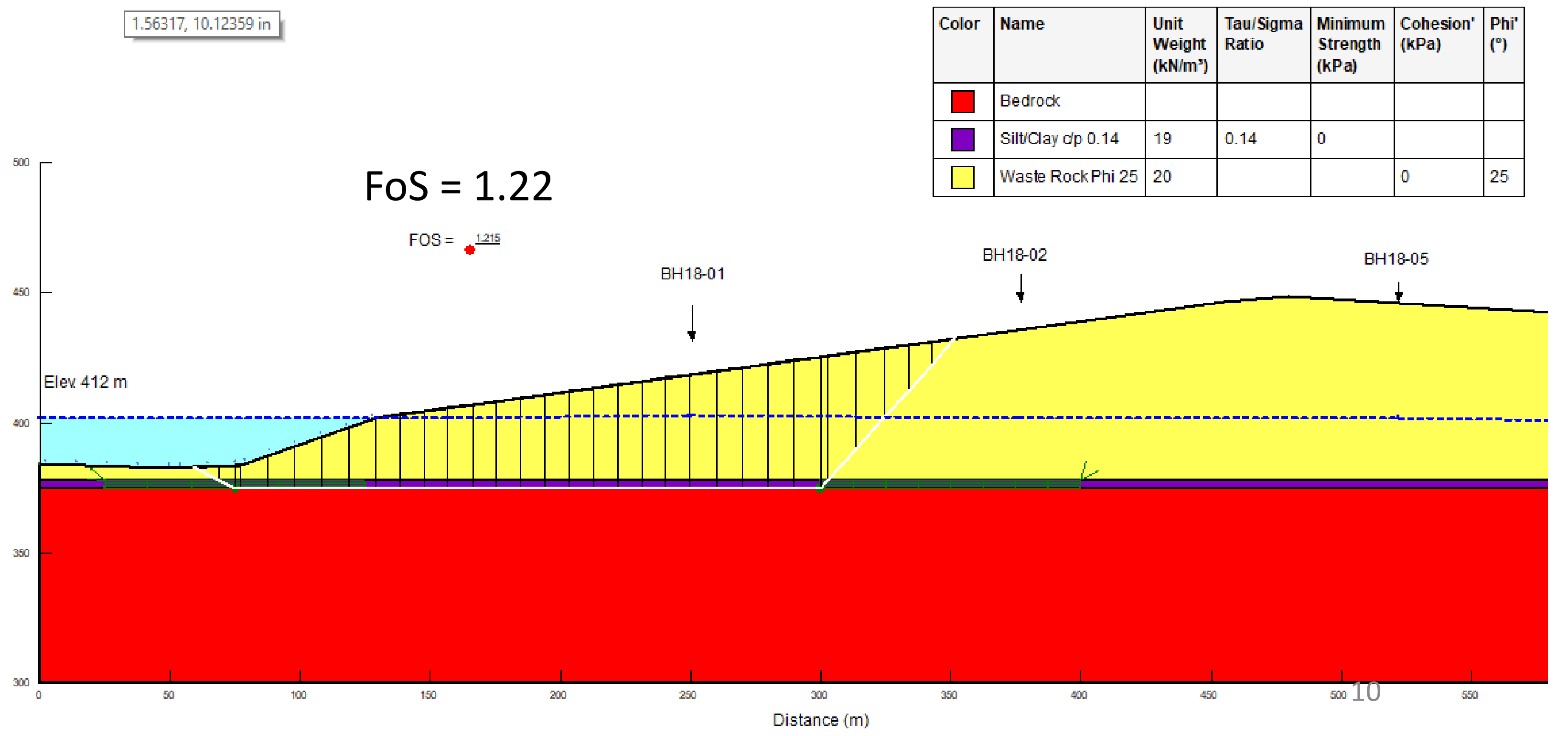
c/p’	FoS Overall	FoS at Toe
0.19	1.26	0.93
0.14	1.05	0.81
0.10	0.87	0.71



6H:1V Overall Slope

c/p'	FoS Overall	FoS at Toe	% Improvement	
0.19	1.46	1.20	16	29
0.14	1.22	1.04	16	28
0.10	1.01	0.89	16	25

- FoS = 1.2 for c/p' = 0.14 for the overall slope
- Meets FoS > 1.001 for liquefaction case of c/p' = 0.10 in base of waste material layer
- FoS > 1.0 at the toe for the c/p' = 0.14 case

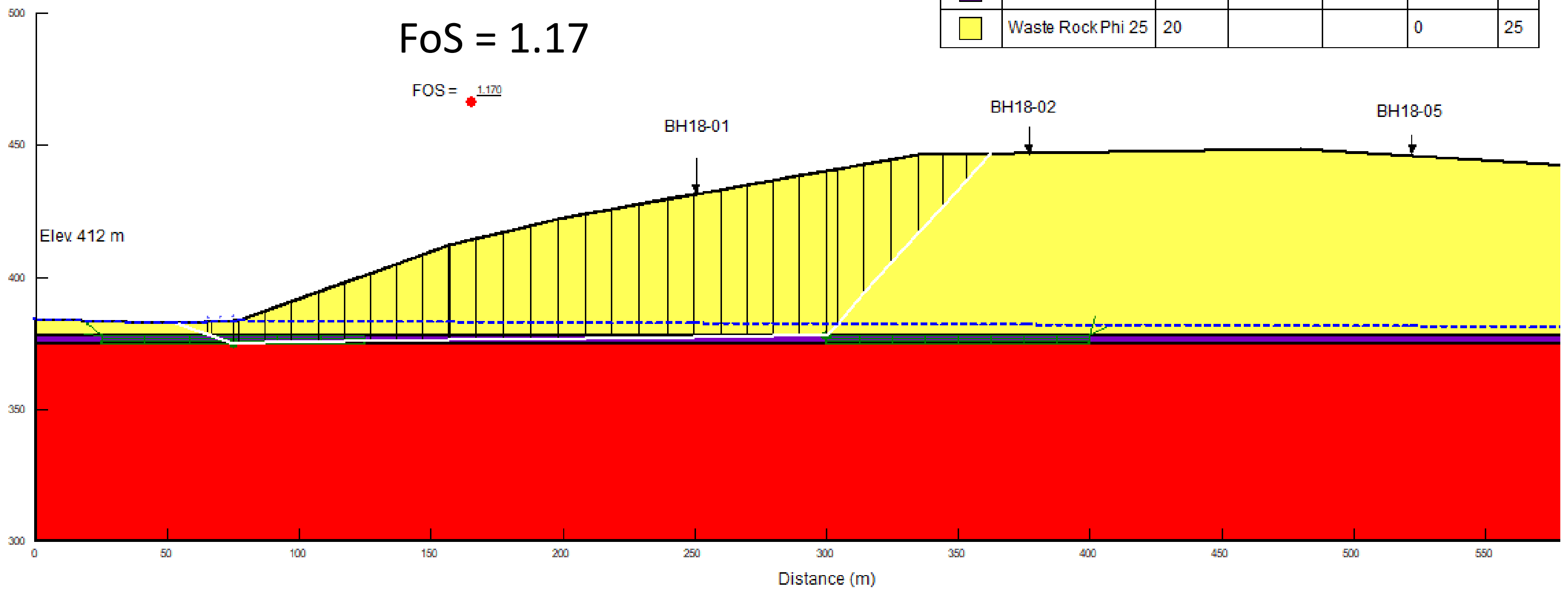


CC3 - No Lake

Current Geometry (4H:1V)

c/p'	FoS Overall	FoS at Toe
0.19	1.41	1.33
0.14	1.17	1.12
0.10	0.98	0.93

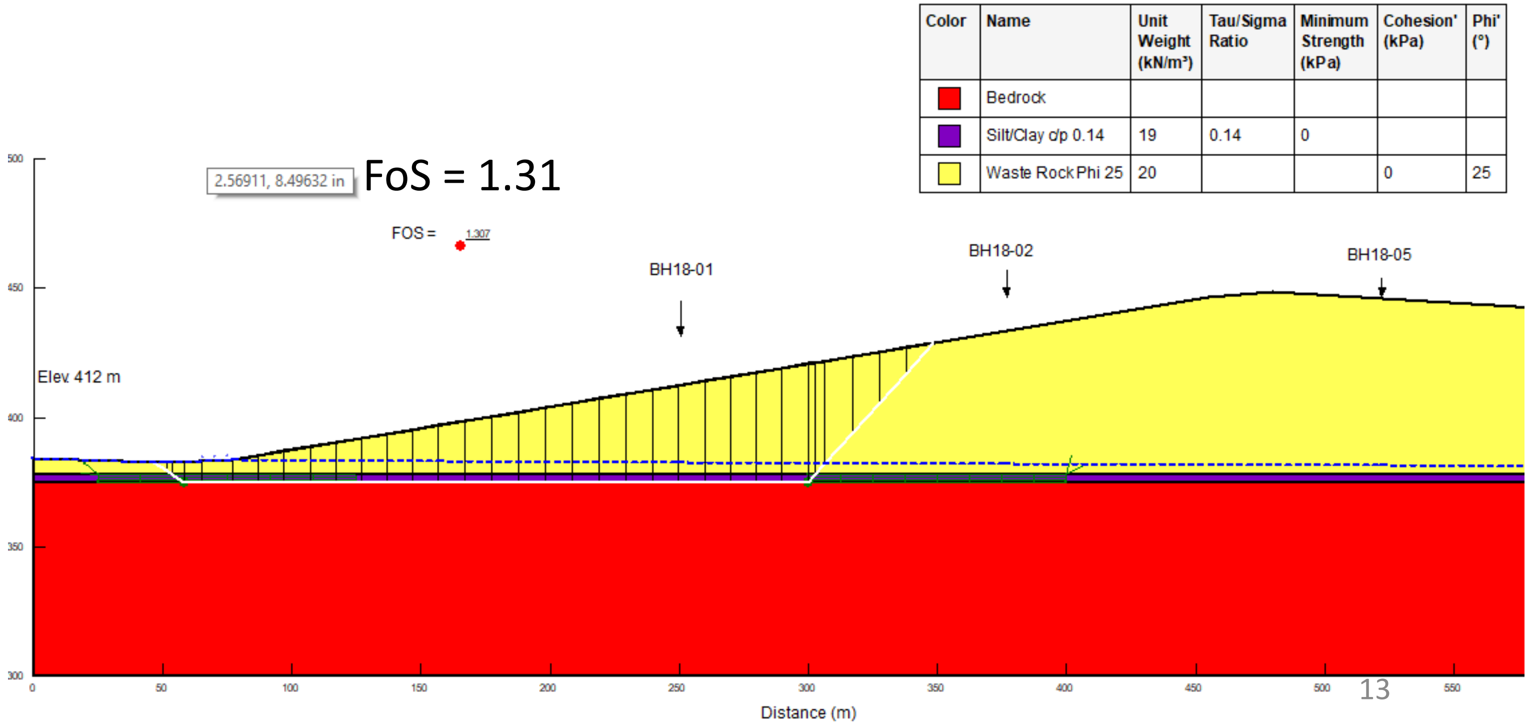
Color	Name	Unit Weight (kN/m³)	Tau/Sigma Ratio	Minimum Strength (kPa)	Cohesion' (kPa)	Phi' (°)
	Bedrock					
	Silt/Clay c/p 0.14	19	0.14	0		
	Waste Rock Phi 25	20			0	25



6H:1V Overall Slope

c/p’	FoS Overall	FoS at Toe	% Improvement	
0.19	1.58	1.48	12	55
0.14	1.31	1.24	12	49
0.10	1.07	1.02	11	42

- FoS = 1.3 for c/p’ = 0.14 for the overall slope
- Meets FoS > 1.001 for liquefaction case of c/p’ = 0.10 in base of waste material layer
- FoS > 1.0 at the toe for the c/p’ = 0.14 case and the liquefaction case



Appendix II

DS4 Slope Stability Summary

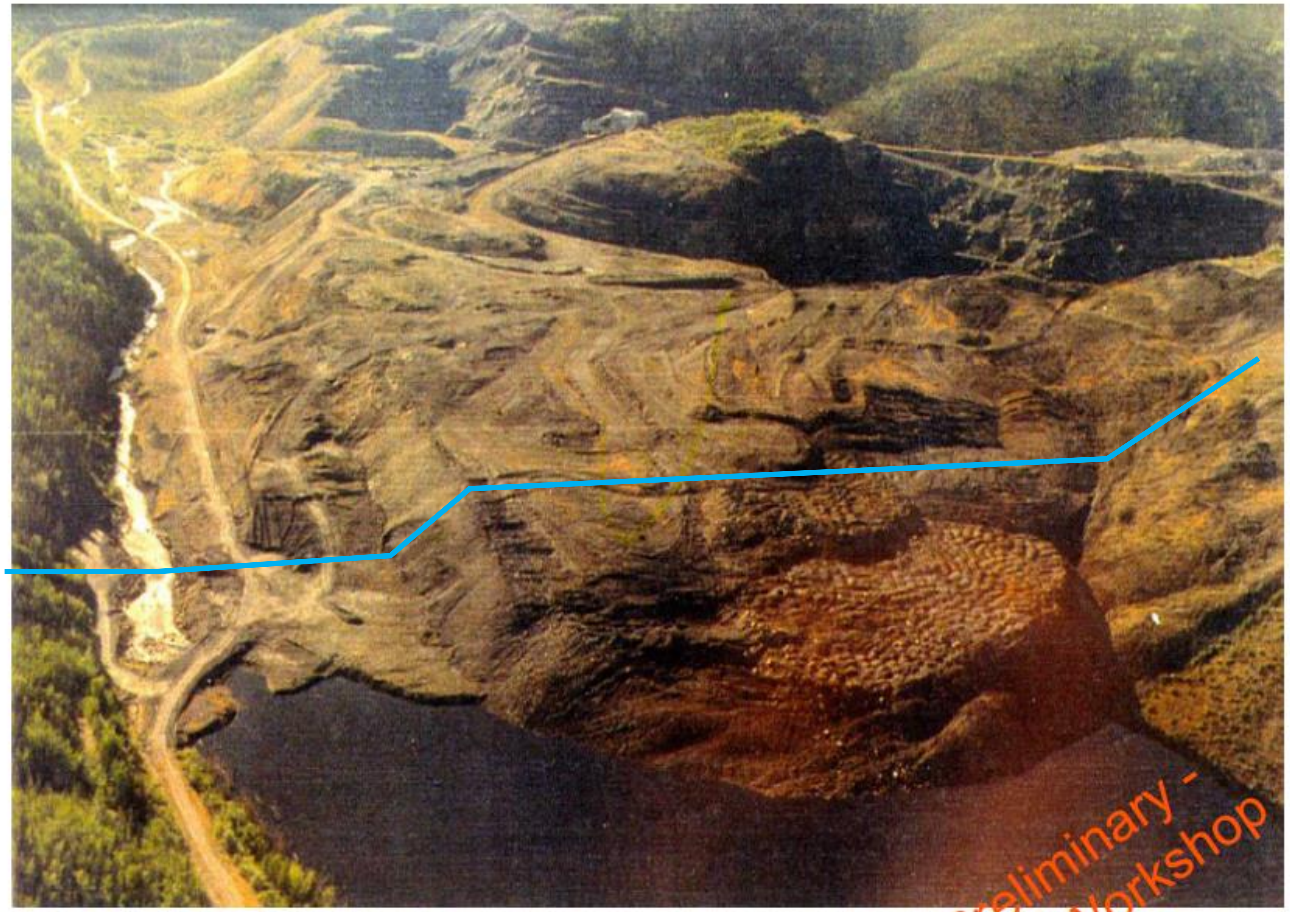
DS4 - Slope Stability Summary

Historical Review of Waste Pile

amec foster wheeler

1985

Water El.
411.5m



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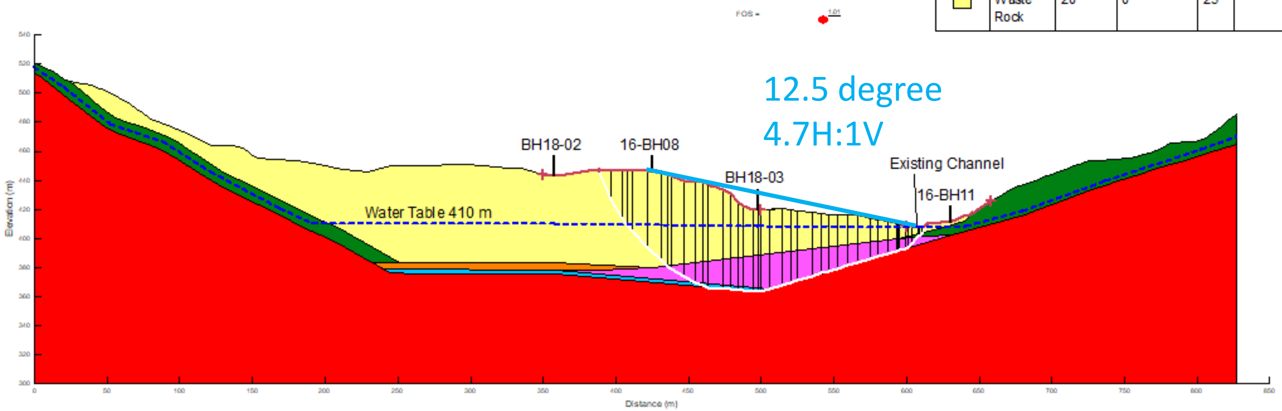
CC1 - Lake at El. 412 m

Base Case – Current Slope Geometry

Color	Name	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Tau/Sigma Ratio	Minimum Strength (kPa)
Orange	Alluvium	20	0	25		
Red	Bedrock					
Green	Colluvium	20	0	25		
Pink	Frozen Colluvium	20	50	0		
Blue	Silt/Clay	19			0.14	0
Yellow	Waste Rock	20	0	25		

FoS = 1.01



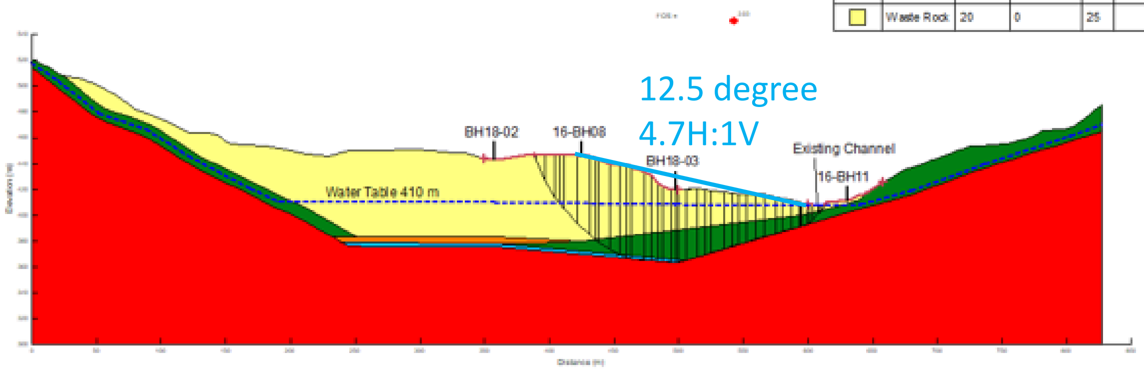
c/p’	FoS Overall
0.14	1.01

- Current conditions: with frozen cohesion of 50 kPa in the high ice content colluvium.
- Long term: permafrost thaws. Waste dump material settles over thawed-out colluvium.
- Liquefied waste dump will control.
- Slope needs to be cut back

Base Case – Current Slope Geometry

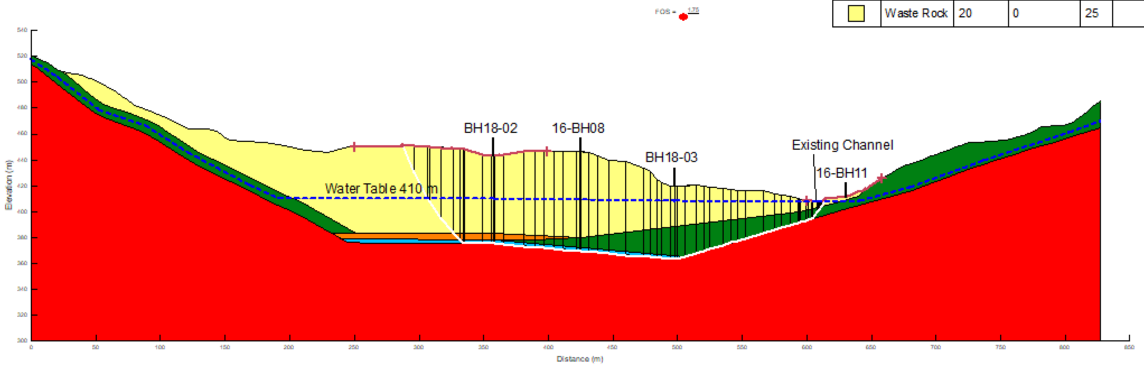
FoS = 2.03

Color	Name	Unit Weight (kN/m^3)	Cohesion' (kPa)	Phi' (°)	Tau/Sigma Ratio	Minimum Strength (kPa)
	Alluvium	20	0	25		
	Bedrock					
	Colluvium	20	0	25		
	Silt/Clay	19			0.14	0
	Waste Rock	20	0	25		



FoS = 1.75

Color	Name	Unit Weight (kN/m^3)	Cohesion' (kPa)	Phi' (°)	Tau/Sigma Ratio	Minimum Strength (kPa)
	Alluvium	20	0	25		
	Bedrock					
	Colluvium	20	0	25		
	Silt/Clay	19			0.1	0
	Waste Rock	20	0	25		

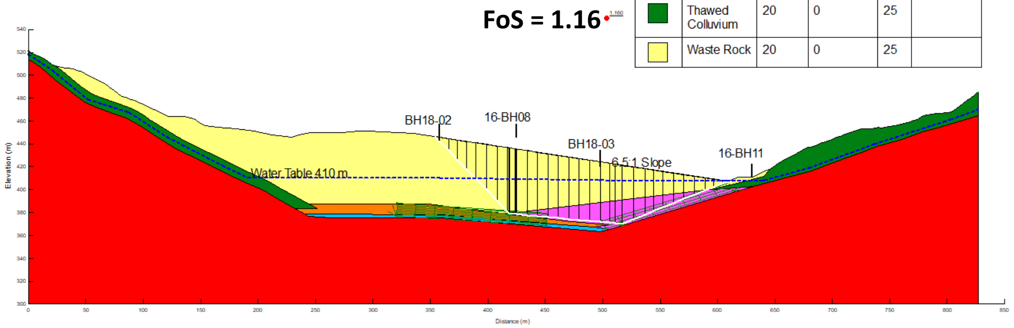


c/p' in silt clay	FoS Overall
0.14	2.03
0.10	1.75

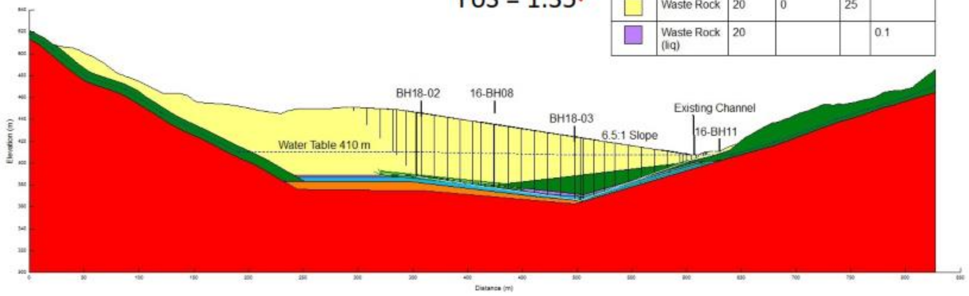
- Assuming that the colluvium encountered in BH18-03 has thawed, with a ϕ=25°
- Alluvium, unfrozen (or ice poor) colluvium and waste material all use a friction angle of 25 degrees

6.5H:1V Overall Slope

Color	Name	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Tau/Sigma Ratio
	Alluvium	20	0	25	
	Bedrock				
	Frozen Colluvium	20	50	0	
	Silt/Clay	19			0.14
	Thawed Colluvium	20	0	25	
	Waste Rock	20	0	25	

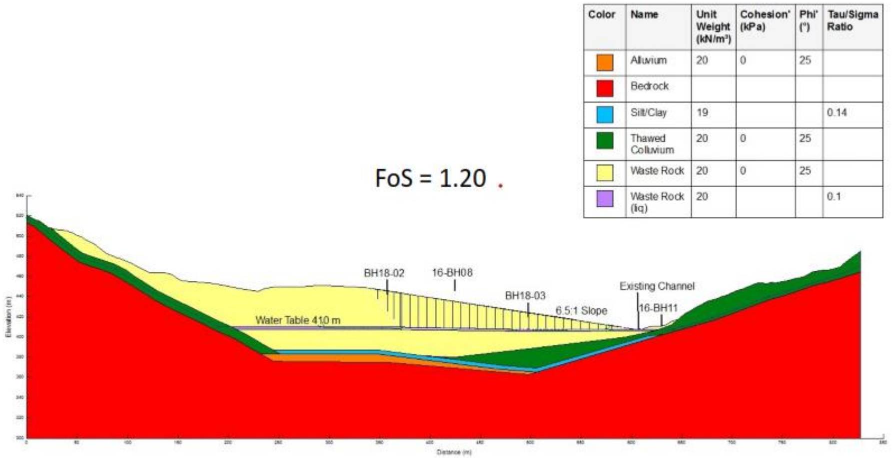


Color	Name	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Tau/Sigma Ratio
	Alluvium	20	0	25	
	Bedrock				
	Silt/Clay	19			0.14
	Thawed Colluvium	20	0	25	
	Waste Rock	20	0	25	
	Waste Rock (liq)	20			0.1

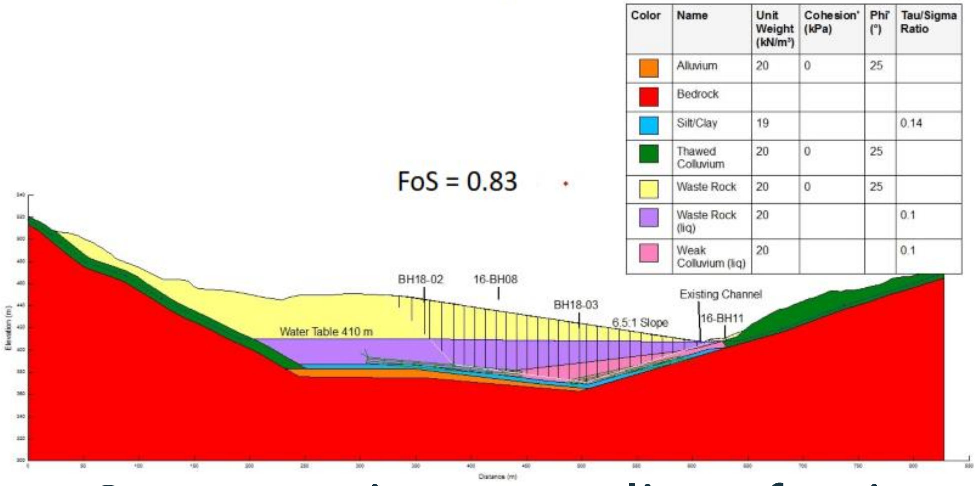


- Determined required slopes for DS4 as follows:
  - Short-term stability governed by frozen ice-rich colluvium provided FoS=1.16 for 6.5H:1V slopes
  - Long term stability improves with thawed colluvium using $\phi=25^o$, FoS = 1.35 for 6.5H:1V slopes

6.5H:1V Overall Slope



Color	Name	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Tau/Sigma Ratio
	Alluvium	20	0	25	
	Bedrock				
	Silt/Clay	19			0.14
	Thawed Colluvium	20	0	25	
	Waste Rock	20	0	25	
	Waste Rock (liq)	20			0.1



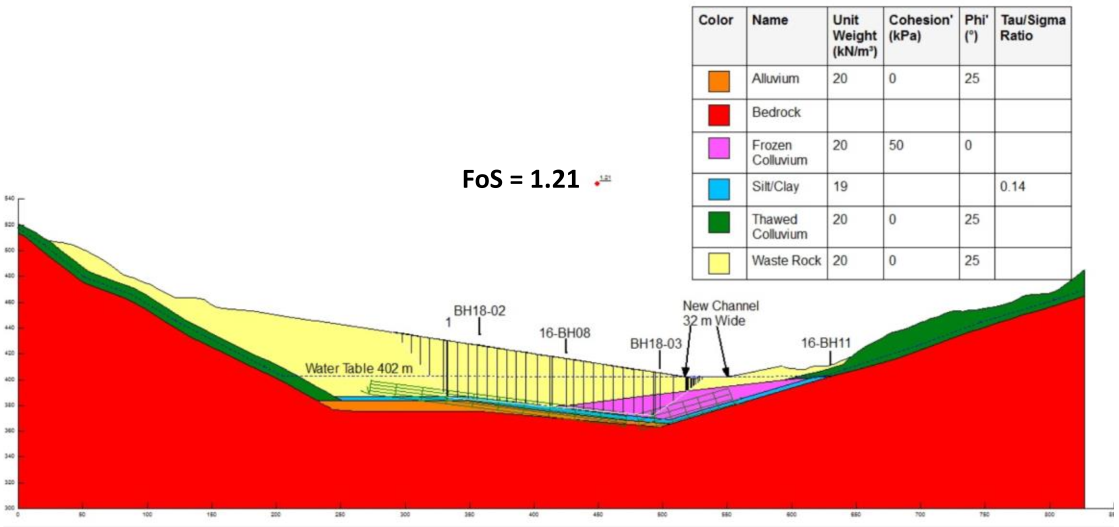
Color	Name	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Tau/Sigma Ratio
	Alluvium	20	0	25	
	Bedrock				
	Silt/Clay	19			0.14
	Thawed Colluvium	20	0	25	
	Waste Rock	20	0	25	
	Waste Rock (liq)	20			0.1
	Weak Colluvium (liq)	20			0.1

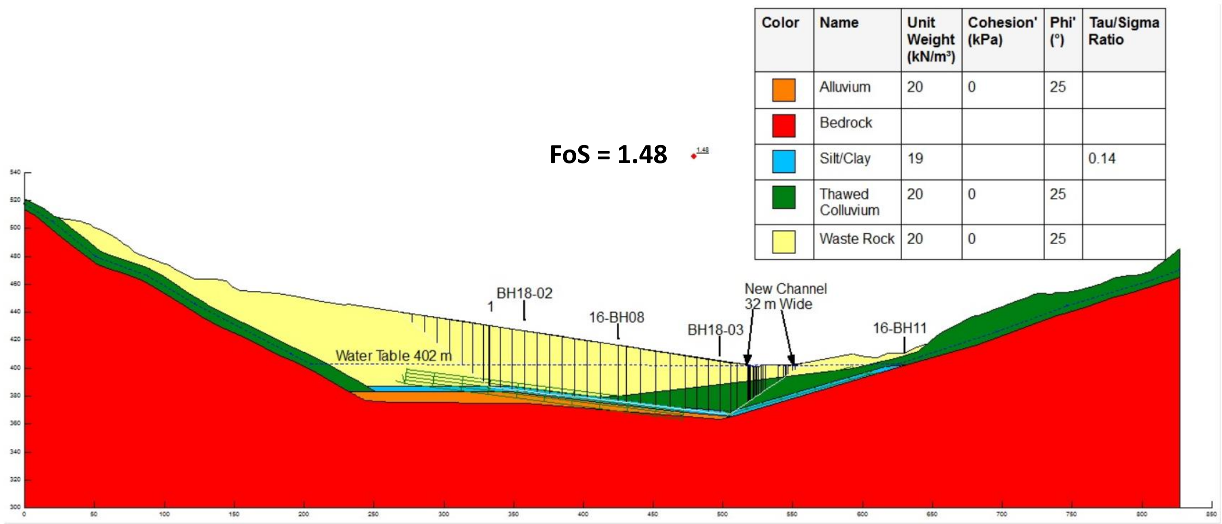
- Determined required slopes for DS4, assuming post-liquefaction strength as follows:
  - If a single layer at the water table, daylighting in the slope develops a post-liquefied strength, the FoS = 1.2
  - If all waste rock below the water table and the colluvium develop post-liquefied strength, the FoS drops below 1.0

SUMMARY: If the entire mass of wastes and thawed silty colluvium below Elevation 410 m mobilize an undrained liquefied strength ratio of 0.1 the FoS is 0.83. If the liquefaction is restricted to a layer either just below the water table or just above the alluvium/bedrock then the FoS as presented in previous analysis is acceptable for an overall 6.5H:1V slope.

CC2 - Lake at El. 402 m

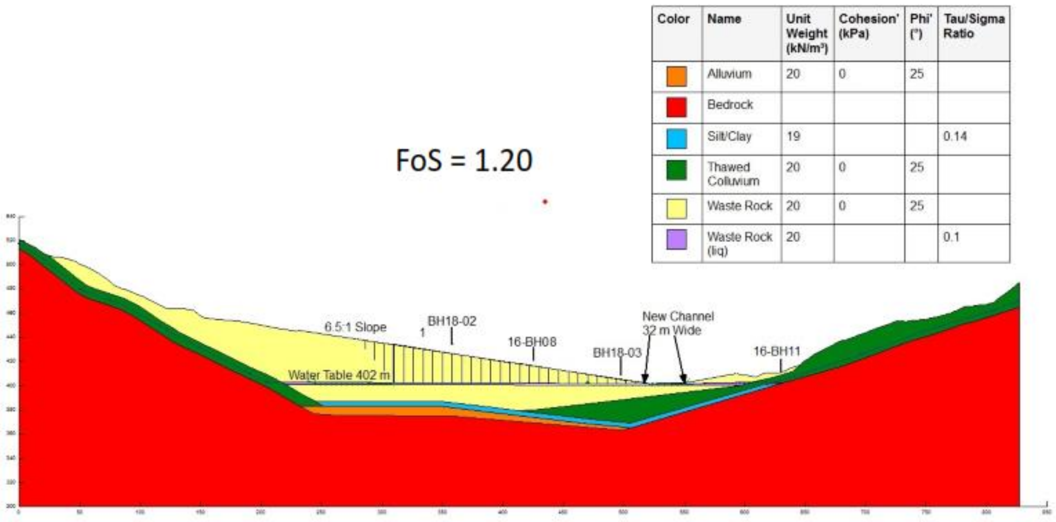
32 m wide channel, 6.5H:1V south slope

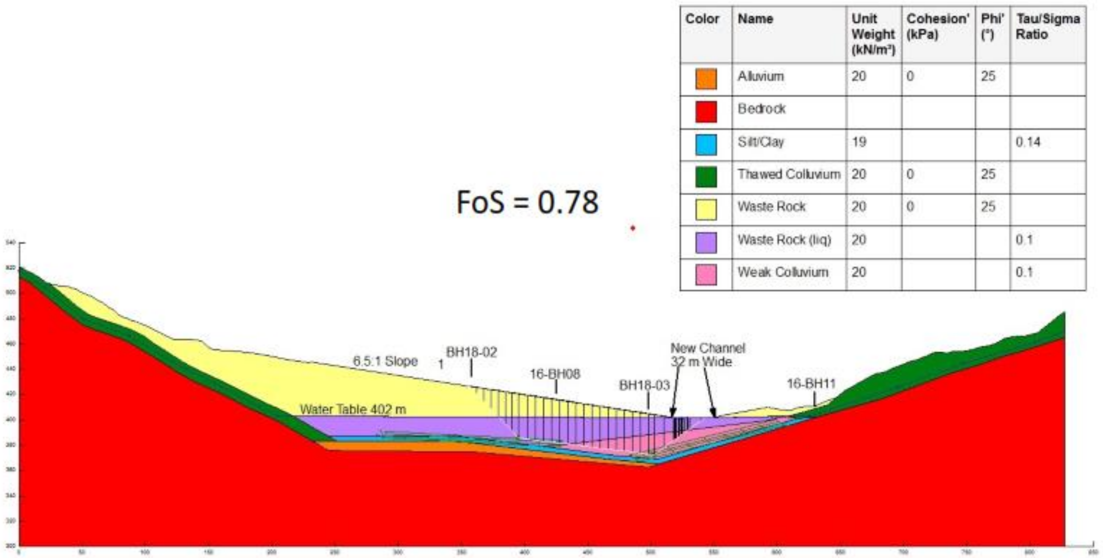




- Determined required slopes for DS4 as follows:
  - Short-term stability governed by frozen ice-rich colluvium provided FoS=1.2 for 6.5H:1V slopes
  - Long term stability improves with thawed colluvium using $\phi=25^{o}$

32 m wide channel, 6.5H:1V south slope

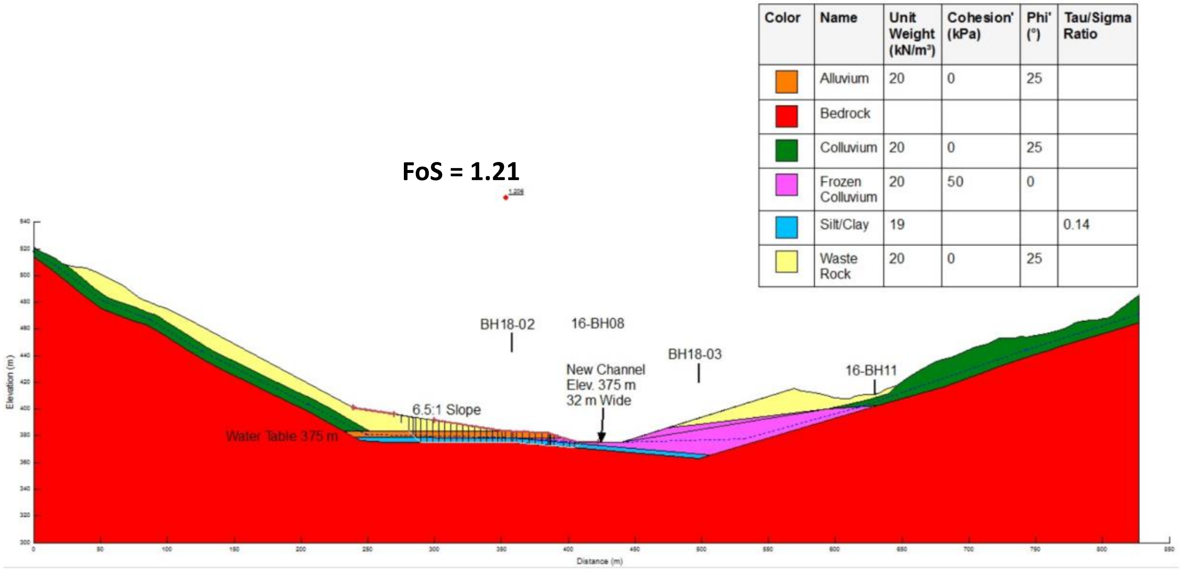


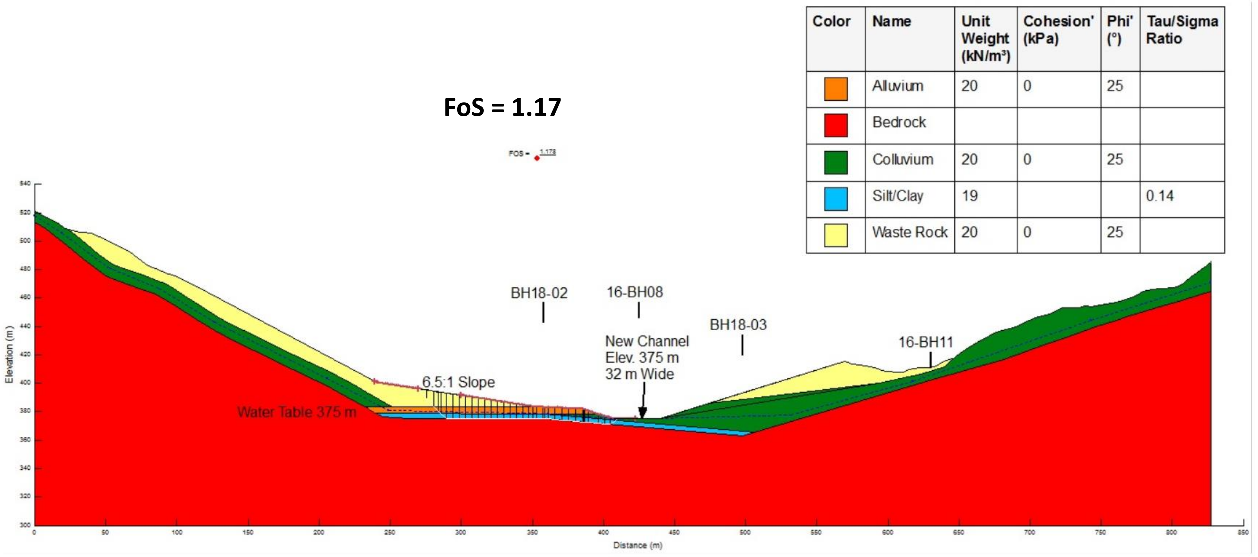


- Determined required slopes for DS4, assuming post-liquefaction strength as follows:
  - If a single layer at the water table, daylighting in the slope develops a post-liquefied strength, the FoS = 1.2
  - If all waste rock below the water table and the colluvium develop post-liquefied strength, the FoS drops below 1.0

CC3 - No Lake

32 m wide channel, 6.5H:1V south slope



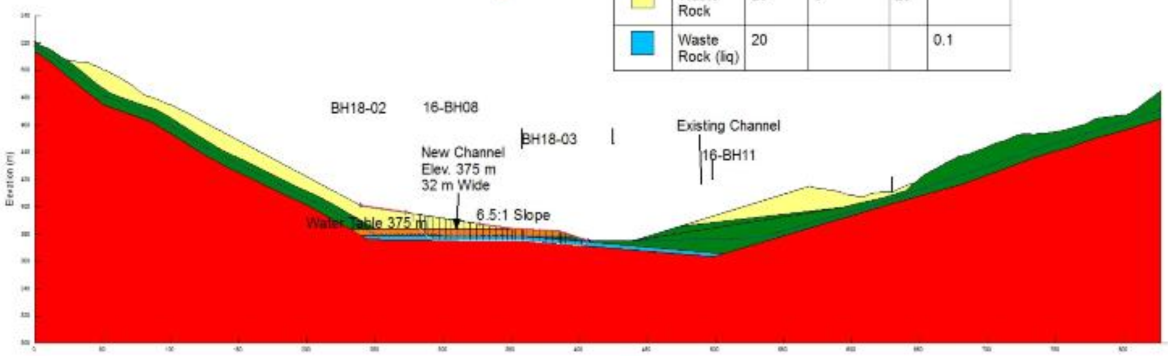


- Determined required slopes for DS4 as follows:
  - Short-term stability governed by frozen ice-rich colluvium provided FoS=1.2 for 6.5H:1V slopes
  - Long term stability does not improve with thawed colluvium using $\phi=25^o$

32 m wide channel, 6.5H:1V south slope

Color	Name	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Tau/Sigma Ratio
	Alluvium	20	0	25	
	Bedrock				
	Colluvium	20	0	25	
	Waste Rock	20	0	25	
	Waste Rock (liq)	20			0.1

FoS = 0.95



- Liquefaction of waste not likely to impact slopes as most waste is removed
- If a thin layer of alluvium or colluvium has a post-liquefied strength of c/p'=0.1 applied:
  - FoS is 0.95 for the case with thawed colluvium
- However, volume of material not likely to re-block channel

Appendix III

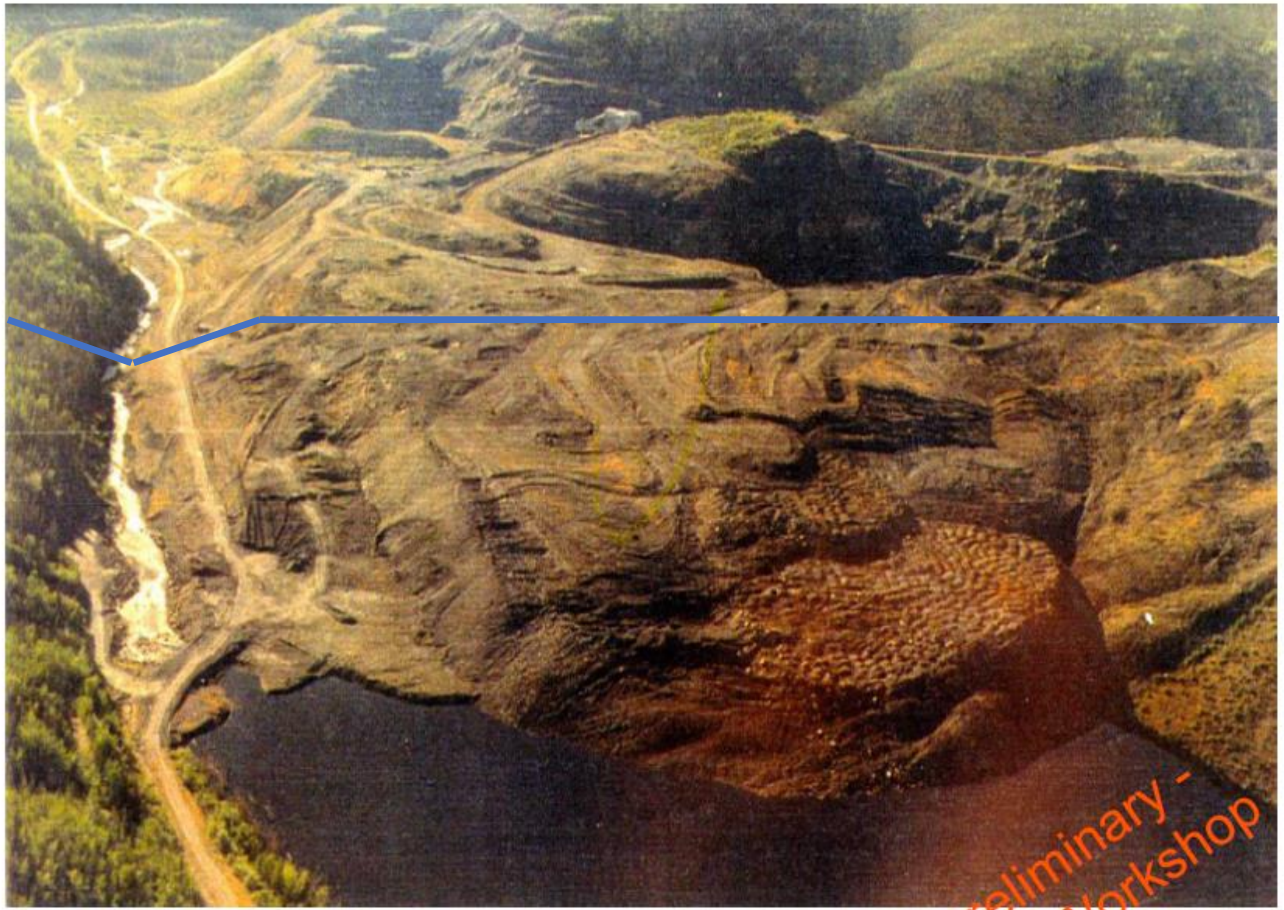
DS2 Slope Stability Summary

DS2 - Slope Stability Summary

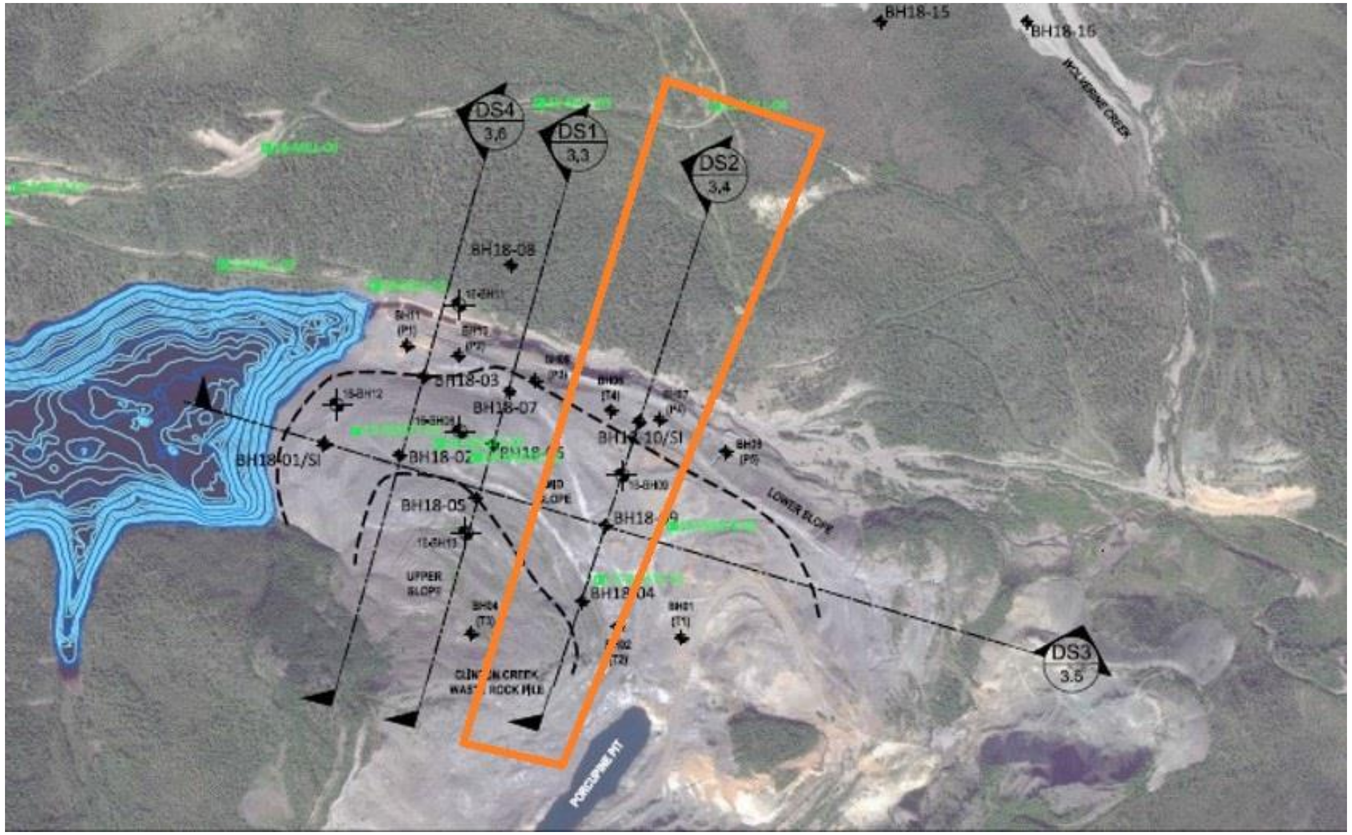
Historical Review of Waste Pile

1985

Water El.
411.5m

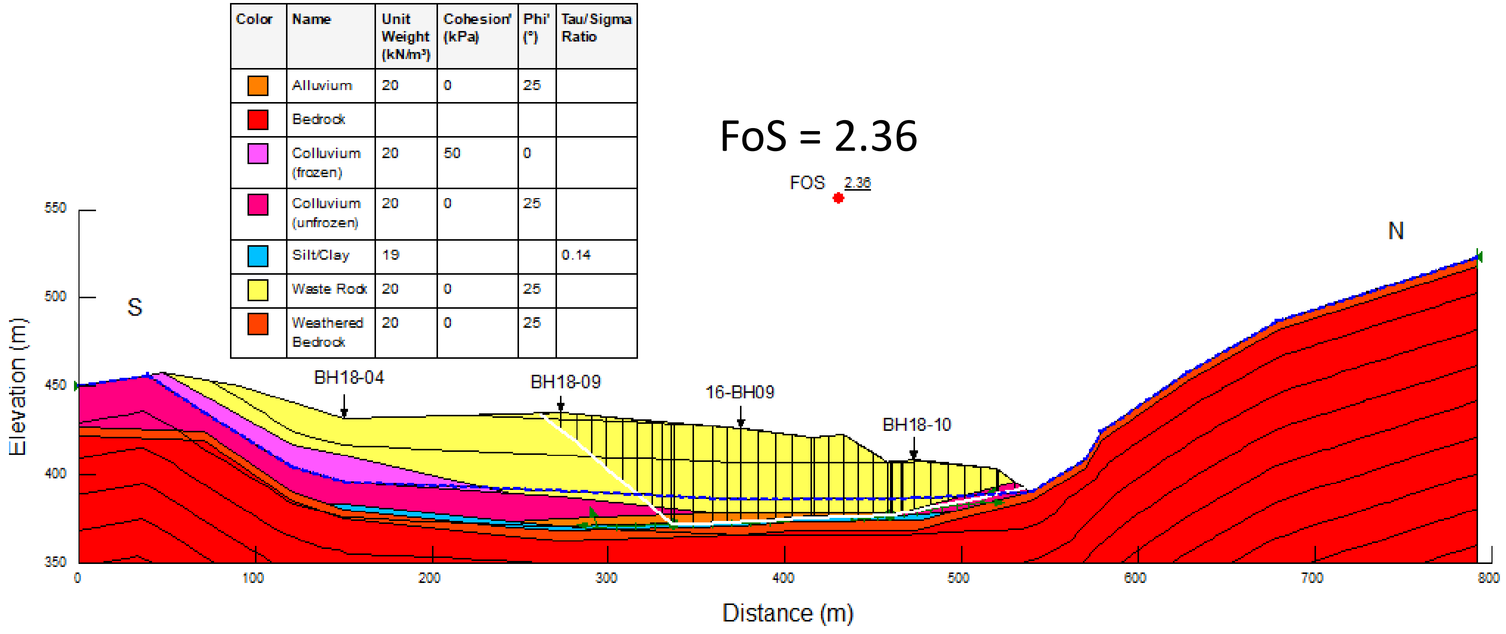


DS2 Section



Color	Name	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Tau/Sigma Ratio
	Alluvium	20	0	25	
	Bedrock				
	Colluvium (ice-rich frozen)	20	50	0	
	Colluvium (unfrozen)	20	0	25	
	Silt/Clay	19			0.14
	Waste Rock	20	0	25	
	Weathered Bedrock	20	0	25	

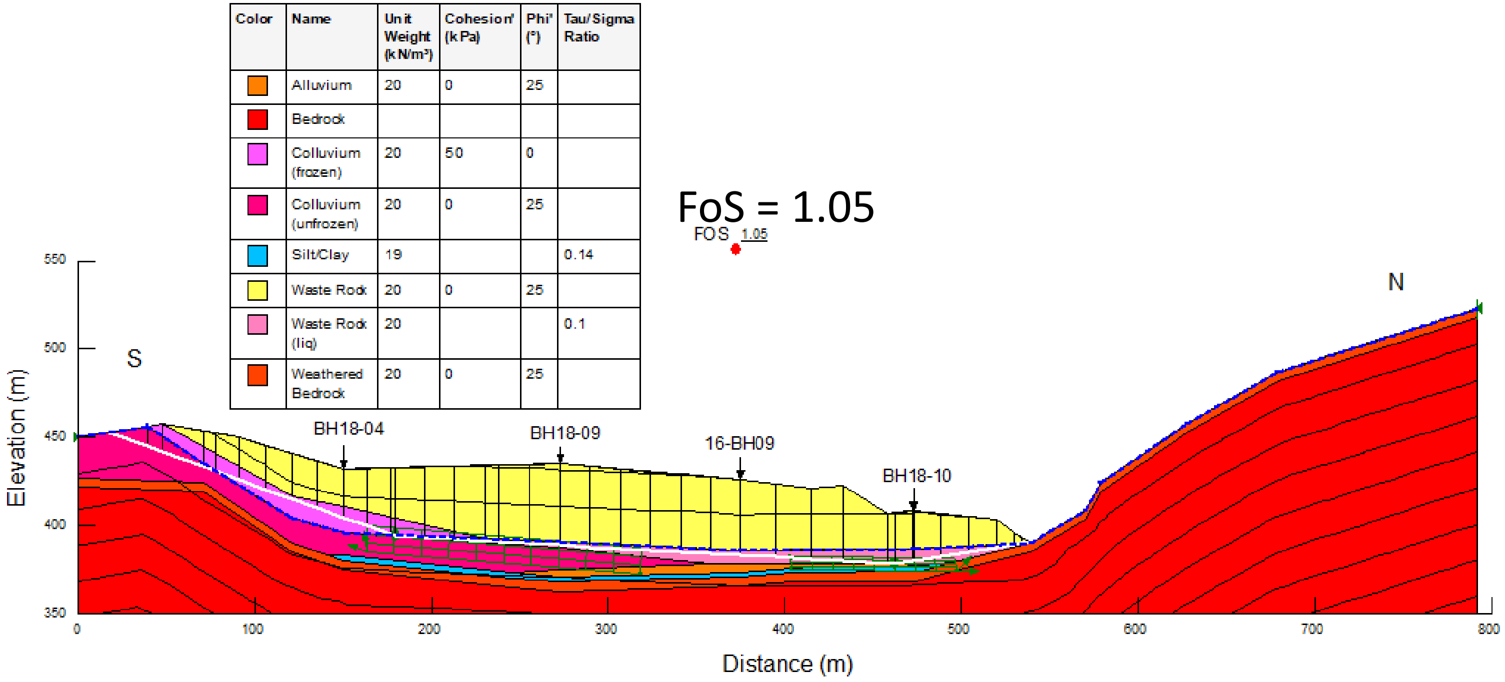
Base Case – Current Slope Geometry



Color	Name	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Tau/Sigma Ratio
	Alluvium	20	0	25	
	Bedrock				
	Colluvium (frozen)	20	50	0	
	Colluvium (unfrozen)	20	0	25	
	Silt/Clay	19			0.14
	Waste Rock	20	0	25	
	Weathered Bedrock	20	0	25	

- Current Slope Geometry (nominally 4H:1V)
- Phreatic surface of ~386 m based on piezometers at location
- Slip surface though silt-clay
- Results in FoS > 2.0 for the overall static stability.

Base Case – Current Slope Geometry

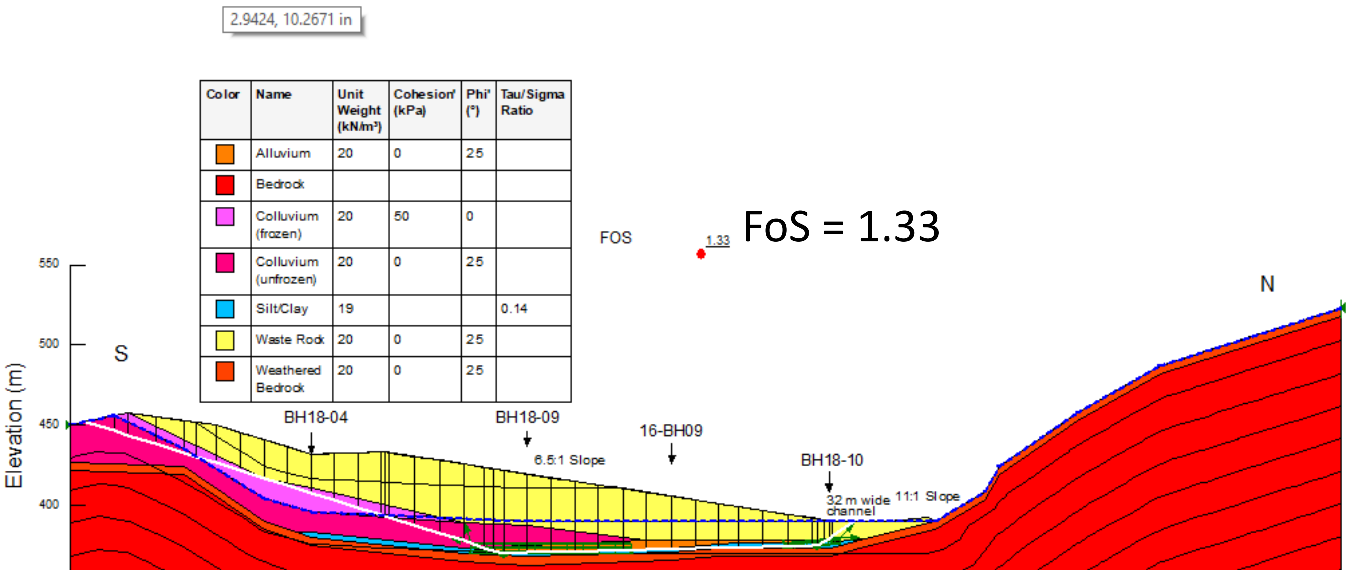


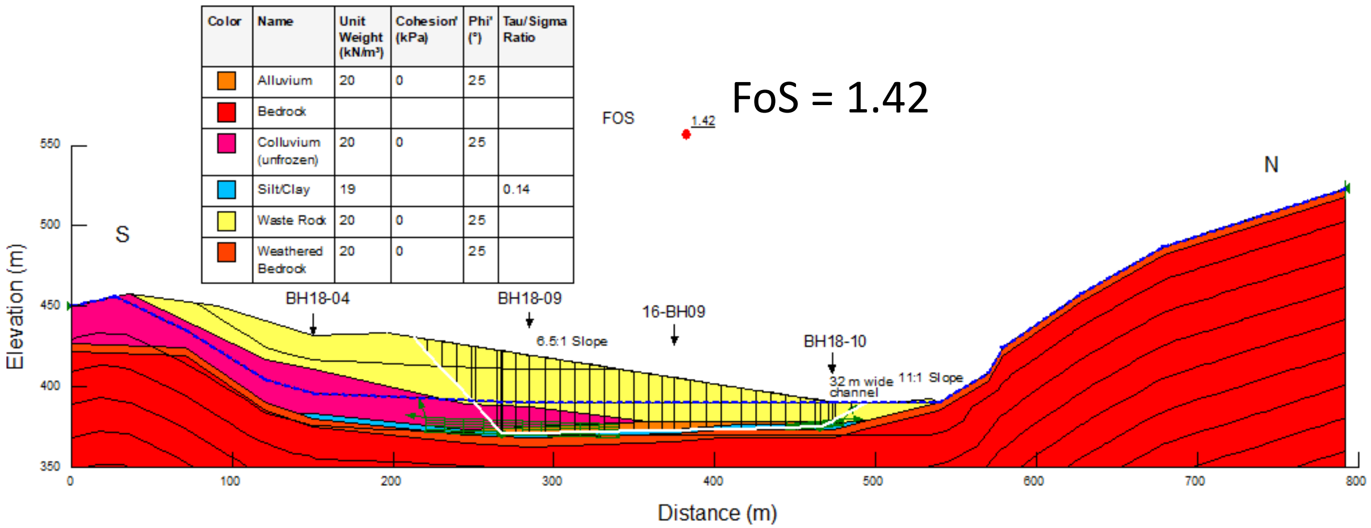
Color	Name	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Tau/Sigma Ratio
	Alluvium	20	0	25	
	Bedrock				
	Colluvium (frozen)	20	50	0	
	Colluvium (unfrozen)	20	0	25	
	Silt/Clay	19			0.14
	Waste Rock	20	0	25	
	Waste Rock (liq)	20			0.1
	Weathered Bedrock	20	0	25	

- Current Slope Geometry (nominally 4H:1V)
- Phreatic surface of ~386 m based on piezometers at location
- Waste Material below water table liquefied
- Short term colluvium frozen

CC1 - Lake at El. 412 m

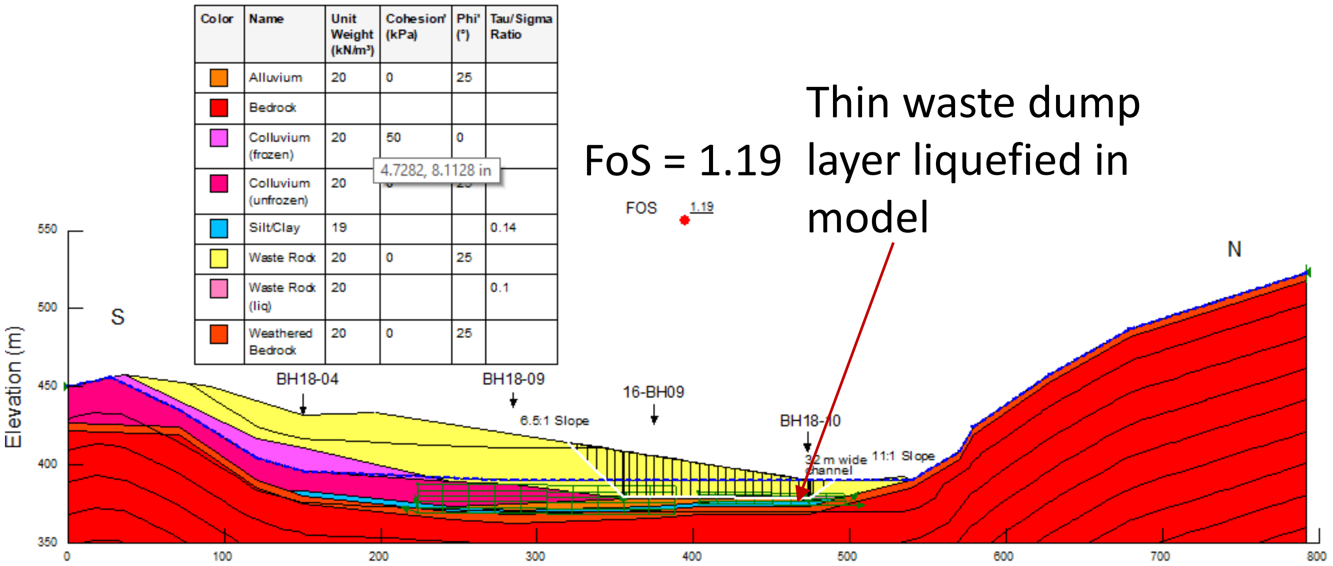
6.5H:1V Overall Slope

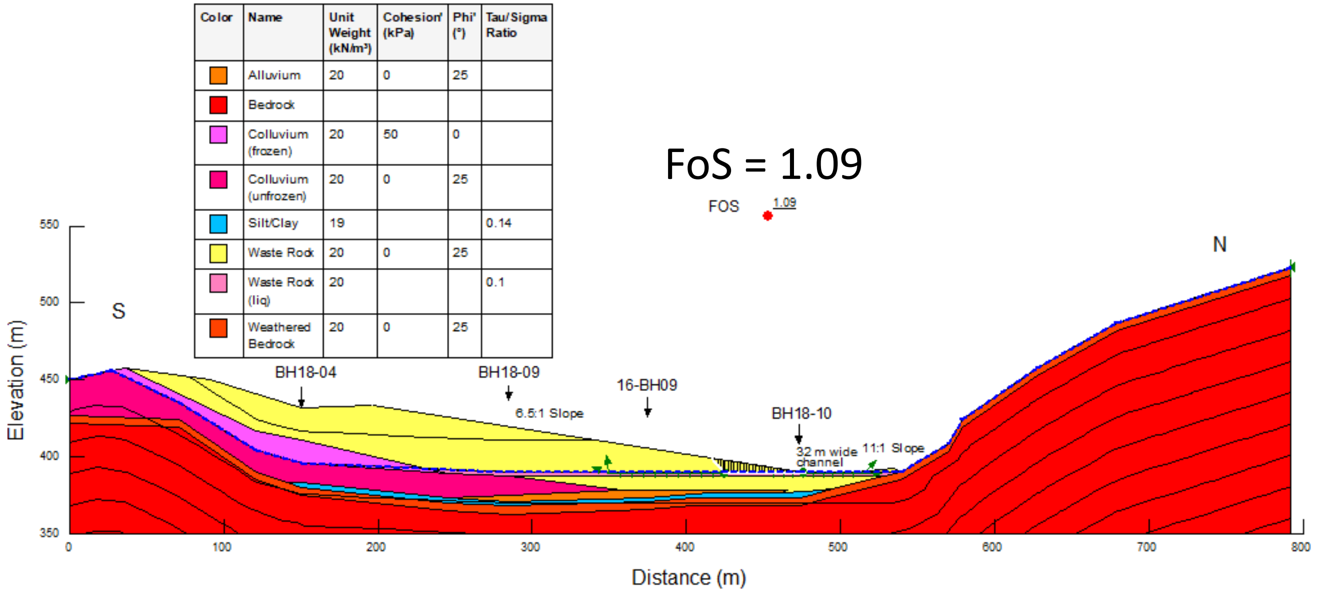




- 32 m wide channel installed on north side of waste material pile with a 6.5:1 side slope
- Lake elevation remains at 412 m
- Static FoS, waste material not liquefying. Silt clay mobilizing c/p’ = 0.14
- Short term static - frozen colluvium 50 kPa FoS = 1.33
- Long term static – thawed colluvium FoS = 1.42

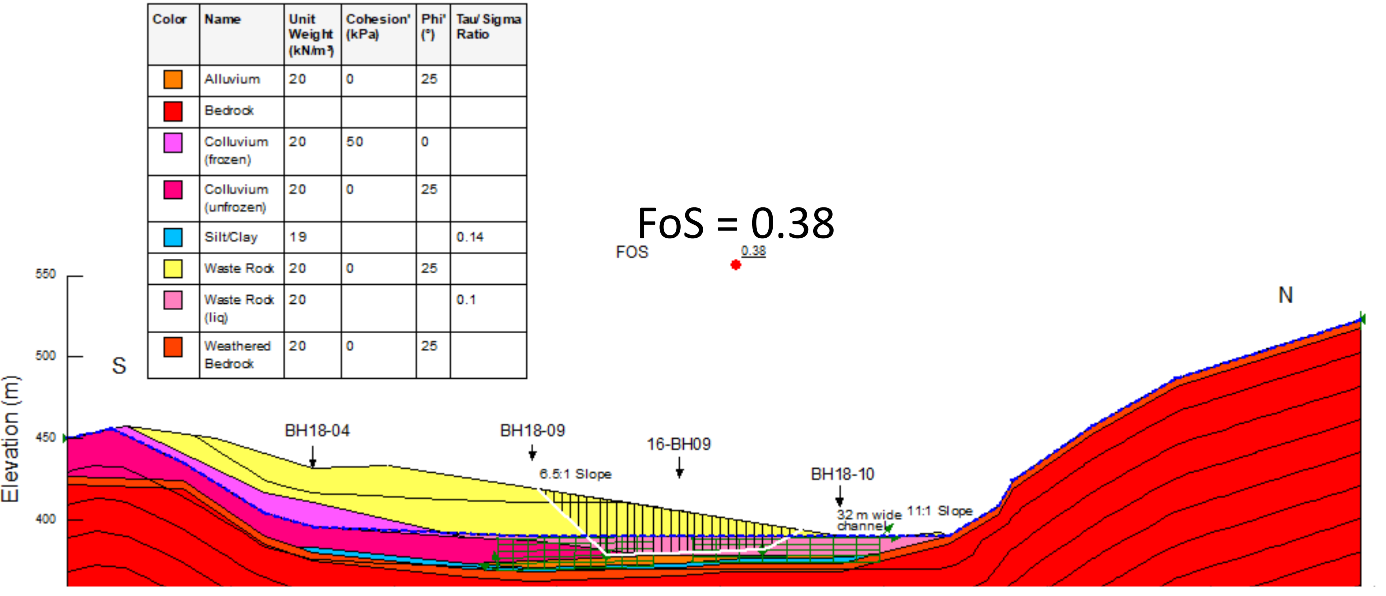
6.5H:1V Overall Slope

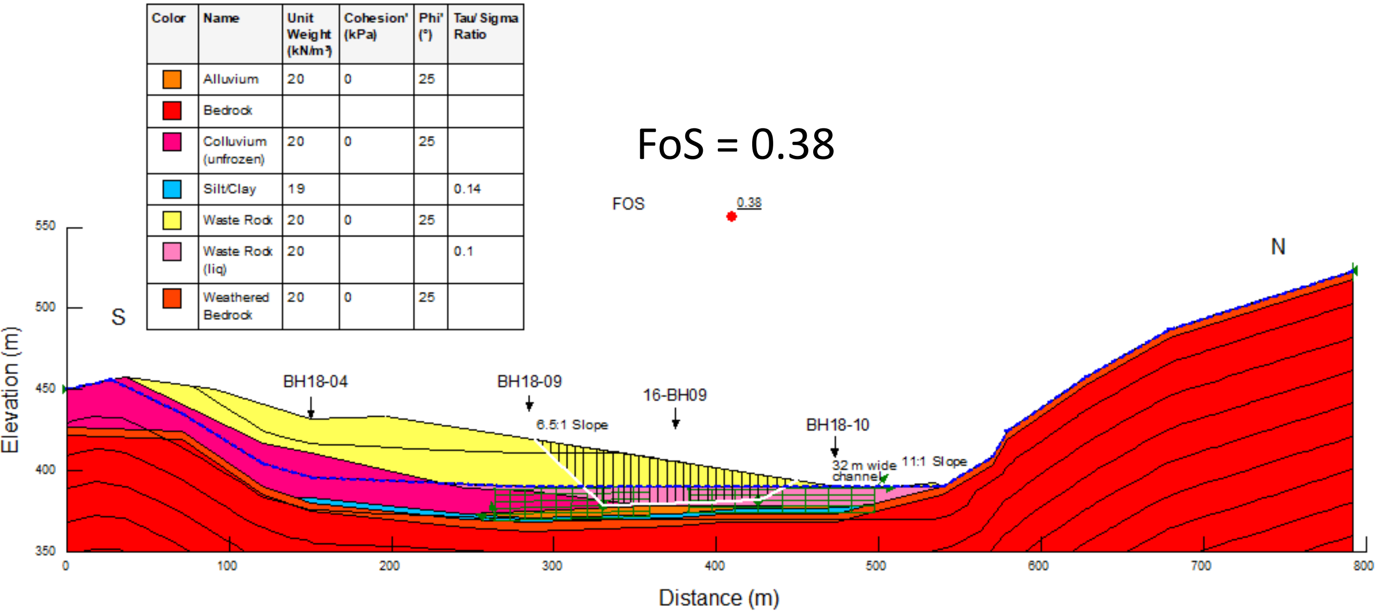




- 32 m wide channel installed on north side of waste pile with a 6.5H:1V slope
- Lake elevation at 412 m
- Thin layer of waste rock along the valley bottom given liquified parameters, FoS = 1.19
- Thin layer of waste rock just below the phreatic surface given liquefied parameters, FoS = 1.09
- FoS does not change for long term conditions of thawed colluvium (failures do not go through frozen colluvium)

6.5H:1V Overall Slope

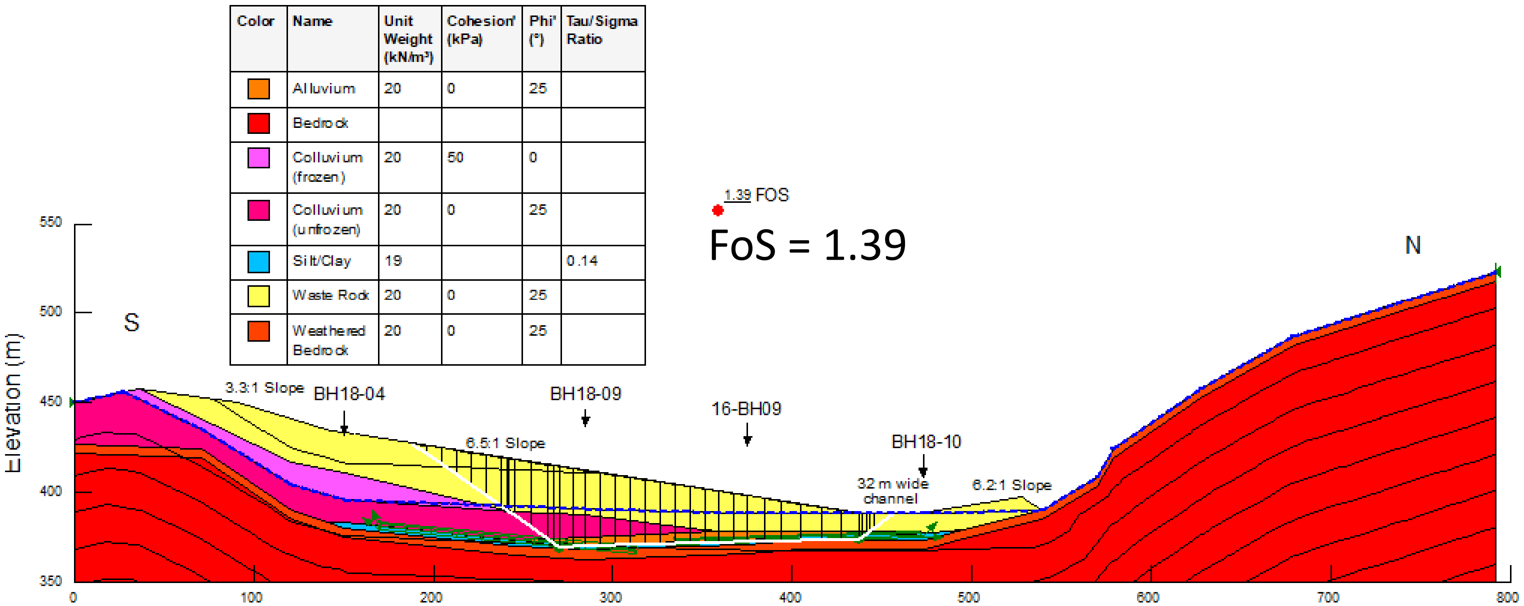


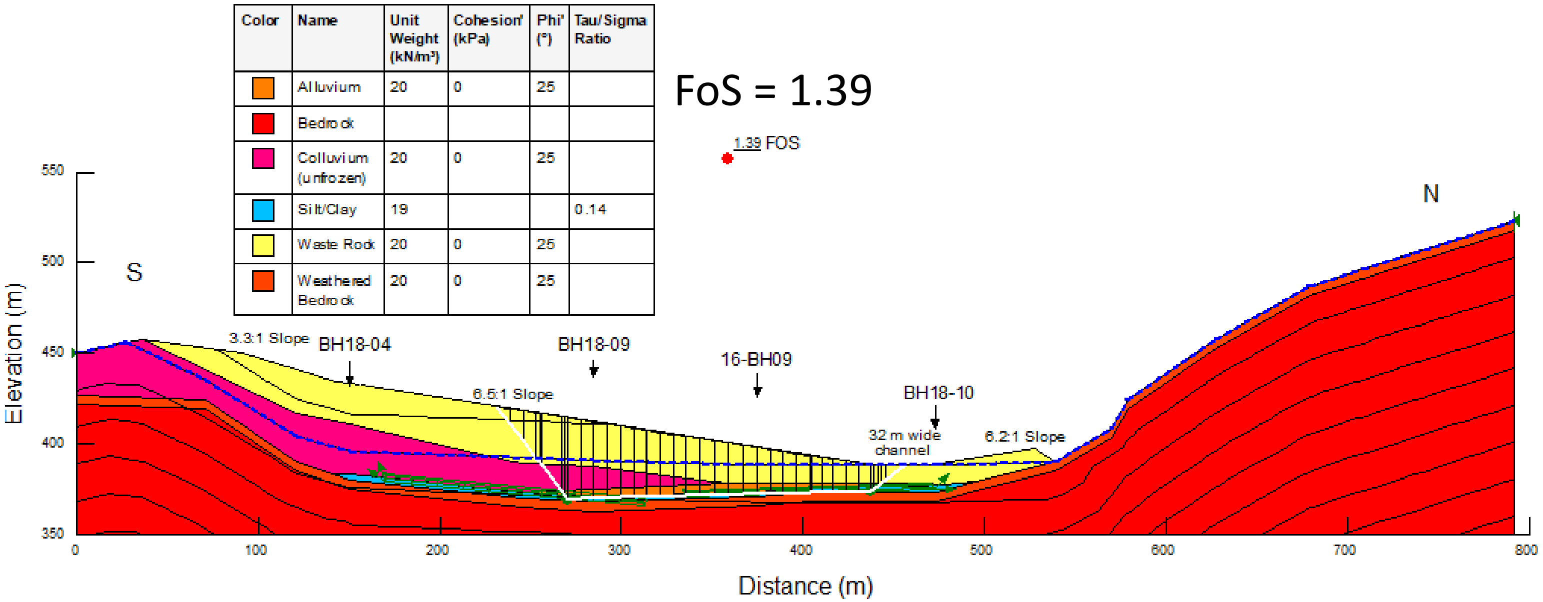


- 32 m wide channel installed on north side of waste pile with a 6.5H:1V slope
- Lake elevation at 412 m
- All waste rock below the phreatic surface liquefies with frozen colluvium (short term case), FoS = 0.38
- All waste rock below the phreatic surface liquefies with thawed colluvium (long term case), FoS = 0.38

CC2 - Lake at El. 402 m

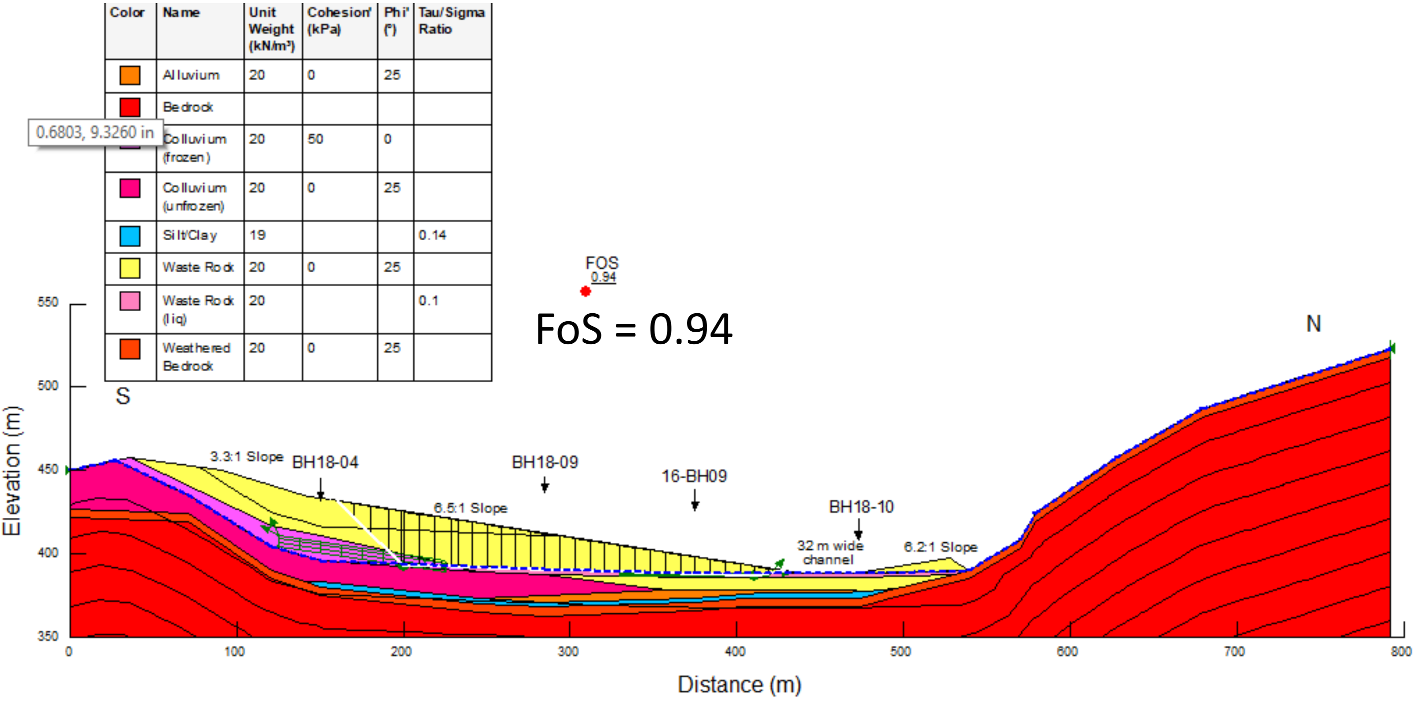
6.5H:1V Overall Slope

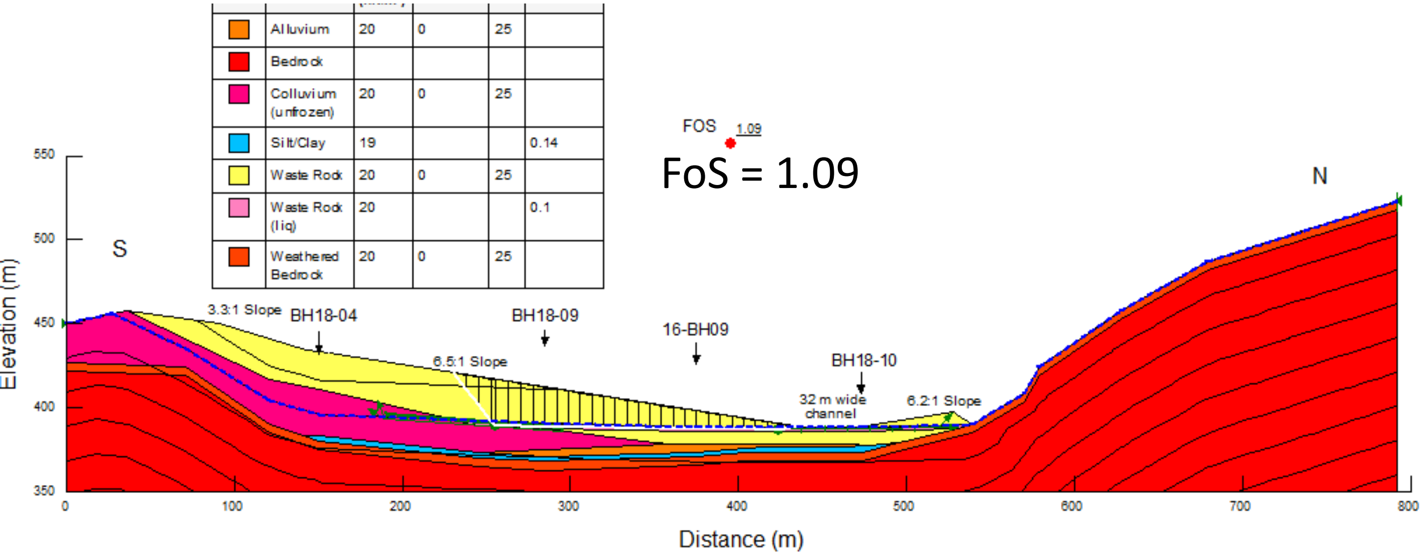




- Lake water elevation dropped to 402 m
- 32 m wide channel goes through waste material pile with side slope of 6.5:1
- Failure surface through bottom of valley in silt clay.
- FoS remains unchanged for short and long term conditions

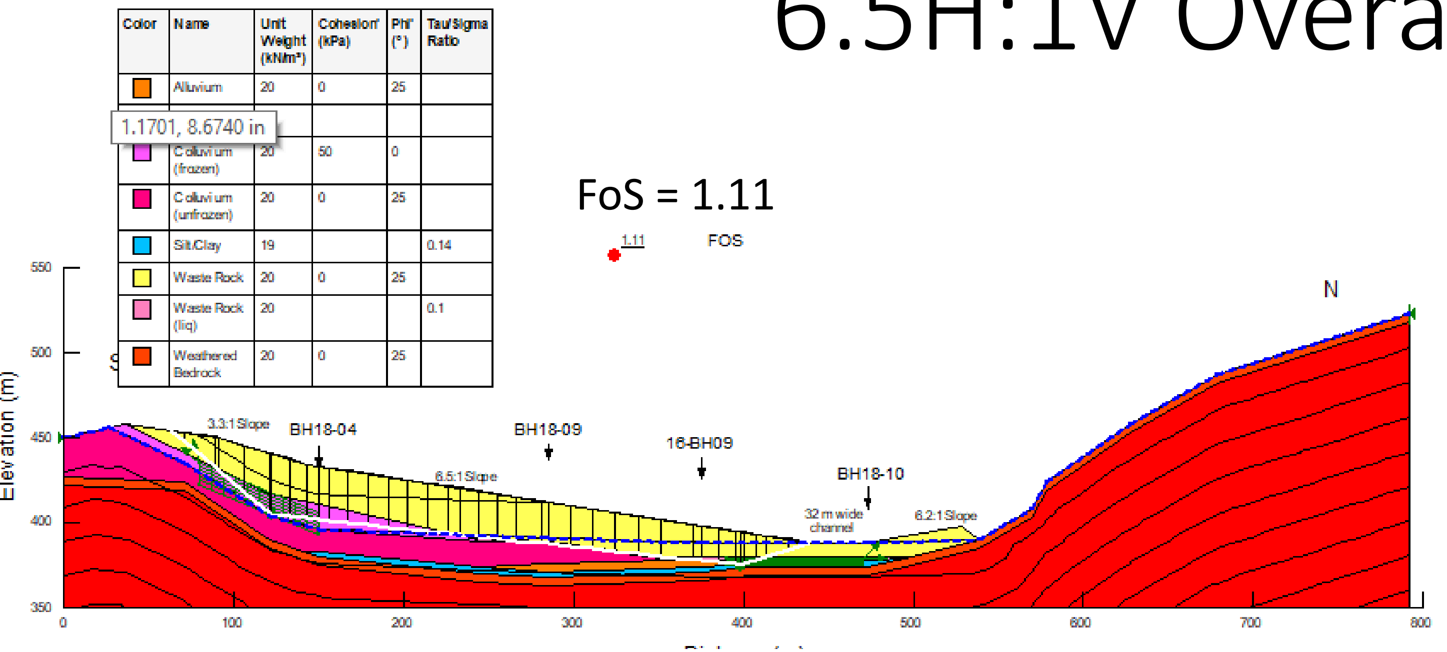
6.5H:1V Overall Slope

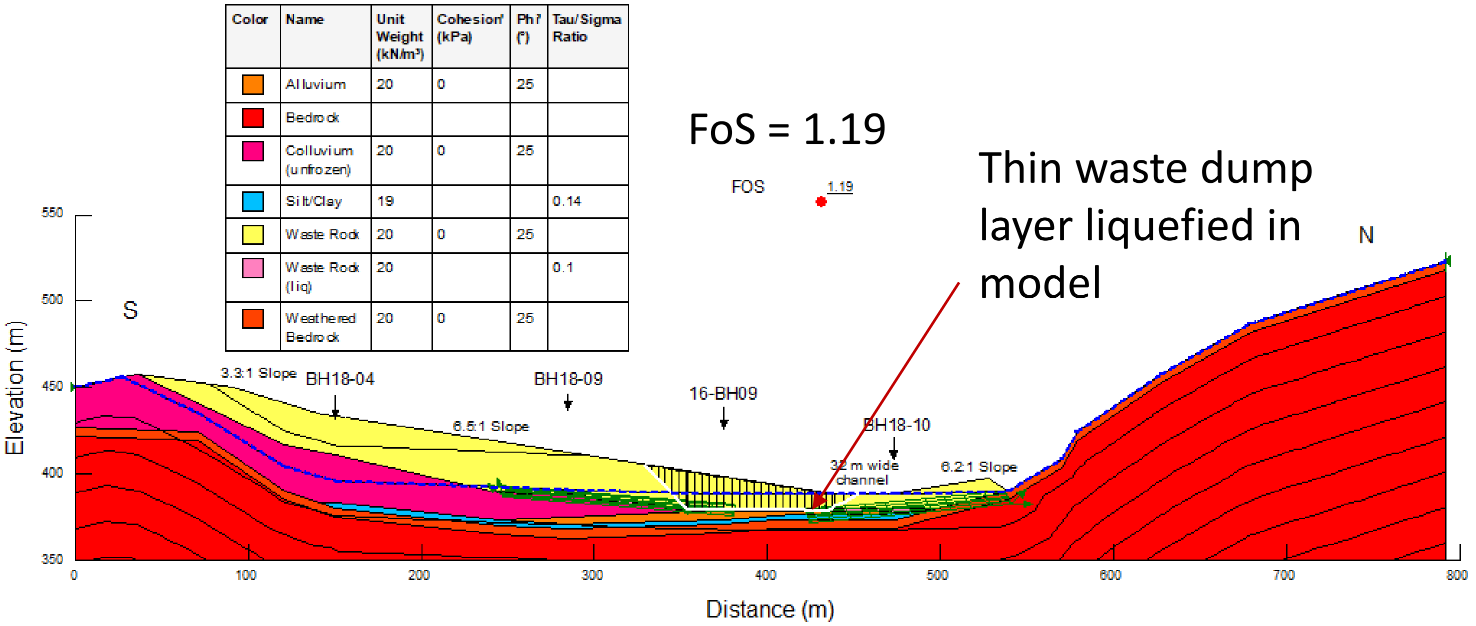




- Lake water elevation dropped to 402 m
- 32 m wide channel goes through waste material pile with side slope of 6.5:1
- Thin layer of waste rock just below the phreatic surface given liquefied parameters:
  - Short term, FoS = 0.94
  - Long term, FoS = 1.09

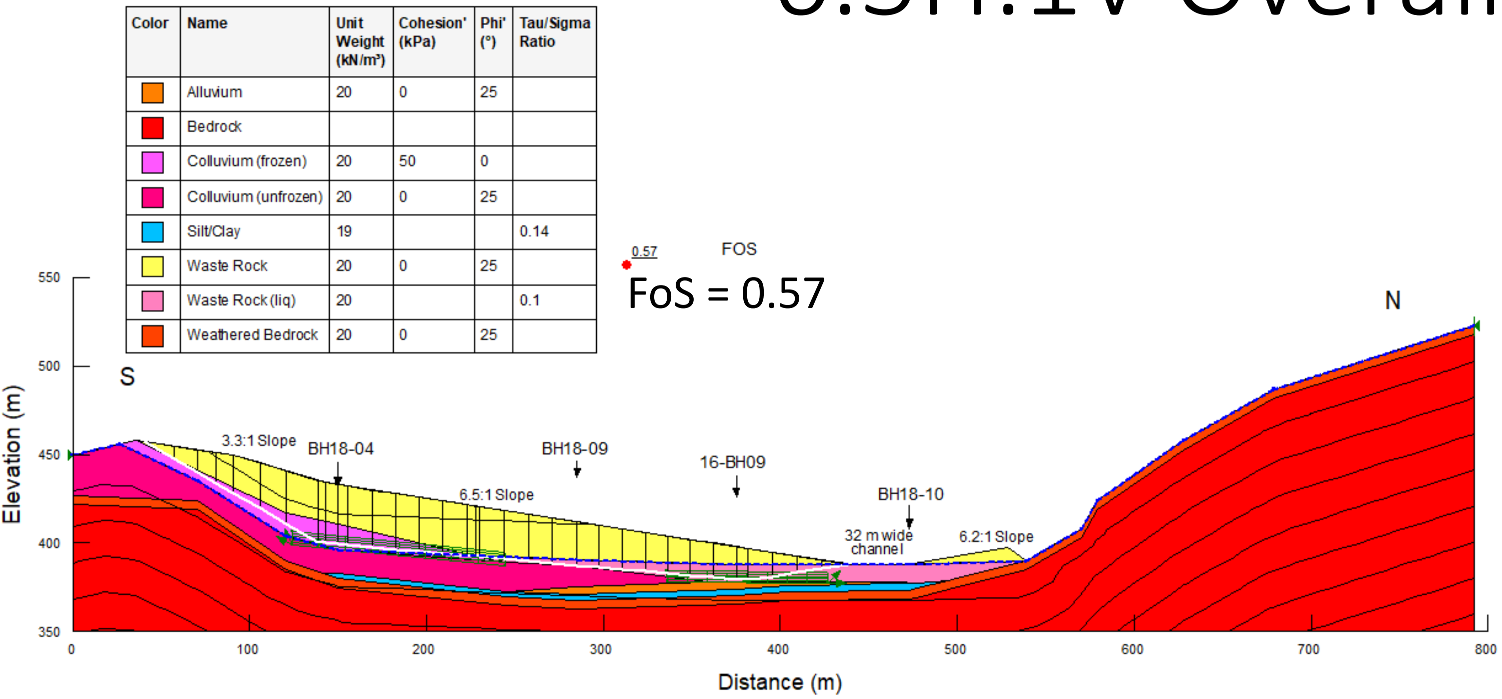
6.5H:1V Overall Slope



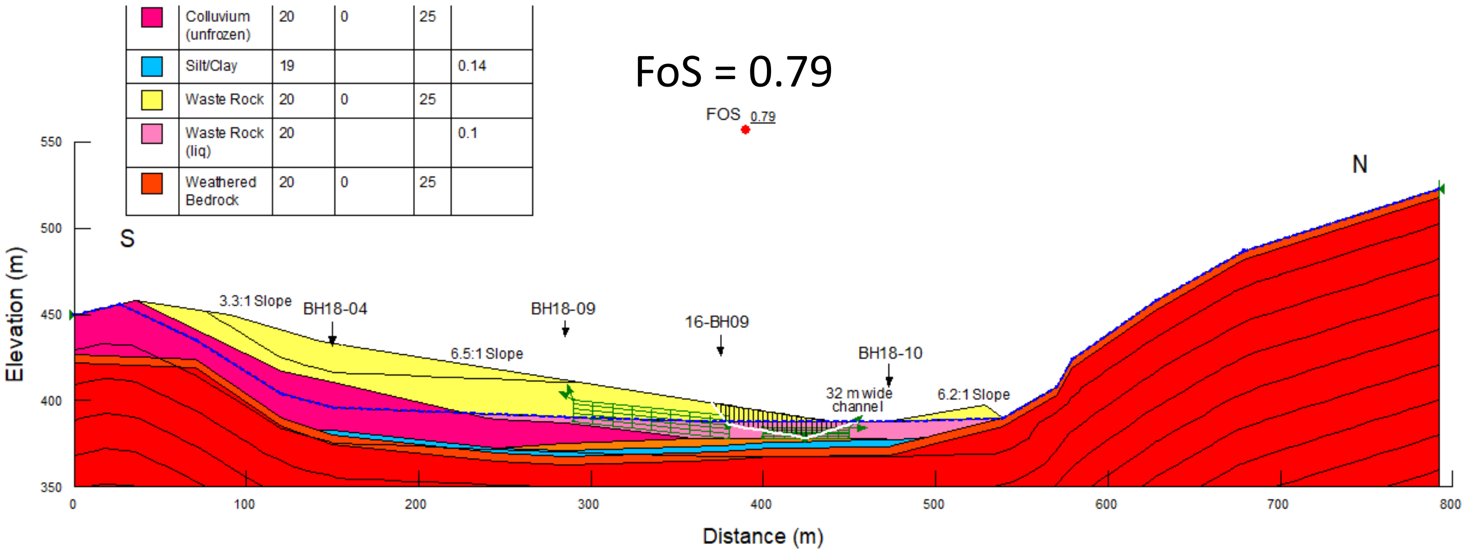


- Lake water elevation dropped to 402 m
- 32 m wide channel goes through waste material pile with side slope of 6.5:1
- Thin layer of waste rock along the base of the valley given liquefied parameters:
  - Short term, FoS = 1.11
  - Long term, FoS = 1.19

6.5H:1V Overall Slope



Color	Name	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Tau/Sigma Ratio
	Alluvium	20	0	25	
	Bedrock				
	Colluvium (frozen)	20	50	0	
	Colluvium (unfrozen)	20	0	25	
	Silt/Clay	19			0.14
	Waste Rock	20	0	25	
	Waste Rock (liq)	20			0.1
	Weathered Bedrock	20	0	25	

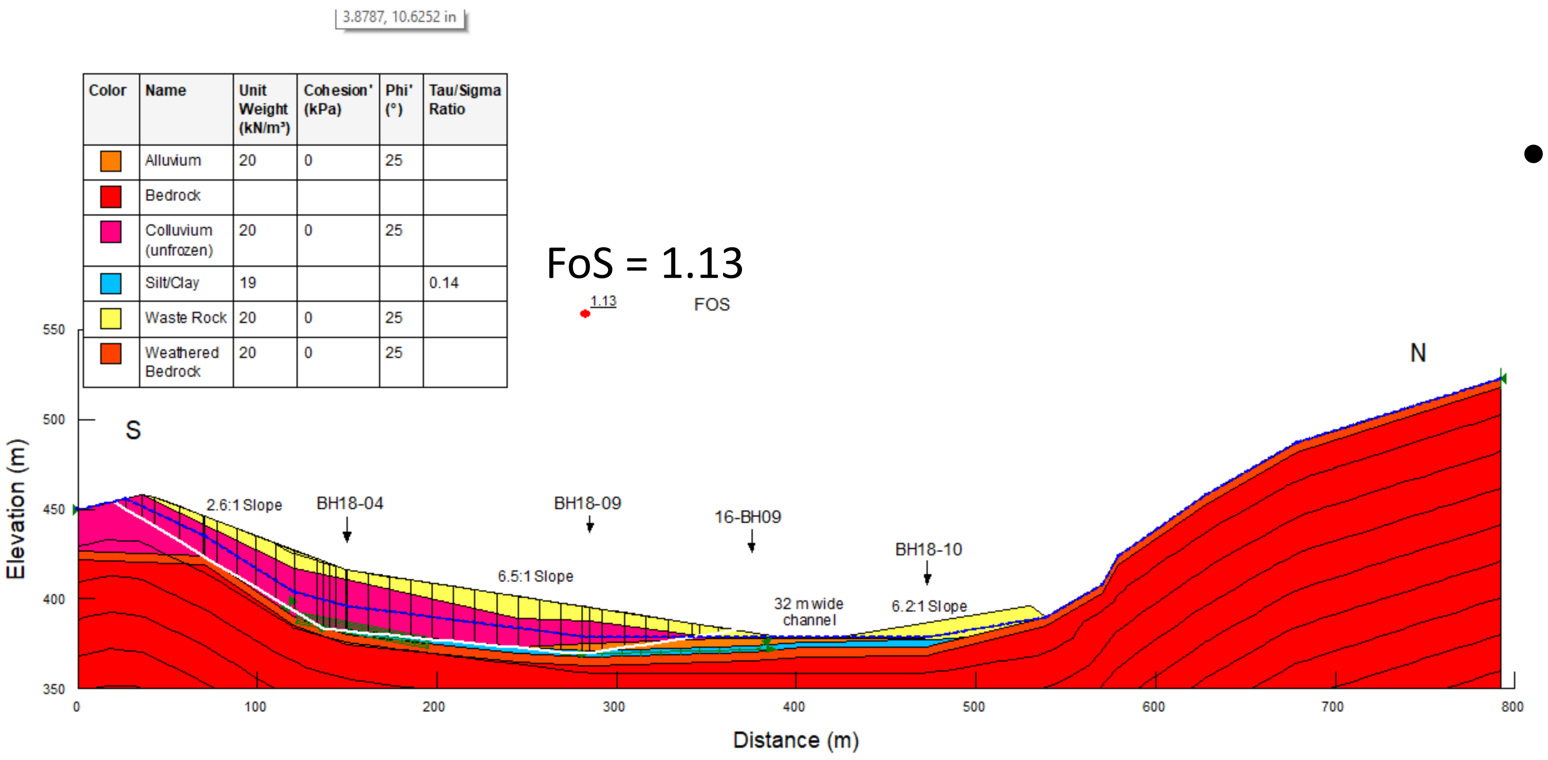


	Colluvium (unfrozen)	20	0	25	
	Silt/Clay	19			0.14
	Waste Rock	20	0	25	
	Waste Rock (liq)	20			0.1
	Weathered Bedrock	20	0	25	

- Lake water elevation dropped to 402 m
- 32 m wide channel goes through waste material pile with side slope of 6.5:1
- All waste material below the phreatic surface liquefied
- Short term (frozen colluvium), FoS = 0.57
- Long therm (thawed colluvium), FoS = 0.79

CC3 - No Lake

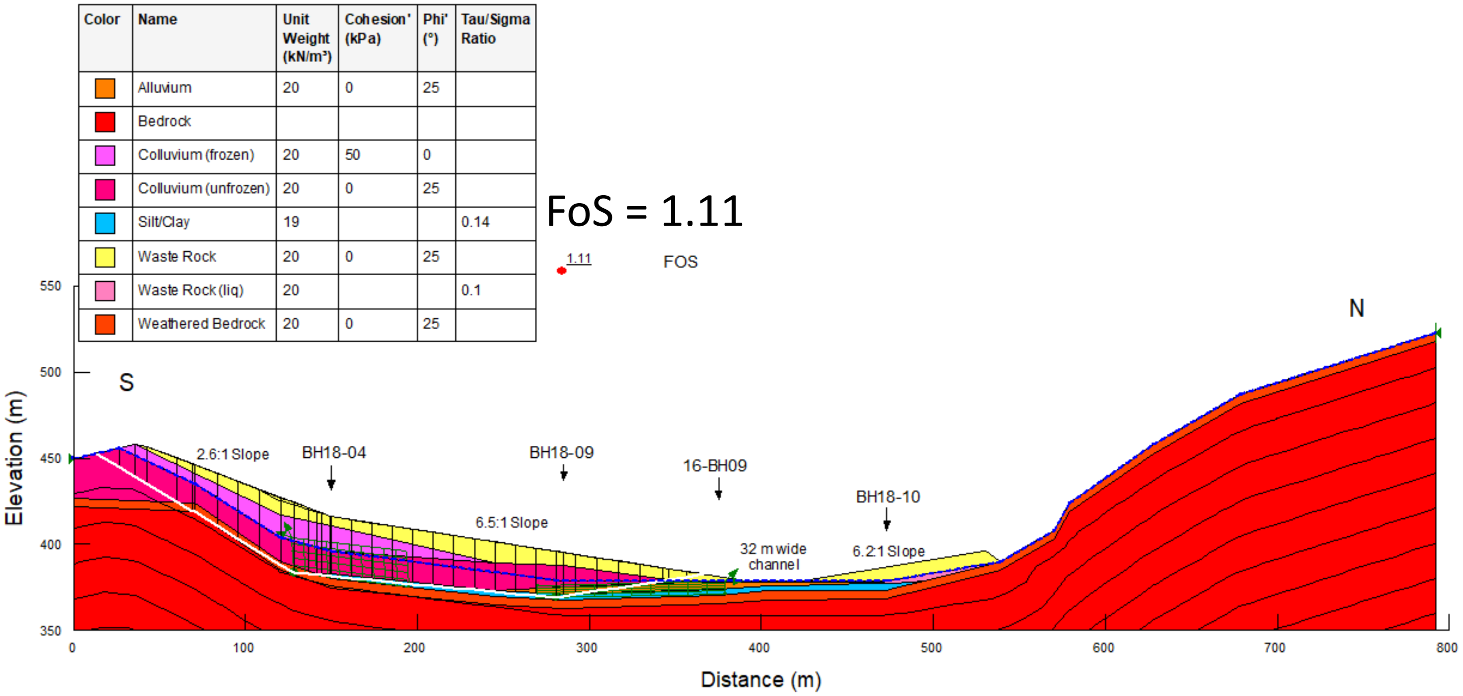
6.5H:1V Overall Slope



Color	Name	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Tau/Sigma Ratio
	Alluvium	20	0	25	
	Bedrock				
	Colluvium (unfrozen)	20	0	25	
	Silt/Clay	19			0.14
	Waste Rock	20	0	25	
	Weathered Bedrock	20	0	25	

- No lake option has 32 m wide channel at El 379 m
- Long term stability through the frozen colluvium results in FoS = 1.13

6.5H:1V Overall Slope



Color	Name	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Tau/Sigma Ratio
	Alluvium	20	0	25	
	Bedrock				
	Colluvium (frozen)	20	50	0	
	Colluvium (unfrozen)	20	0	25	
	Silt/Clay	19			0.14
	Waste Rock	20	0	25	
	Waste Rock (liq)	20			0.1
	Weathered Bedrock	20	0	25	

- No lake option has 32 m wide channel at El 379 m
- Waste material below phreatic surface is liquefied
- Liquefied waste material section is so small that liquefaction causes negligible difference in FoS, as presumed silt clay governs

Appendix IV

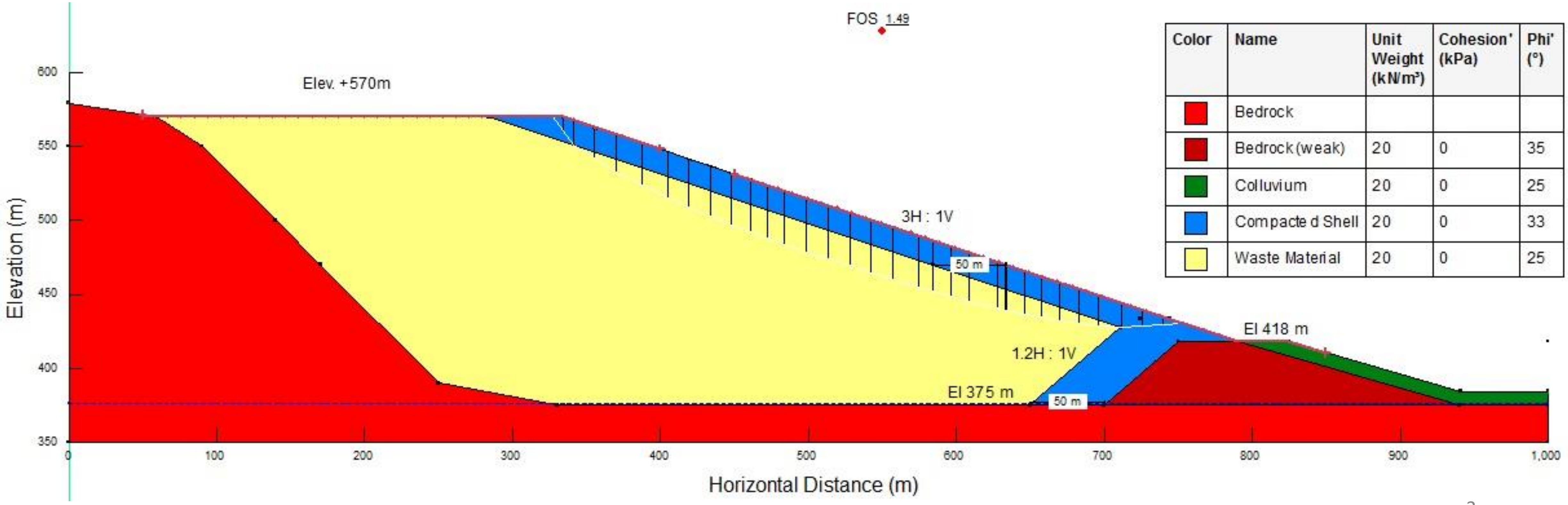
Porcupine Pit Shell Summary

Porcupine Pit with Outer Shell - Stability Assessment

Store waste material in Porcupine Pit with a 50 m thick layer of compacted waste material on the outside

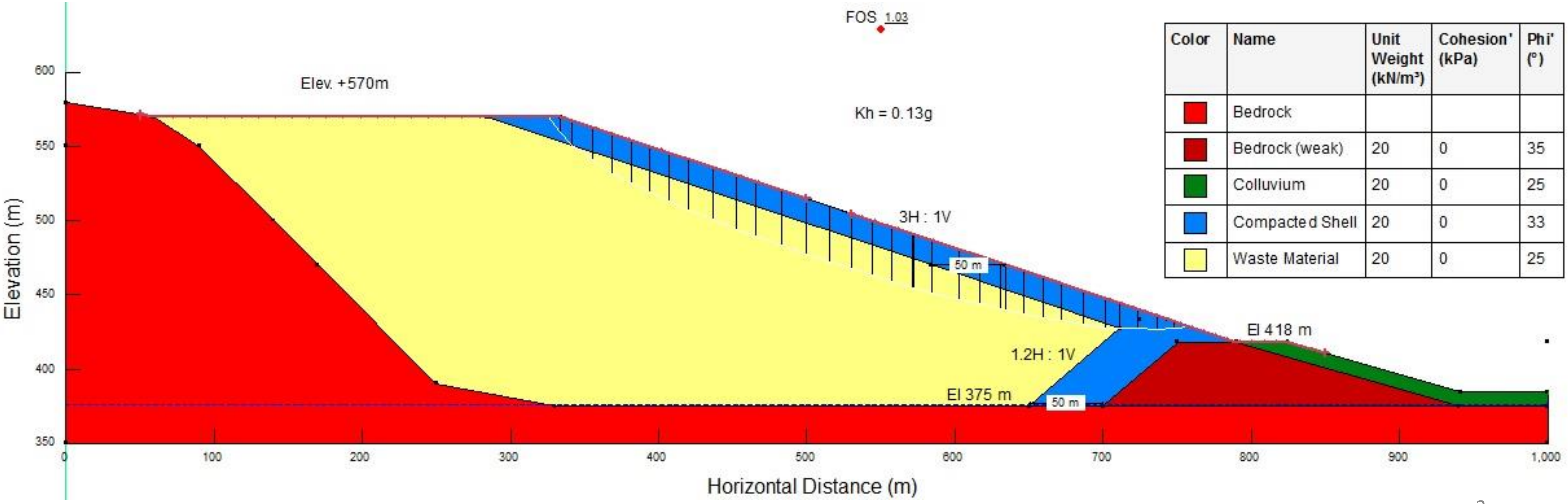
Porcupine Pit – Static Analysis

FoS = 1.49



Porcupine Pit – Seismic Analysis

FoS = 1.03



Color	Name	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)
	Bedrock			
	Bedrock (weak)	20	0	35
	Colluvium	20	0	25
	Compacted Shell	20	0	33
	Waste Material	20	0	25

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a Division of Wood Canada Limited
600 – 4445 Lougheed Highway
Burnaby, BC V5C 0E4
Canada
T: 604-294-3811

Memo

To:	File		
From:	Surinder Garewal, MEng, PEng (BC)	**Reviewer:**	Ed McRoberts, PhD, PEng
cc:	Karen Hincks, MSc, PGeo (NT)	**Wood File No.:**	VE52705D.100.4
Date:	26 August 2019		
Re:	**Wolverine Creek Tailings Dump Assessment**		

1.0 Introduction

This memo addresses the geotechnical aspects pertaining to the closure concepts for the Wolverine Tailings Dump at the Clinton Creek mine, located 80 km northwest of Dawson City, Yukon. The tailings were placed in two main piles on the west slope of Wolverine Creek valley during the mine operations.

Subsequently both lobes of the tailings dump have failed downslope and have partially blocked Wolverine Creek, moving the creek up in elevation and to the east, causing impoundment of water on the upstream side of both lobes.

In 2016, the Project Partners selected three closure concepts for the Wolverine Creek side to be advanced to the 10% design phase. The closure concepts (part of the mine closure plan) aim to either leave the tailings in their existing profile, stabilize the tailings slopes and restore flows in the Wolverine Creek channel or remove all the tailings. This memo addresses the stability of the tailings pile slopes for the proposed three closure concepts.

For additional information on assumptions made for strength, liquefaction susceptibility and geology, this memorandum should be read in conjunction with the Geological & Geotechnical Site Characterization and Model (Wood, 2019) and the Design Basis Memorandum (DBM, Appendix I of this overall report).

2.0 Tailings Pile Closure Concepts

Three closure concepts have been considered by the Project Partners as part of the Wolverine Creek side closure concept study, as outlined below.

2.1 Sediment Control Only (Not in the LCCA): Option WC1

This concept involves construction of a sediment control structure downstream of the Wolverine Creek channel. It involves leaving the tailings in the present state with no work being done on the tailings pile and the channel.

2.2 Water Passage and Stability Improvement (LCCA Option B, C, D, D2): Option WC2

This concept does not provide for a remediation measure for the tailings pile. It involves conducting sufficient work at the base of the tailings pile (in the immediate area of the creek per drawings for LCAA Option B, C, D, D2) to minimize the tailings movement. A water conveyance channel would be created at the base of the tailings pile, per the options presented for LCAA, or some other option per hydrotechnical studies. None of the drawings provided in the LCCA show any buttress being placed above the very base of the slope, in the valley bottom.

Wood is proposing the placement of a buttress/berm at the base of the tailings pile and considerably up the slope to stabilize the overall tailings mass and thereby minimize tailings movement under both static and dynamic loading. The source material for this buttress would be waste from the existing waste dumps or tailings pile. Subsequent to the DRAFT version of this memorandum, additional work has been completed on this option, including the design of a buttress dam and required cuts and/or toe berms along the perimeter of the pile. Details of this options (now considered Option WC2) are presented in a standalone memorandum in Appendix H of this overall report.

2.3 Isolate the Asbestos Tailings (LCCA Option E, E2): Option WC3

This concept involves either stabilizing the existing tailings pile by removing material from upslope of the pile and placing it on the downslope and toe, thereby flattening the overall tailings pile slope, and allowing it to be covered as per LCCA Option E. An alternative in this option would be to completely relocate the tailings pile from its present location, which would require no further analysis since the entire tailings mass would been removed.

Subsequent to the completion of the DRAFT of this memorandum, additional engineering work was completed and the LCCA Option E, E2 was considered to be similar to Option WC2, and a single Option WC2 was developed; this option is described in detail in Appendix H of this overall report. Option WC3 then is limited to the option to remove all tailings to alternative locations.

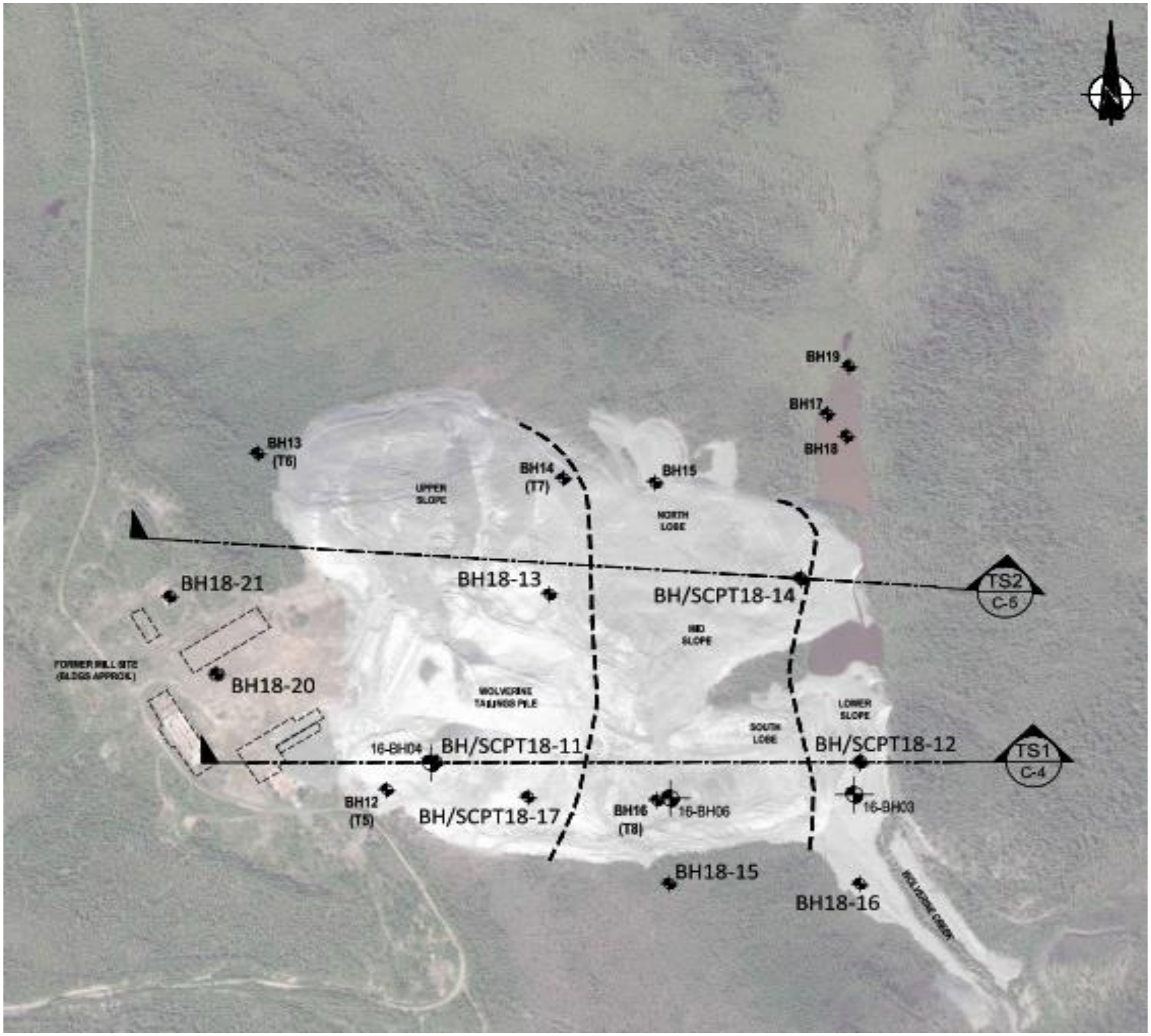
3.0 Methodology and Results

The tailings piles on the existing slopes were analyzed for their stability. Stability analyses were performed along two cross-sections TS1 and TS2 (see Figure 3-1 for locations). The stability analyses were carried out using SlopeW software (Geostudio 2016) incorporating subsurface data obtained from the 2016 and 2018 field programs. An additional cross section, TS3, was considered (see Figure 3-2), which investigates the northeast facing tailings slope, which appears to also exhibit past and present instability. The implications of closure for this portion of the pile has not been considered in this memorandum. However, if the base of the tailings is, or becomes, saturated and liquefies seismically, it would be expected that tailings in this lobe could reach Wolverine Creek.

The subsurface profiles below the tailings, as obtained from borehole data, comprised a colluvium layer on the slopes underlain by a layer of weathered bedrock over competent bedrock. The creek channel contains alluvial soils below the tailings, underlain by weathered and intact bedrock.

Analyses of the current slope configurations was completed to determine the factor of safety (FoS) of the slopes at the current time. Sensitivity analyses were also completed, using different scenarios based on field borehole data. Some of the sensitives that were analyzed included the presence/absence of ice-rich permafrost in the colluvium and the potential for a saturated layer being present at the base of the tailings, which could liquify, either due to static liquefaction with rising water levels in the tailings or under earthquake loading, where a saturated layer is present at the base of the tailings. The modelled rise in the phreatic surface could be attributed to precipitation and/or thaw of underlying permafrost in the colluvium. Based on available water contents of samples near the boundary, the tailings at the base may be up to 85% saturation at the current time and therefore seismic liquefaction along the boundary is considered likely.

Per the DBM, a target FoS of 1.2 for static stability and equal to or greater than 1.0 for a post-liquefied strength were used in the analyses.



wood.

Wood Environment & Infrastructure Solutions

wood.

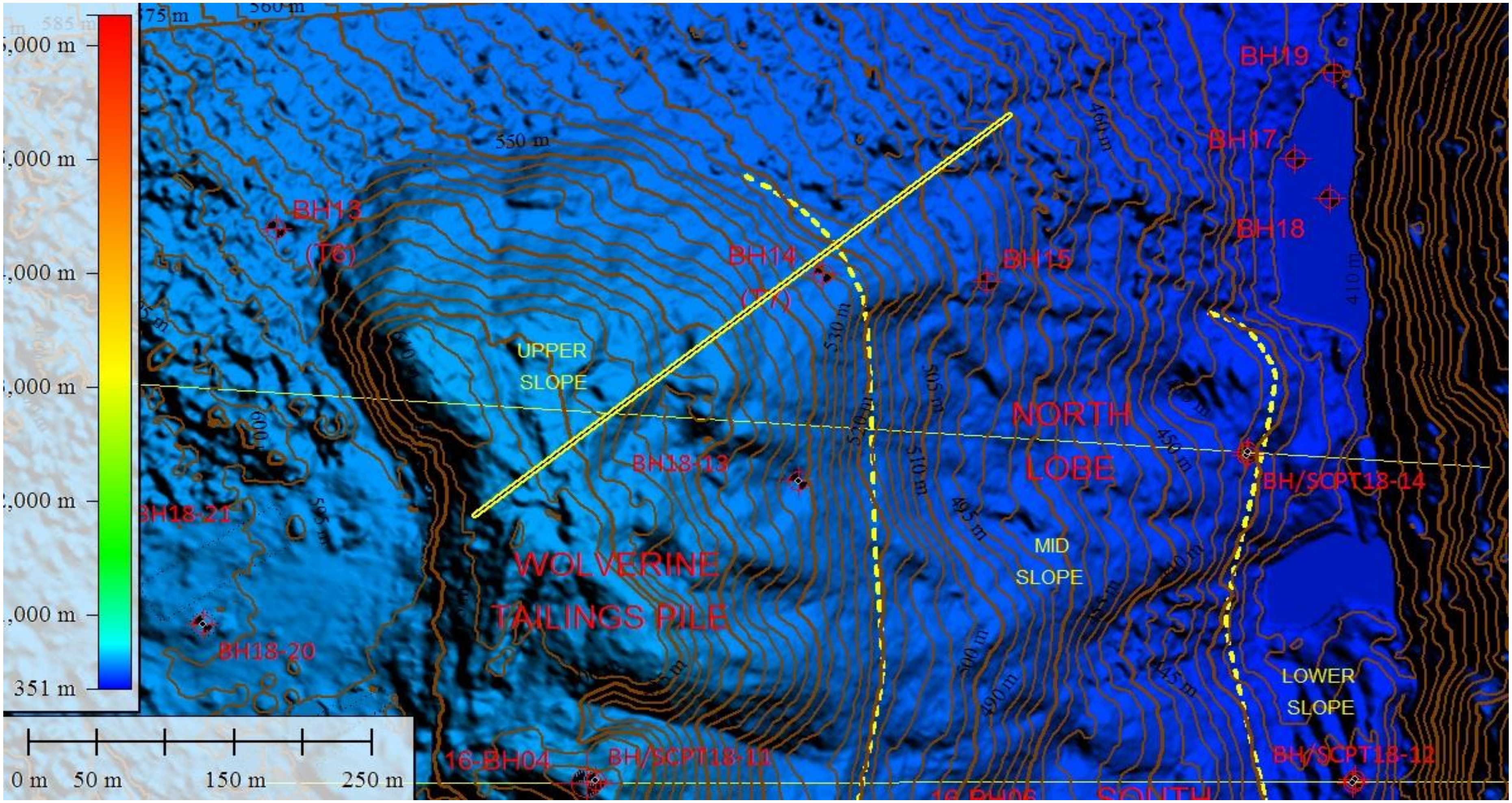
CLIENT:

Government of Yukon

PROJECT: **Clinton Creek Remediation Project**

TITLE: **Wolverine Tailings Dump Borehole and Cross-Section Locations**

DATE:	JOB No.:	FILE:	FIGURE No.:	REV.
August 2019	VE52705D	WC Stability Figures.xlsx	3-1	A



Wood Environment & Infrastructure Solutions	PROJECT:	**Clinton Creek Remediation Project**
CLIENT: **Government of Yukon**	TITLE:	**Northeast Lobe (TS3) Location**

DATE:	JOB No.:	FILE:	FIGURE No.:	REV.
August 2019	VE52705D	WC Stability Figures.xlsx	3-2	A

3.1 Option WC1 – Stability of Current Slopes

Appendix I presents a summary of the stability analysis undertaken for this closure concept. Table 3-1 presents a summary of the results of these stability analyses.

Table 3-1: FoS for Option WC1 (Existing Slopes)

Section	FOS Static (Drained)_	FoS Tailings Liquefied	FoS Colluvium (Undrained)
TS1	1.04	0.48	0.67
TS2	1.14	0.43	0.80
TS3	1.36	0.37	0.89

Results of the analyses indicate that the tailings in their existing profile are marginally stable in the static condition with FoS ranging between 1.04 and 1.36, assuming that the tailings are fully drained. If the tailings liquefy, the FoS drops to 0.37 to 0.48. If an undrained strength ratio of 0.2 is invoked in unfrozen colluvium, the FoS ranges from 0.67 to 0.89. Marginally stable conditions along TS1 is consistent with Slope Indicator data from BH18-17.

3.2 Option WC2

Appendix II presents a summary of the stability analyses undertaken for the preliminary understanding of this closure concept. These analyses are along the design cross-sections TS1 and TS2 only, and do not address side slopes or the required dam down Wolverine Creek from the buttress. Please see Appendix H of this overall report for additional assessment.

Closure Concept WC2 is to stabilize the existing tailings mass by creating a buttress/berm at the base of the tailings to provide passive support against slope failure. The buttress would be constructed of imported waste or tailings from the existing dumps placed and compacted at the tailings base and extending west, up the slopes to the extent required to obtain a FoS of 1.2 for the static condition and FoS of equal to or greater than 1.0 in the case of a liquefied layer at base of the tailings. The buttress elevations required where the design slope intersects the east, or far, side of the valley, and the corresponding FoS are presented in Table 3-2. These berms are of different elevations depending on the strength assumptions. It is considered unlikely that the undrained colluvium strength in a thaw condition is as low as 0.2, as thawed colluvium likely has some degree of overconsolidation.

As is to be expected, the quantity of buttress waste required for stabilizing the tailings pile in the case of liquefaction exceeds that for the static case. Relative quantities of waste rock required for each section are presented in Appendix II.

Table 3-2: FoS and Required Buttress for Option WC2

Section	FOS Drained	BERM on East Slope	FoS Liquefied	BERM on East Slope	FoS colluvium undrained	BERM on East Slope
TS1	1.2	428 masl	1.00	435 masl	1.00	440 masl
TS2	1.2	415 masl	1.00	430 masl	1.00	430 masl

3.3 Option WC3

Appendix III presents a summary of the stability analyses undertaken for this closure concept following Option LCCA E.

Closure Concept LCCA E involves re-profiling the existing tailings slope by removing the upper portion of the tailings pile and placing it in the lower part, in order to create an overall tailings slope of 3.75H to 1V. Placement of the tailings at the lower end would extend across the valley floor to buttress against the native slopes on the east side of the valley.

Analyses were carried out on the above slope profile and found to yield acceptable factors of safety (1.5) for the static case (see Table 3-3). Potentially liquefiable cases of failure through a weak, saturated layer at the base of the tailings were then considered. Factors of safety obtained in this latter liquefaction scenario were significantly less than 1.0, indicating that the tailings slope would be unstable during liquefaction, for the 3.75H: 1V scenario.

Table 3-3: WC3 LCCA E 3.75H:1V Slope

Section	FOS Drained	BERM on East Slope	FoS Liquefied	BERM on East Slope	FoS colluvium undrained	BERM on East Slope
TS1	1.49	422 masl	0.71	422 masl	0.89	422 masl
TS2	1.37	422 masl	0.57	422 masl	-	-
TS3	1.69	-	0.83	-	1.20	-

4.0 Slope Displacements

Additional analyses were carried out on the tailings pile slopes to obtain earthquake-induced displacements using the Bray and Travasarou (2007) method per the DBM. This procedure required the usage of a yield horizontal seismic coefficient (Kyield) which was obtained from a pseudostatic analysis run of the slope in the analyses carried and summarized in Appendix IV. The seismic parameters are as previously discussed for Site Class C by NRCan; however, they were modified based on characterization of the tailings foundations as Site Class D. Predicted displacements were obtained for the static cases for WC2 and LCCA E.

Tables 4-1 and 4-2 summarize the results of these analyses:

Table 4-1: Expected Displacements for Option WC2

Expected Displacement	84% Probability of Exceedance Value (cm)	Mean Value (cm)	84% Probability of Exceedance Value (cm)
Not considering earthquake magnitude dependence	5.2	10.1	19.7
Considering magnitude earthquake dependence	5.2	10.1	19.5

Table 4-2: Expected Displacements for Option LCCA E

Expected Displacement	84% Probability of Exceedance Value (cm)	Mean Value (cm)	84% Probability of Exceedance Value (cm)
Not considering earthquake magnitude dependence	5.6	10.9	21.3
Considering magnitude earthquake dependence	5.6	10.9	21.1

Sincerely,

Wood Environment & Infrastructure Solutions
a Division of Wood Canada Limited

Prepared by:

S. S. GAREWAL
ENGINEER

Garewal
August 28, 2019

Surinder Garewal, MEng, PEng (BC)
Associate Geotechnical Engineer

REGISTERED PROFESSIONAL GEOSCIENTIST
K.D. HINCKS
LICENSEE
NT/NU

K Hincks
08 August 2019.

Karen Hincks, MSc, PGeo
Senior Engineering Geologist

Reviewed by:

Ed McRoberts, PhD, PEng
Principal Geotechnical Engineer

Appendix I
Wolverine Creek Stability Assessment: WC-1

Wolverine Creek Stability Assessment: WC-1

Option WC-1

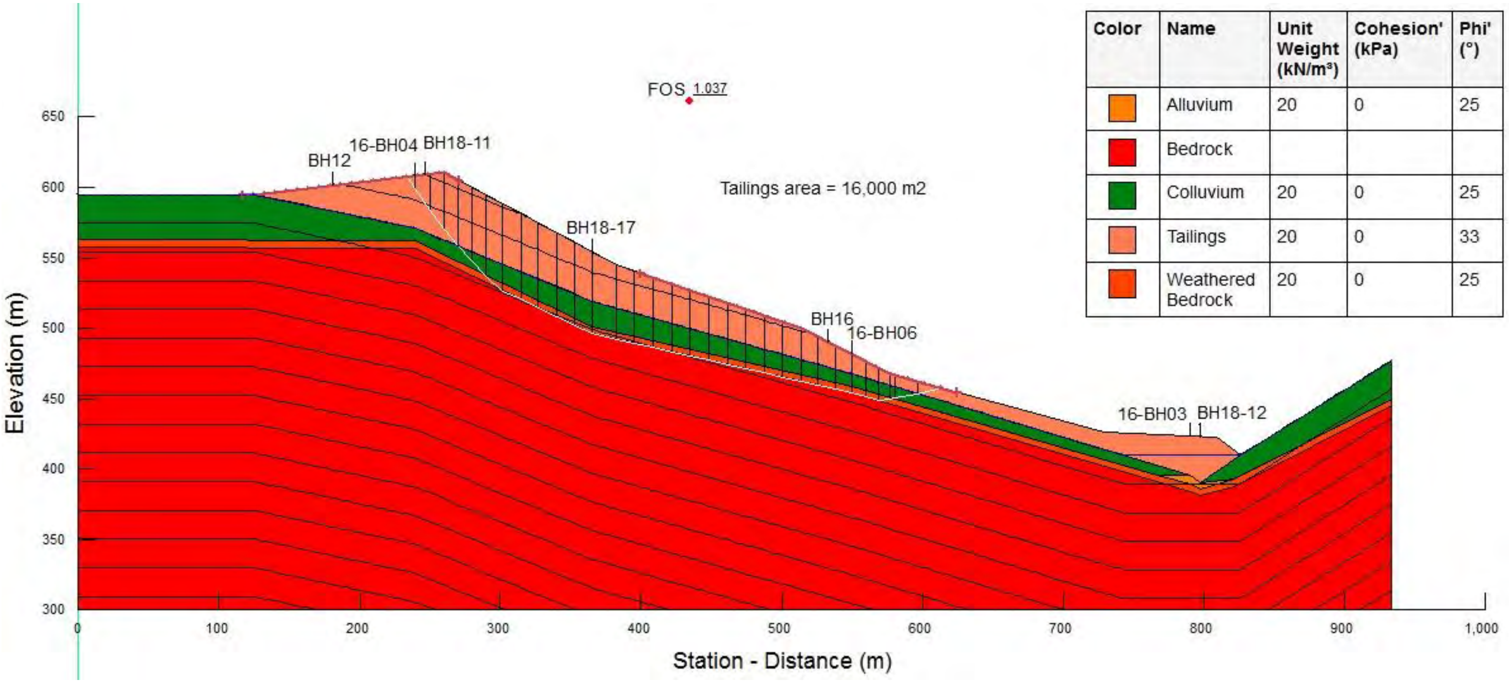
Leave the tailings pile in its current state

Assess current stability

South Lobe: TS1 – Base Case (Existing Slope)

- Created model from cross-section TS-1
- Colluvium, alluvium, weathered bedrock use ϕ=25°, best estimate
- Tailings use ϕ=33°
- Bedrock is impenetrable
- Phreatic surface follows the tailings/colluvium boundary
- Does not include any high ice content frozen material

FoS = 1.04

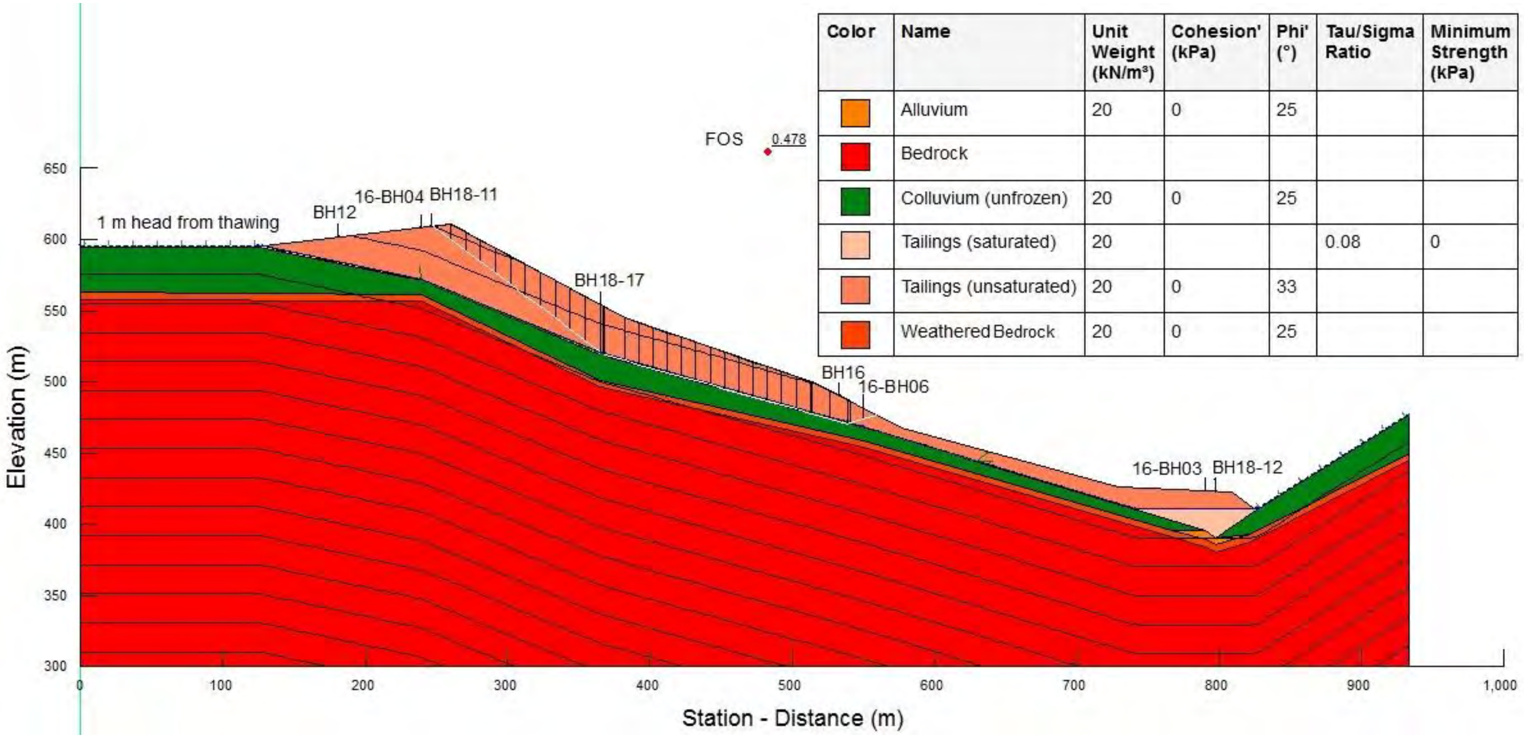


Color	Name	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)
	Alluvium	20	0	25
	Bedrock			
	Colluvium	20	0	25
	Tailings	20	0	33
	Weathered Bedrock	20	0	25

TS1 – Liquefaction in Tailings

- Liquefied tailings use c/p’ = 0.08
- Tailings (unsaturated) use ϕ=33°
- Phreatic surface 1 m above tailings/colluvium boundary

FoS = 0.48

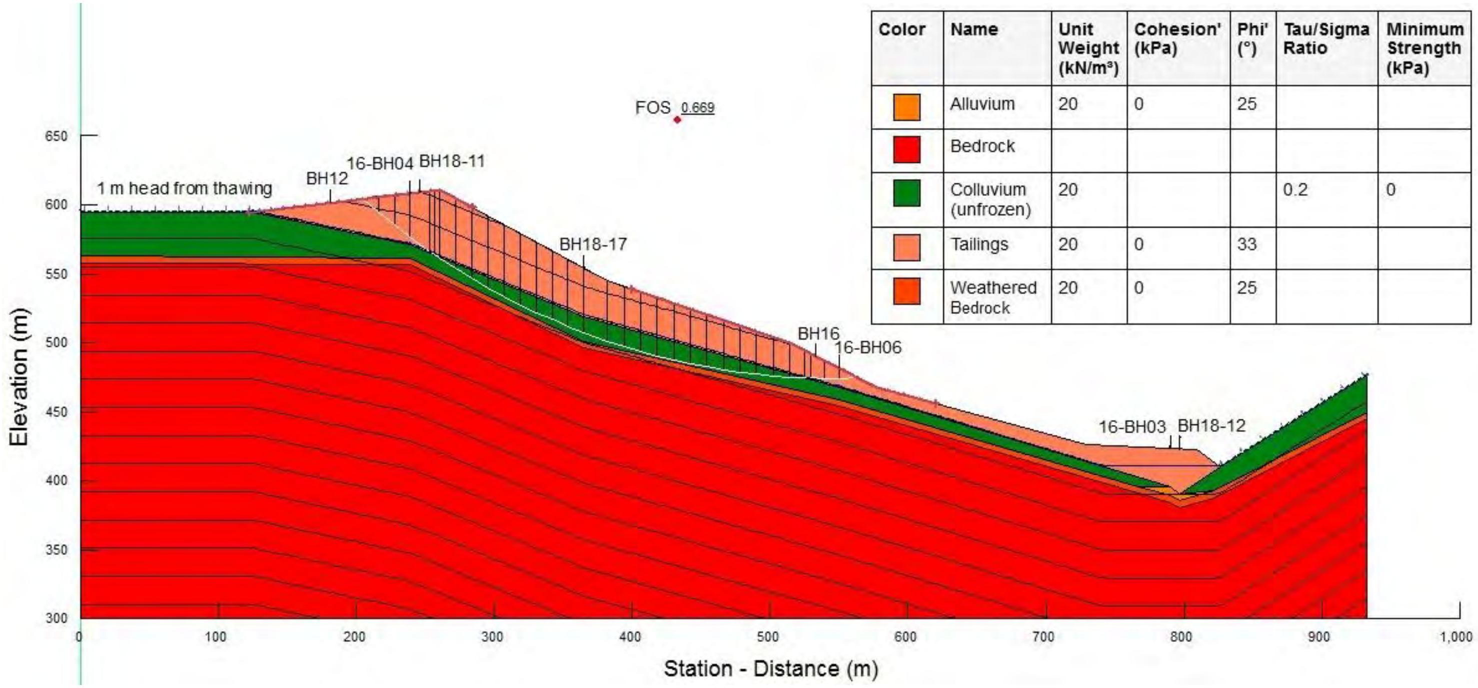


Color	Name	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Tau/Sigma Ratio	Minimum Strength (kPa)
	Alluvium	20	0	25		
	Bedrock					
	Colluvium (unfrozen)	20	0	25		
	Tailings (saturated)	20			0.08	0
	Tailings (unsaturated)	20	0	33		
	Weathered Bedrock	20	0	25		

TS1 – Undrained strength ratio in Colluvium

- Colluvium undrained strength ratio c/p’ = 0.2
- Tailings use $\phi = 33^{o}$
- Phreatic surface 1 m above tailings/colluvium boundary

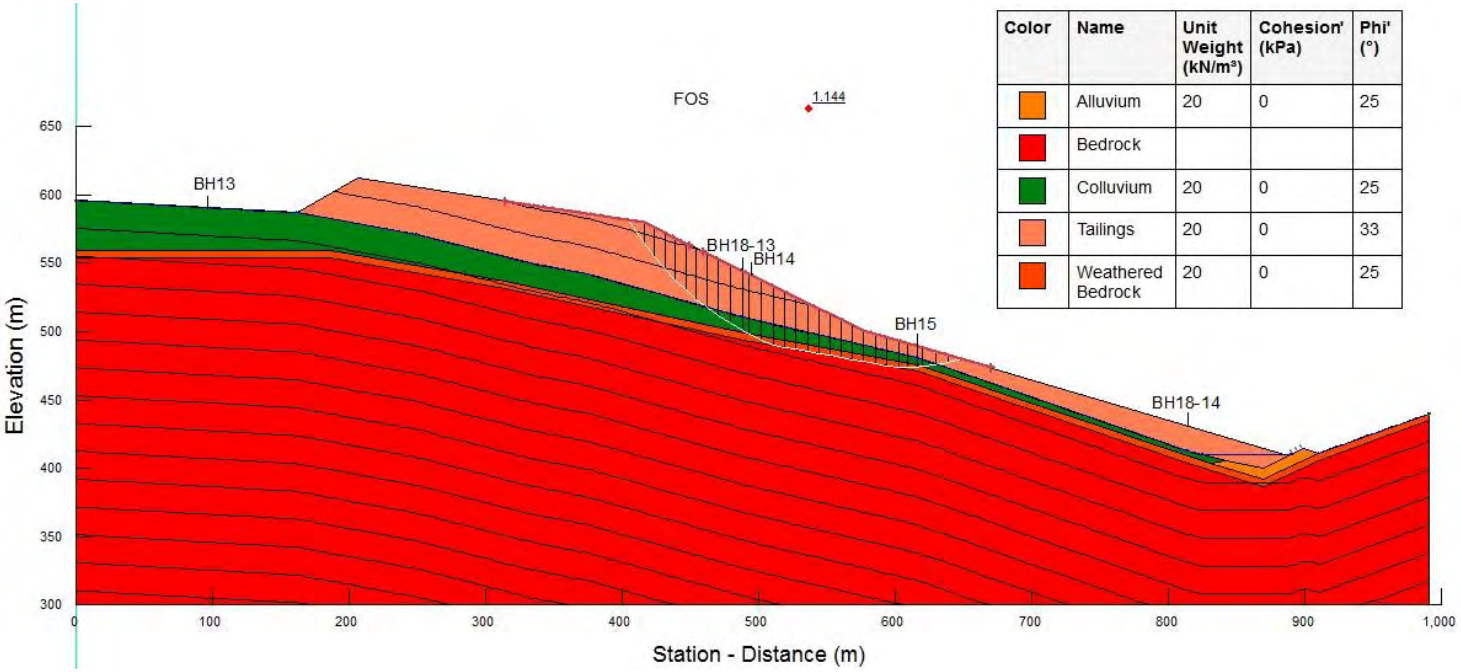
FoS = 0.67



North Lobe: TS2 – Base Case (Existing Slope)

- Model from cross-section TS-2
- Colluvium, alluvium, weathered bedrock use $\phi=25^{o}$, best estimate
- Tailings (unsaturated) use $\phi=33^{o}$
- Bedrock is impenetrable
- Phreatic surface follows the tailings/colluvium boundary
- Does not include any high ice content frozen material

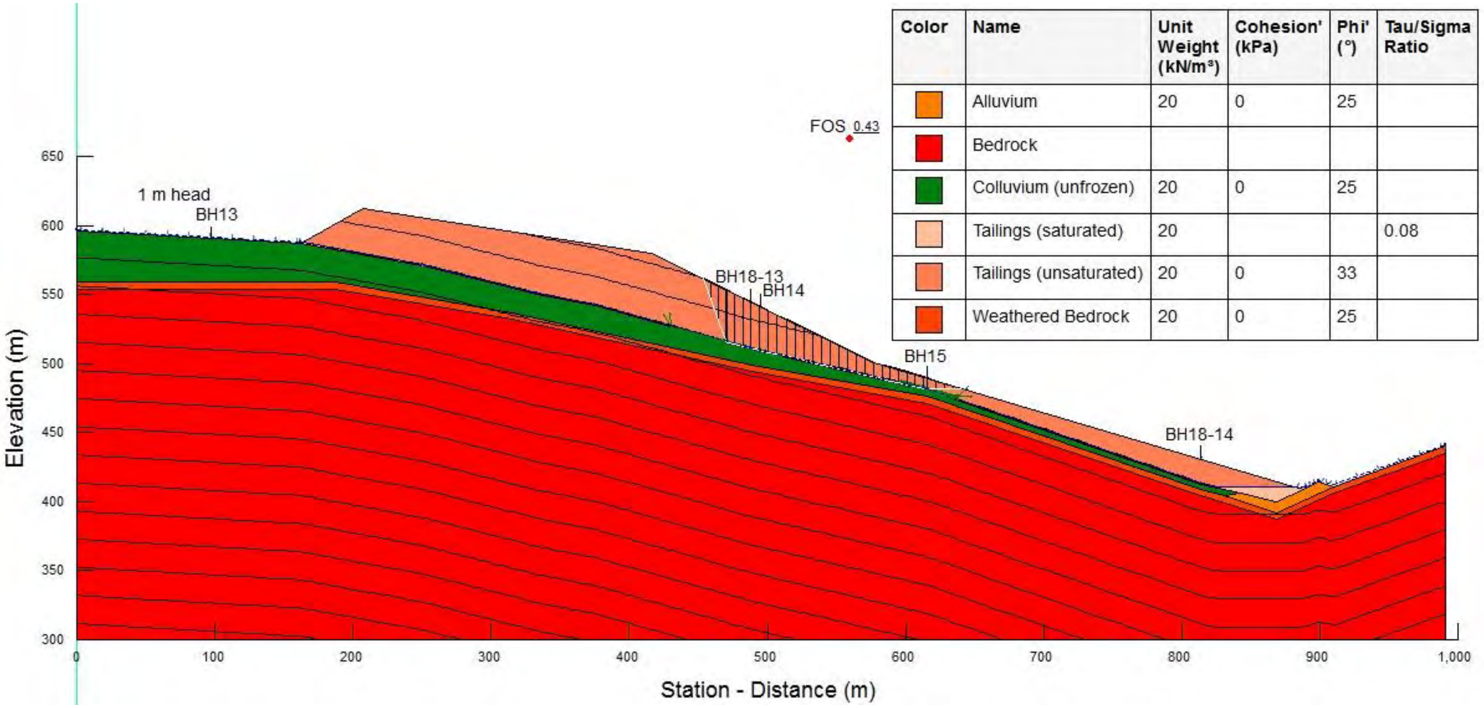
FoS = 1.14



TS2 – Liquefaction in Tailings

- Liquefied tailings use c/p’ = 0.08
- Tailings (unsaturated) use ϕ=33º
- Phreatic surface 1 m above tailings/colluvium boundary

FoS = 0.43

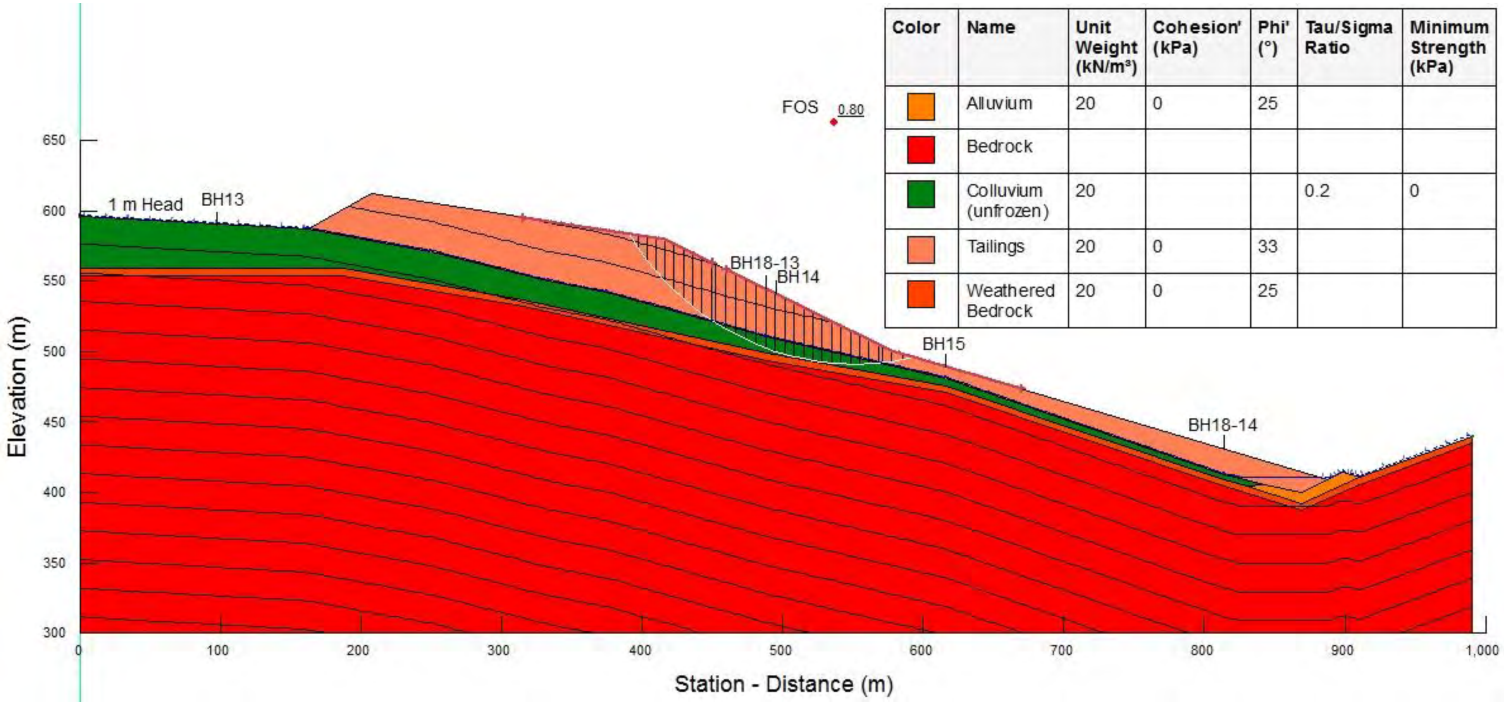


Color	Name	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Tau/Sigma Ratio
	Alluvium	20	0	25	
	Bedrock				
	Colluvium (unfrozen)	20	0	25	
	Tailings (saturated)	20			0.08
	Tailings (unsaturated)	20	0	33	
	Weathered Bedrock	20	0	25	

TS2 – Undrained strength ratio in Colluvium

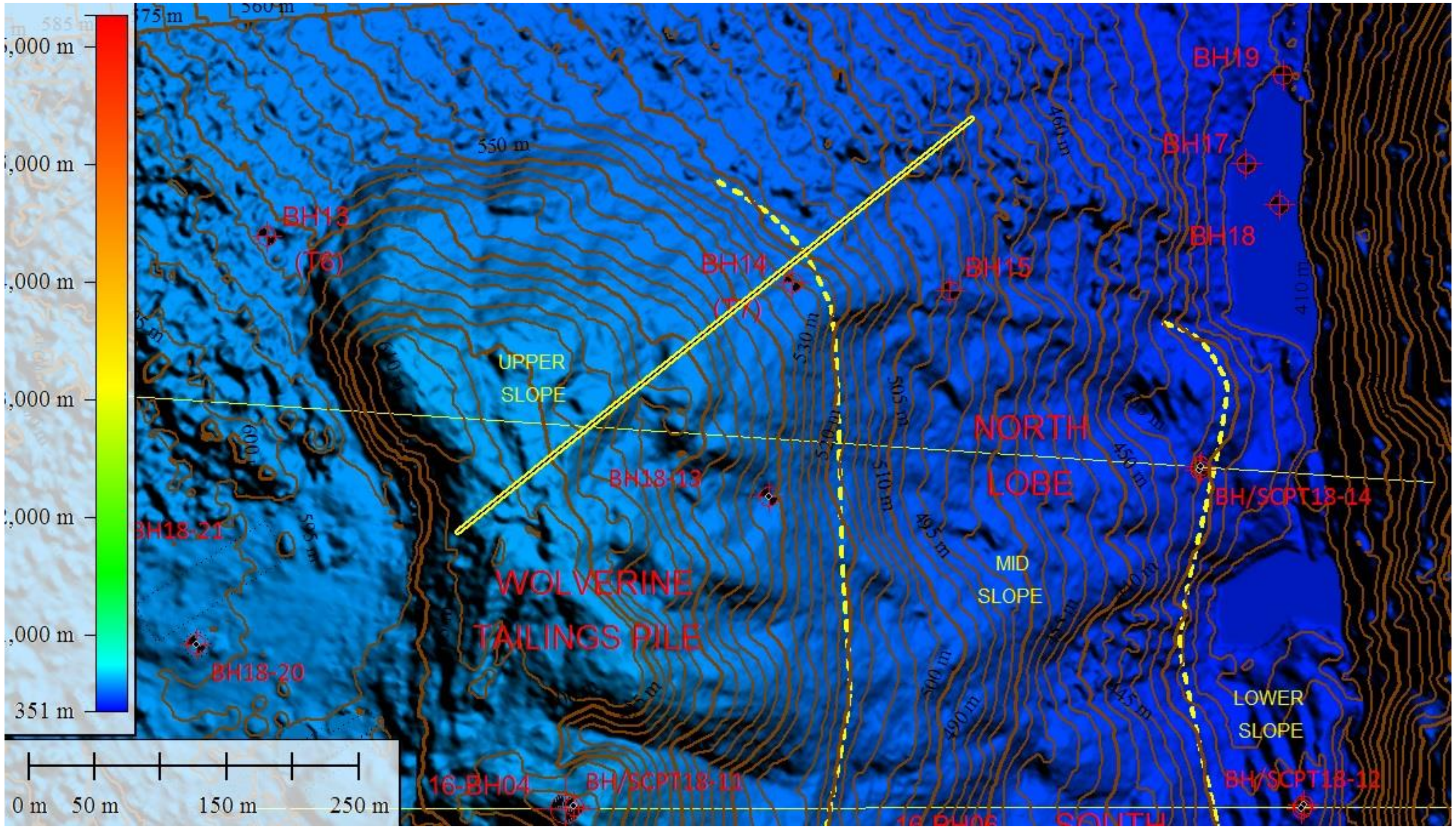
- Colluvium undrained strength ratio c/p’ = 0.2
- Tailings use $\phi=33^{o}$
- Phreatic surface 1 m above tailings/colluvium boundary

FoS = 0.80



Color	Name	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Tau/Sigma Ratio	Minimum Strength (kPa)
	Alluvium	20	0	25		
	Bedrock					
	Colluvium (unfrozen)	20			0.2	0
	Tailings	20	0	33		
	Weathered Bedrock	20	0	25		

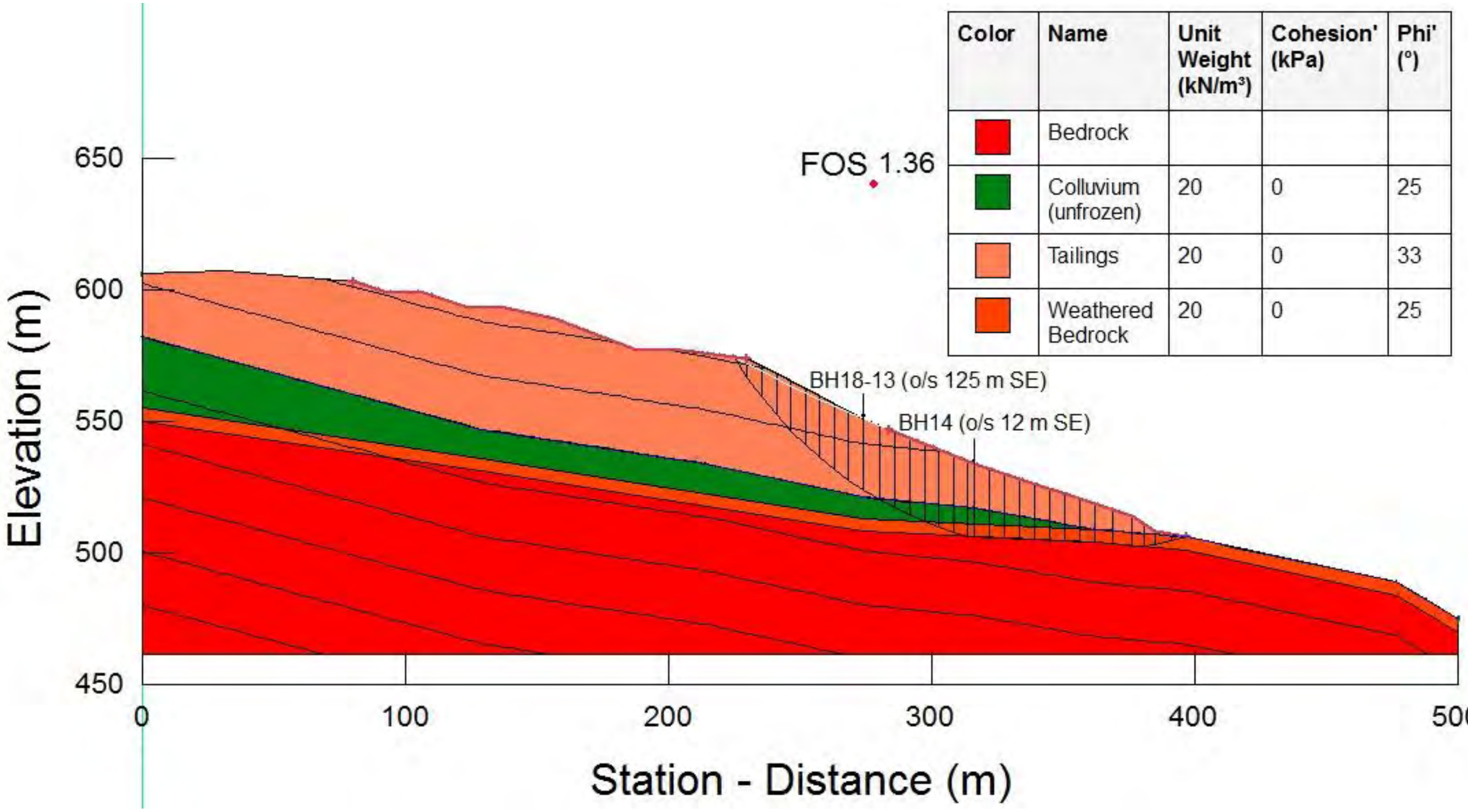
TS3 – Running SW – NE on Upper Slope



TS3 – Base Case (Existing Slope)

- Colluvium, weathered bedrock use $\phi=25^{o}$, best estimate
- Tailings (unsaturated) use $\phi=33^{o}$
- Bedrock is impenetrable
- Phreatic surface follows the tailings/colluvium boundary
- Does not include any high ice content frozen material

FoS = 1.36



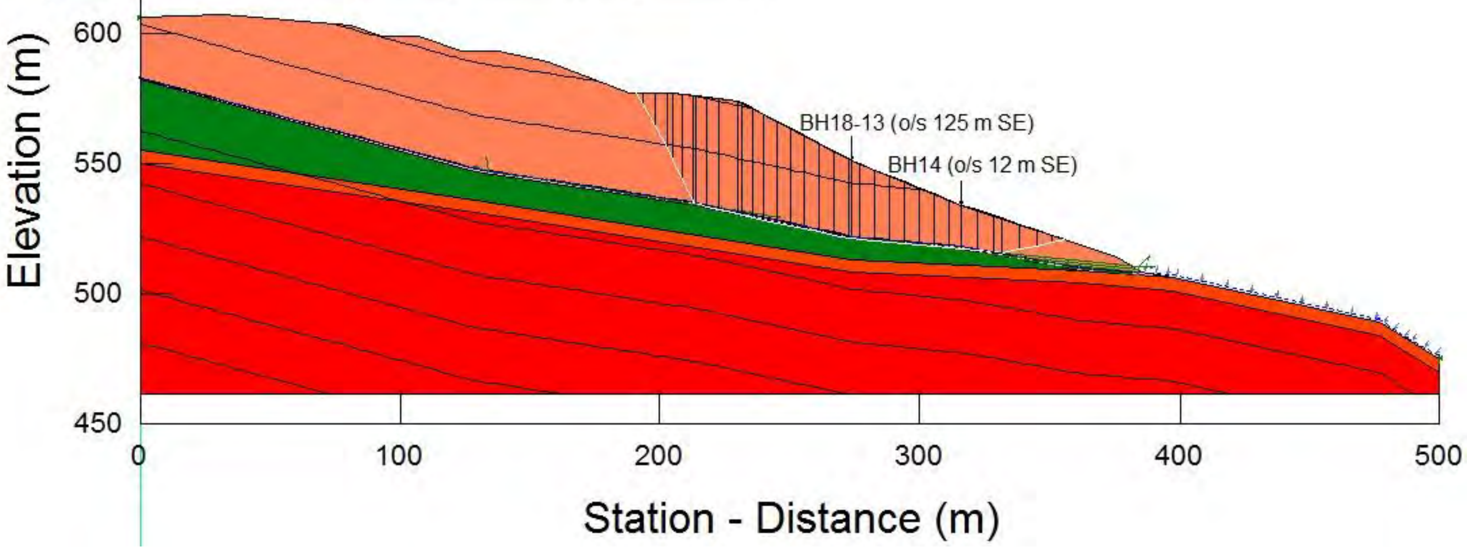
TS3 – Liquefaction in Tailings

FoS = 0.65

- Liquefied tailings use $c/p' = 0.08$
- Tailings (unsaturated) use $\phi=33^{o}$
- Phreatic surface 1 m above tailings/colluvium boundary

Color	Name	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Tau/Sigma Ratio
	Bedrock				
	Colluvium	20	0	25	
	Tailings (saturated)	20			0.08
	Tailings (unsaturated)	20	0	33	
	Weathered Bedrock	20	0	25	

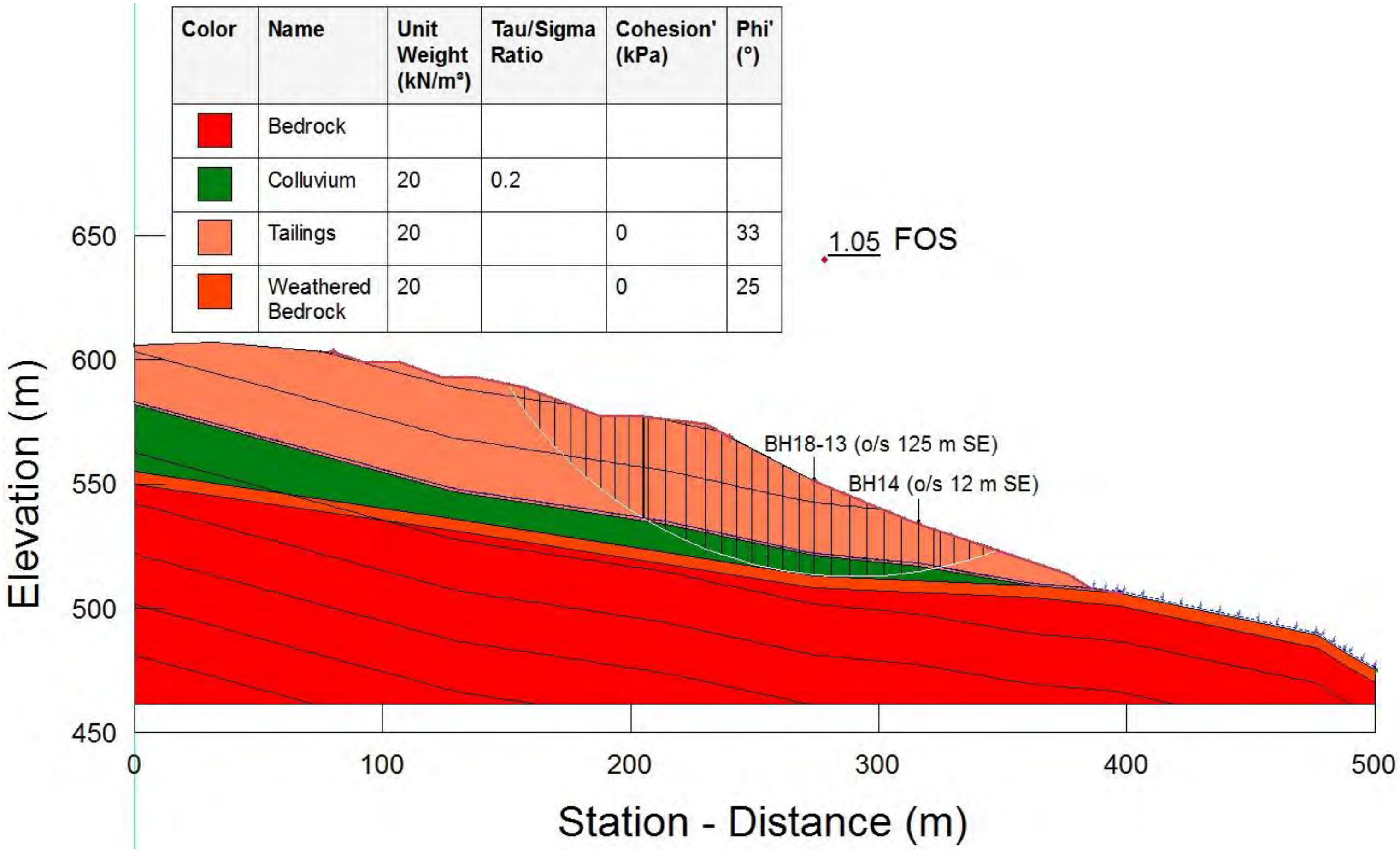
FOS 0.65



TS3 – Undrained strength ratio in Colluvium

- Colluvium undrained strength ratio c/p' = 0.2
- Tailings use ϕ=33°
- Phreatic surface 1 m above tailings/colluvium boundary

FoS = 1.05



Appendix II
Wolverine Creek Stability Assessment: WC-2

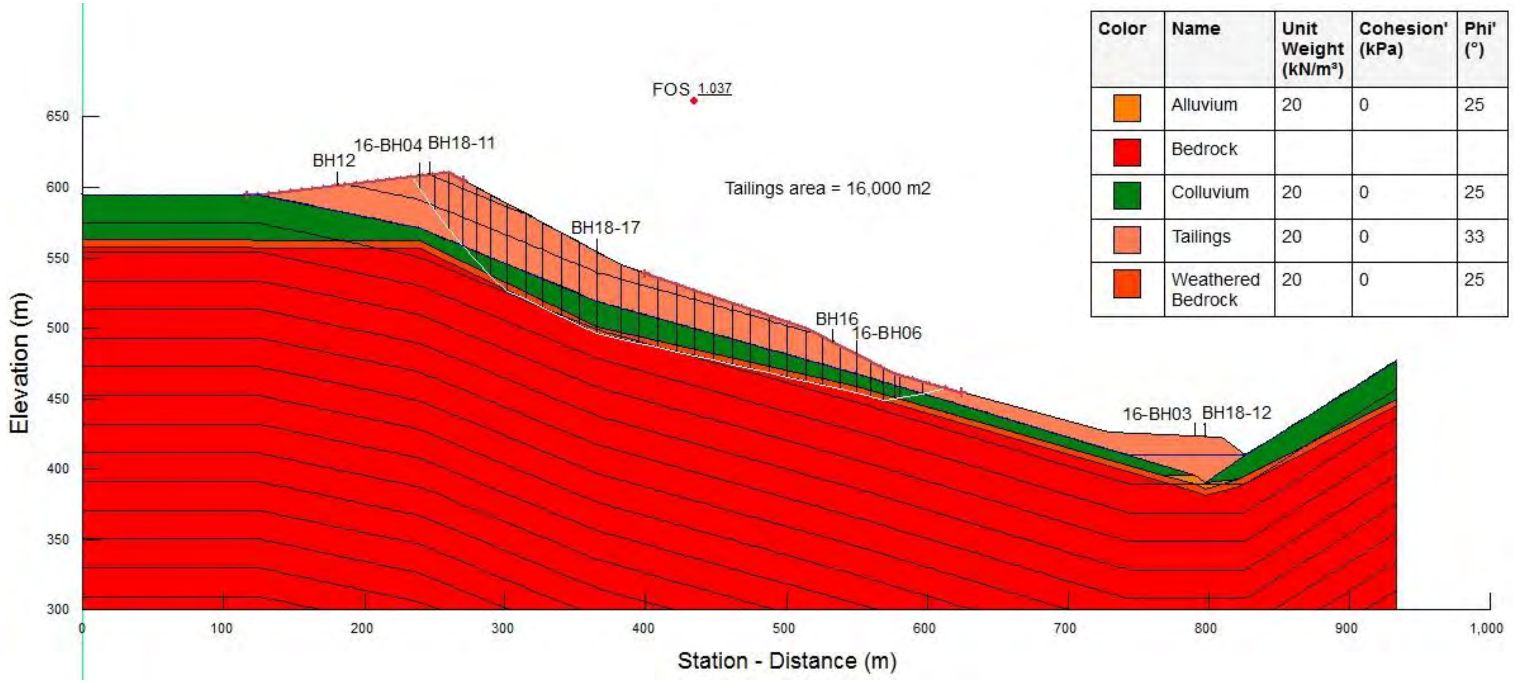
Wolverine Creek Stability Assessment: Option WC-2

Option WC-2 is required to stabilize movements on slopes facing Wolverine Creek, without disrupting the tailings surface.
Movements considered to mean both Static and Post Seismic Liquefaction

South Lobe: TS1 – Base State (Existing Slope)

- Model from cross-section TS-1
- Colluvium, alluvium, weathered bedrock use ϕ=25°, best estimate
- Tailings (unsaturated) use ϕ=33°
- Bedrock is impenetrable
- Phreatic surface follows the tailings/colluvium boundary
- Does not include any high ice content frozen material

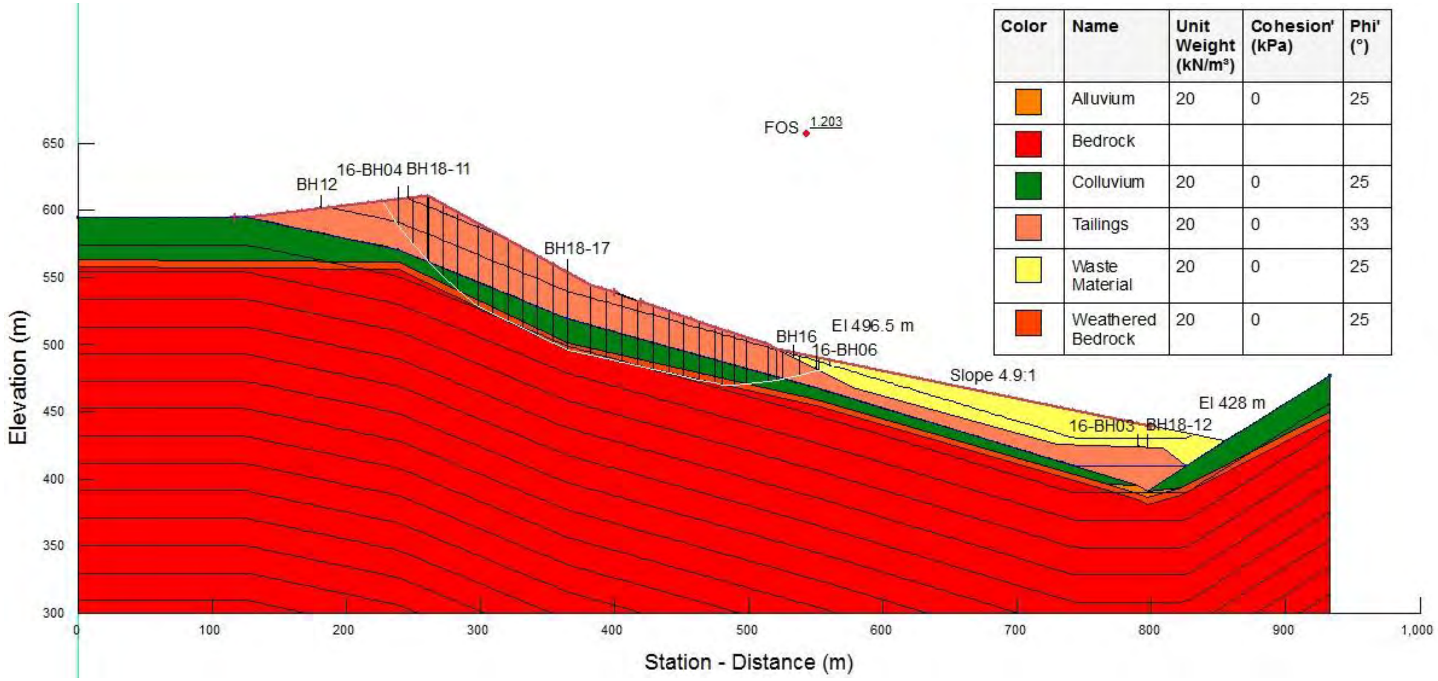
FoS = 1.04



TS1 – Option WC-2: Static Analysis, berm added

- Added Waste Material over tailings slope base until FoS = 1.20
- Phreatic surface follows the tailings/colluvium boundary
- WM area: 6,240 m^2
- ~330 m of WM placed from mid-slope to valley

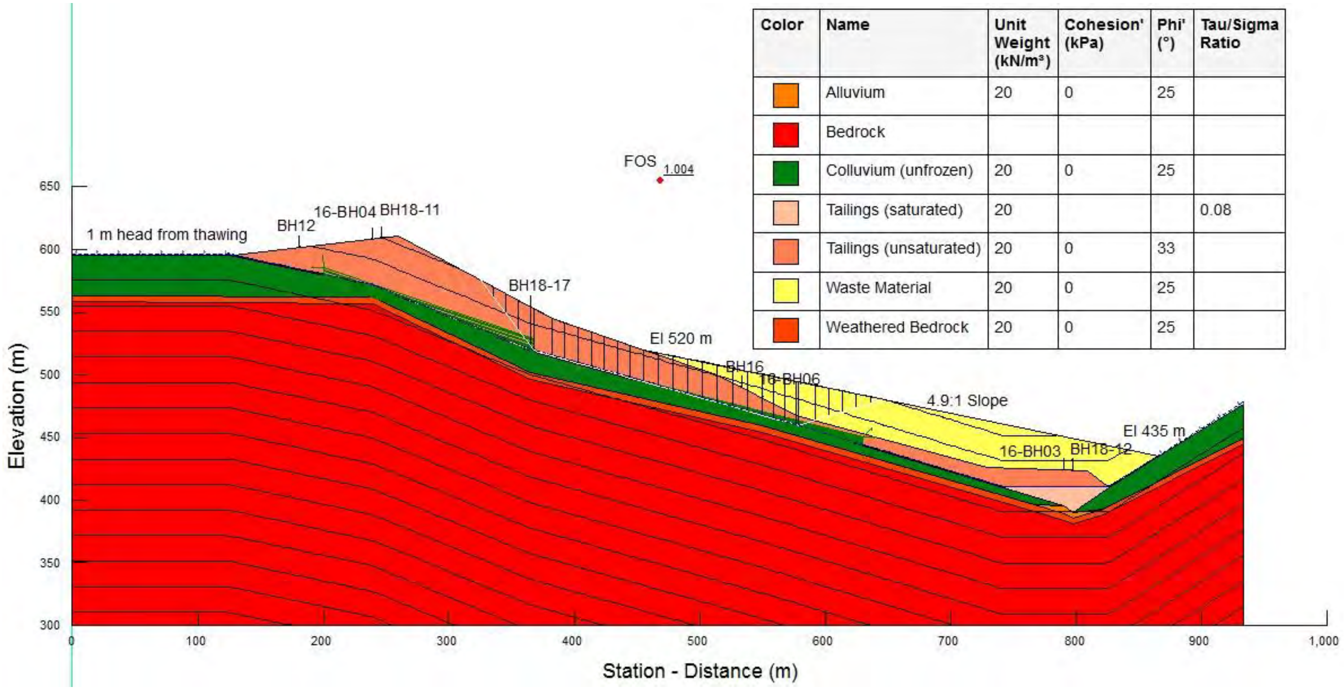
FoS = 1.20



TS1 – Option WC-2: Tailings Liquefied, berm added

- Tailings c/p’ = 0.08
- Added Waste Material over tailings toe until FoS => 1.00
- Phreatic surface 1 m above tailings/colluvium boundary
- WM area: 9,800 m^2
- ~400 m of WM placed from mid-slope to valley

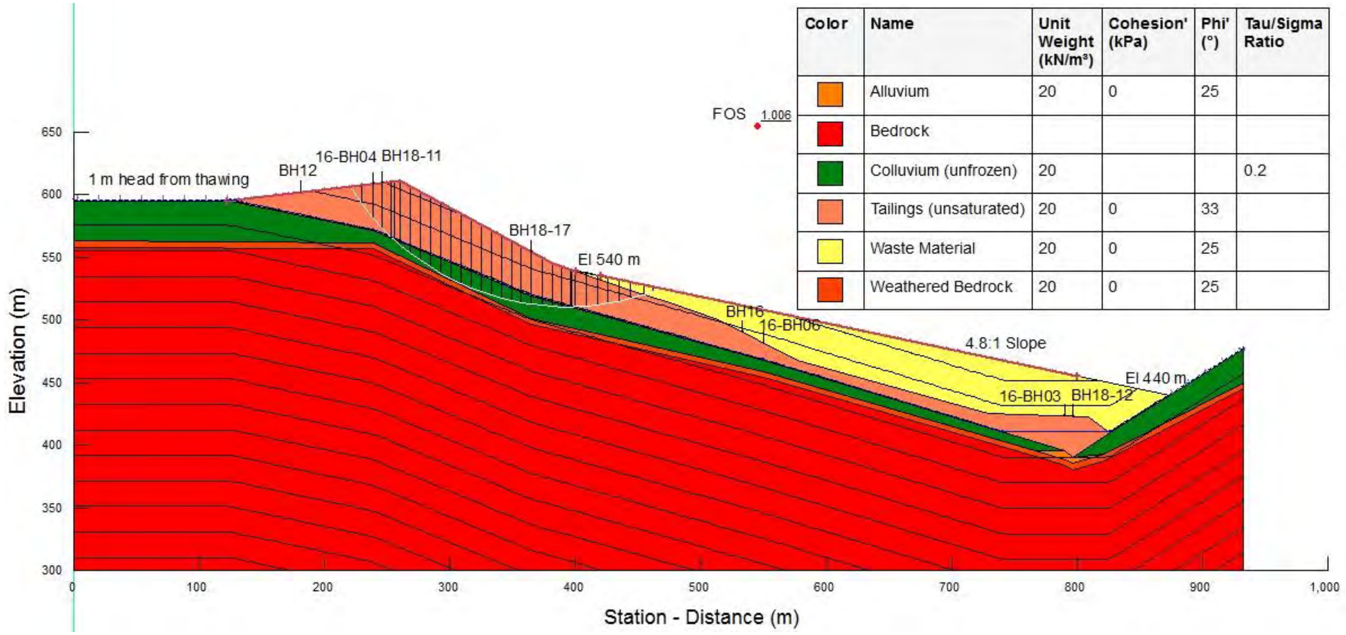
FoS = 1.00



TS1 – Option WC-2: Undrained Colluvium, berm added

- Colluvium c/p' = 0.20
- Added Waste Material over toe until FoS >= 1.00
- Phreatic surface 1 m above tailings/colluvium boundary
- WM area: 13,000 m^2
- ~480 m of WM placed from mid-slope to valley

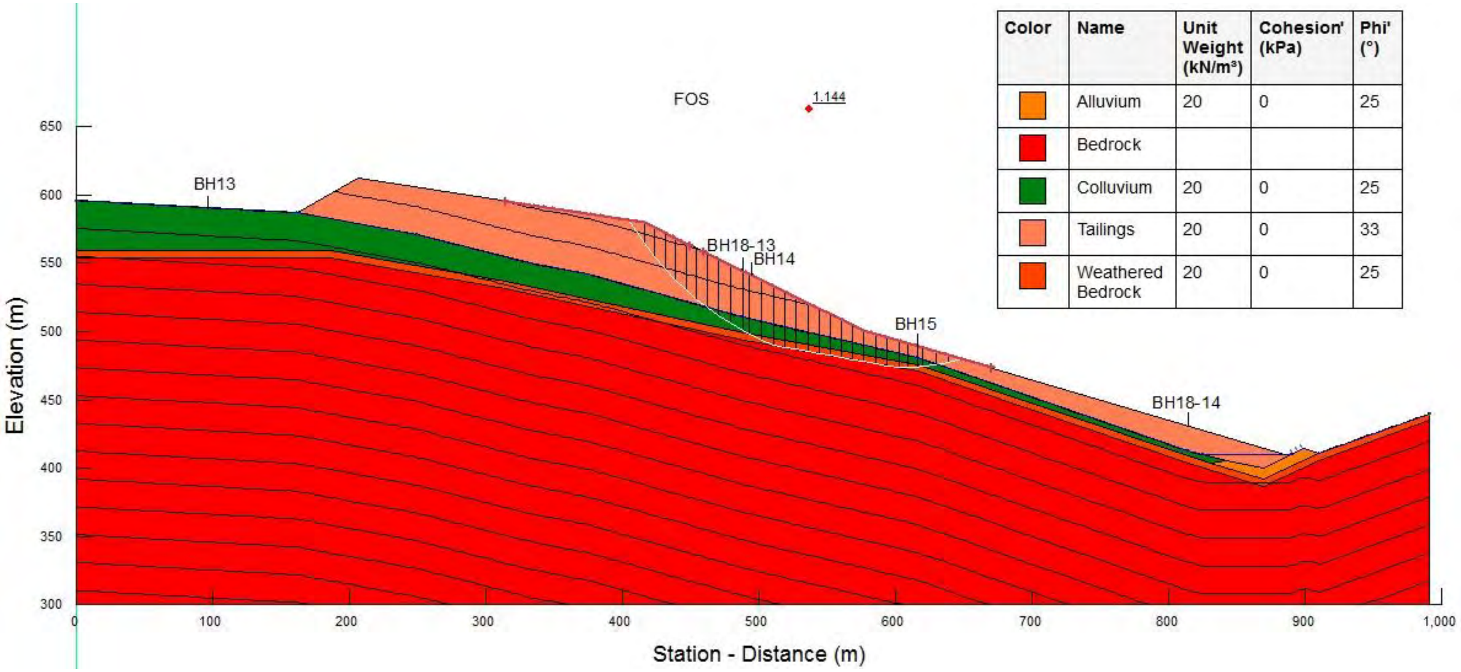
FoS = 1.01



North Lobe: TS2 – Base State (Existing Slope)

- Model from cross-section TS-2
- Colluvium, alluvium, weathered bedrock use ϕ=25º, best estimate
- Tailings (unsaturated) use ϕ=33º
- Bedrock is impenetrable
- Phreatic surface follows the tailings/colluvium boundary
- Does not include any high ice content frozen material

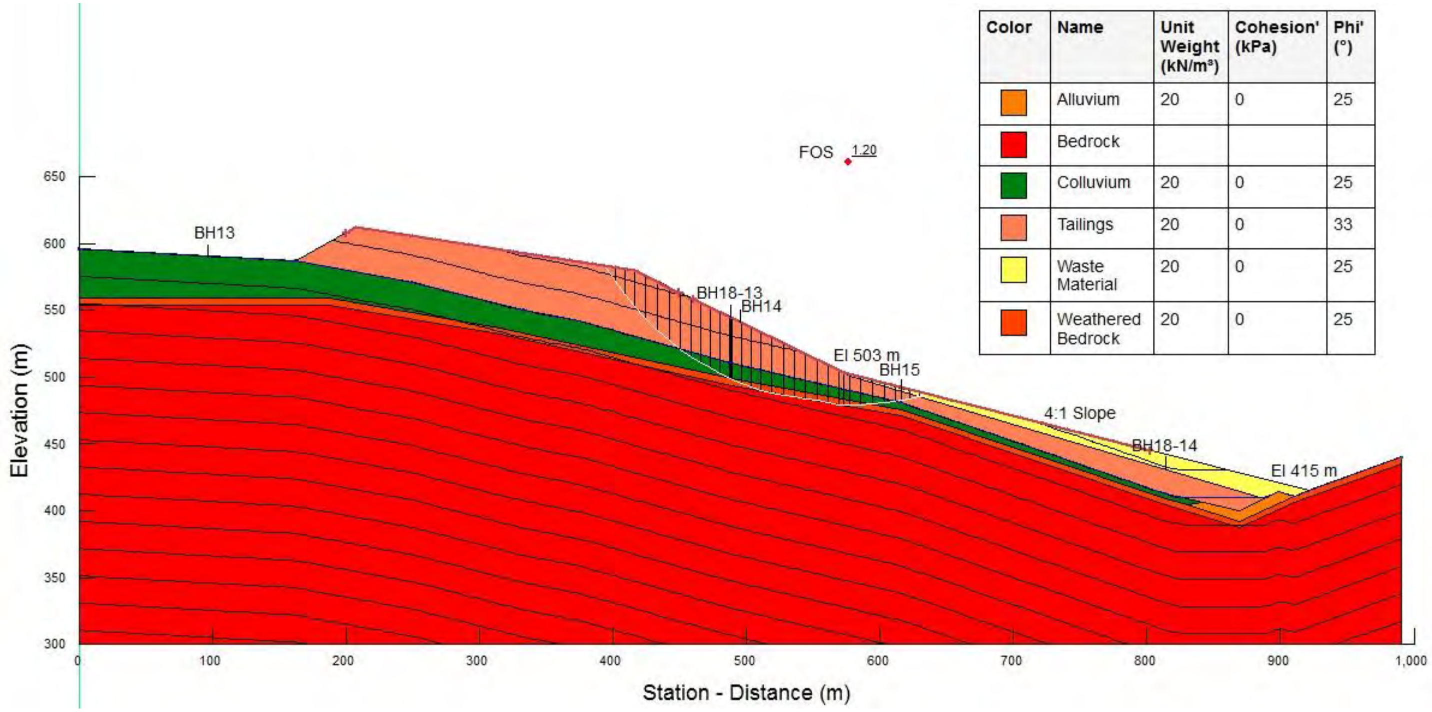
FoS = 1.14



TS2 – Option WC-2: Static Analysis, berm added

- Added Waste Material over base until FoS = 1.20
- Phreatic surface follows the tailings/colluvium boundary
- WM area: 2,630 m^2
- ~350 m of WM placed from mid-slope to valley

FoS = 1.20

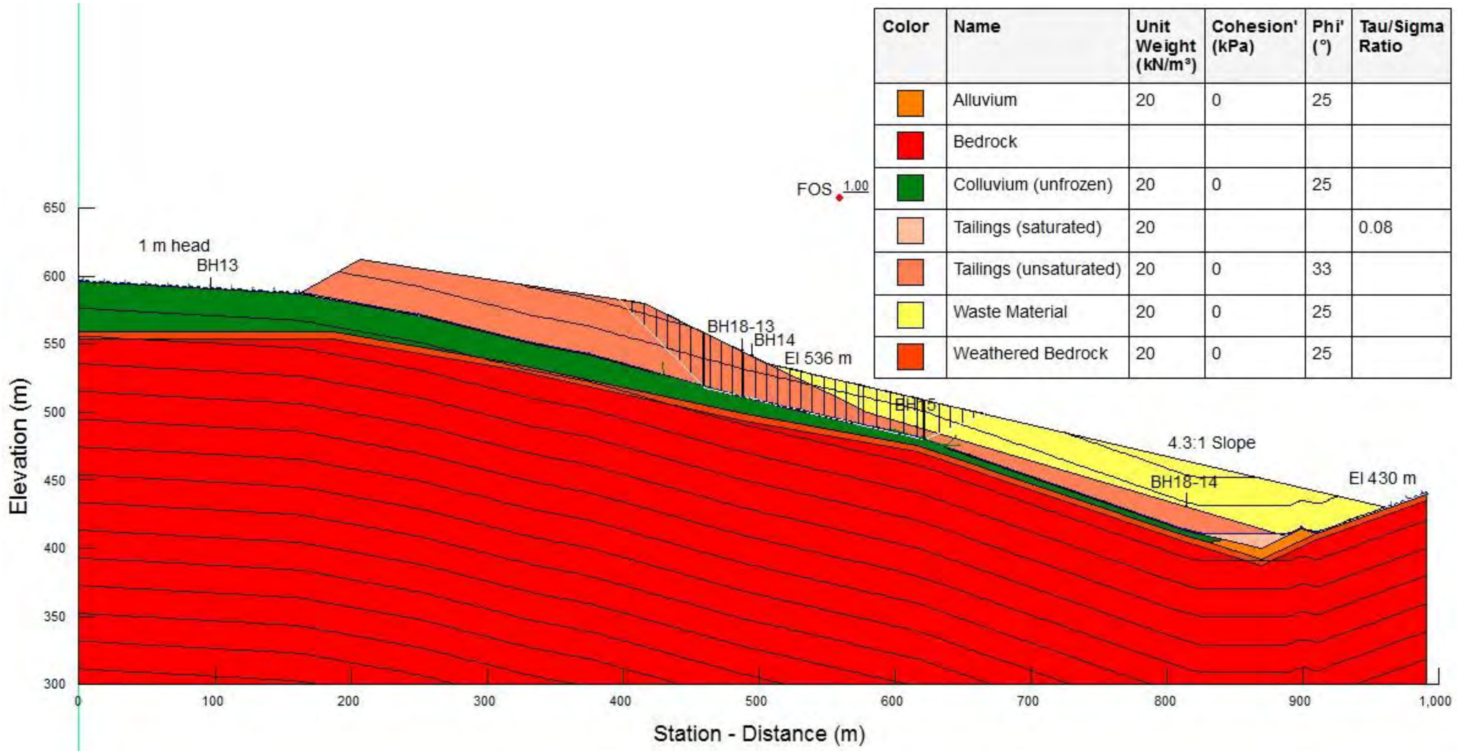


Color	Name	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)
	Alluvium	20	0	25
	Bedrock			
	Colluvium	20	0	25
	Tailings	20	0	33
	Waste Material	20	0	25
	Weathered Bedrock	20	0	25

TS2 – Option WC-2: Tailings Liquefied, berm added

- Tailings (saturated) c/p’ = 0.08
- Added Waste Material over toe until FoS = 1.00
- Phreatic surface 1 m above tailings/colluvium boundary
- WM area: 11,000 m^2
- ~470 m of WM placed from mid-slope to valley

FoS = 1.00

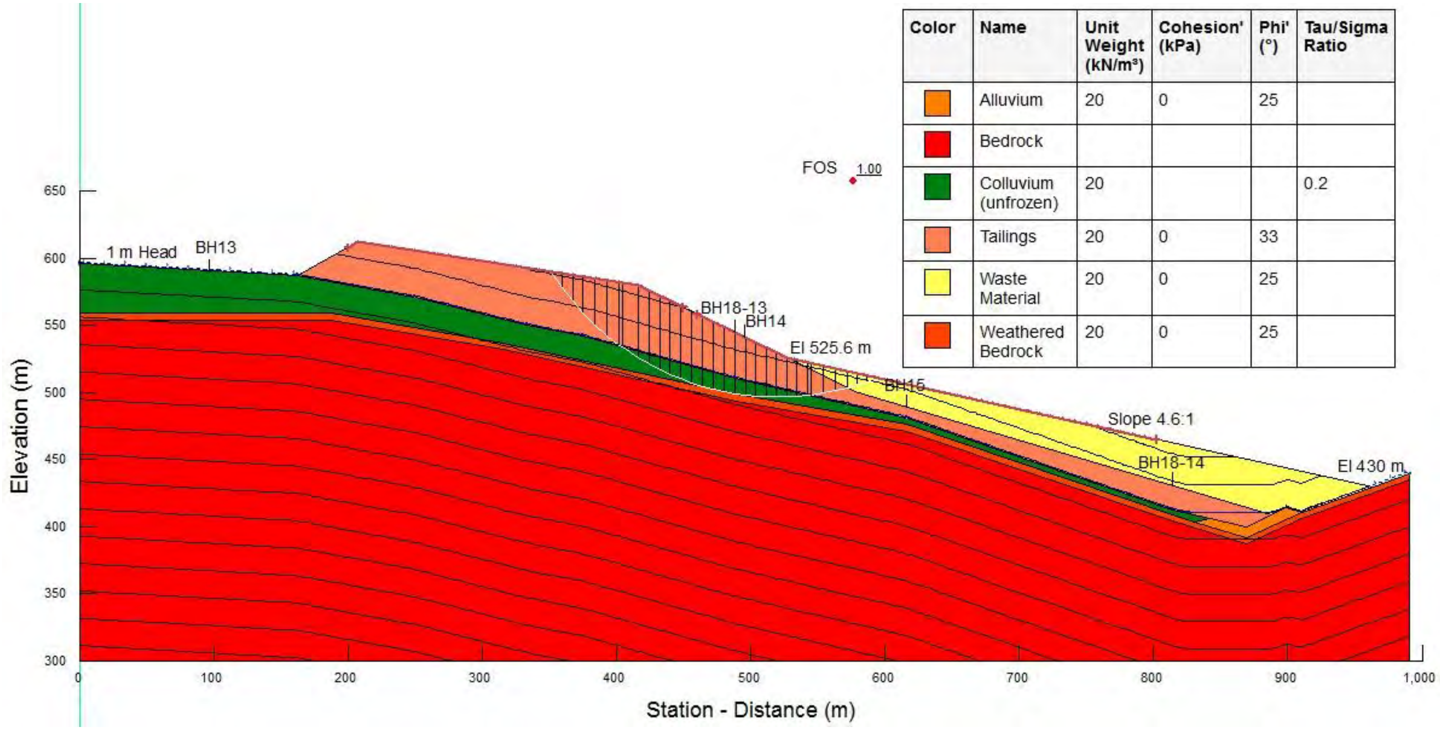


Color	Name	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Tau/Sigma Ratio
	Alluvium	20	0	25	
	Bedrock				
	Colluvium (unfrozen)	20	0	25	
	Tailings (saturated)	20			0.08
	Tailings (unsaturated)	20	0	33	
	Waste Material	20	0	25	
	Weathered Bedrock	20	0	25	

TS2 – Option WC-2: Colluvium undrained, berm added

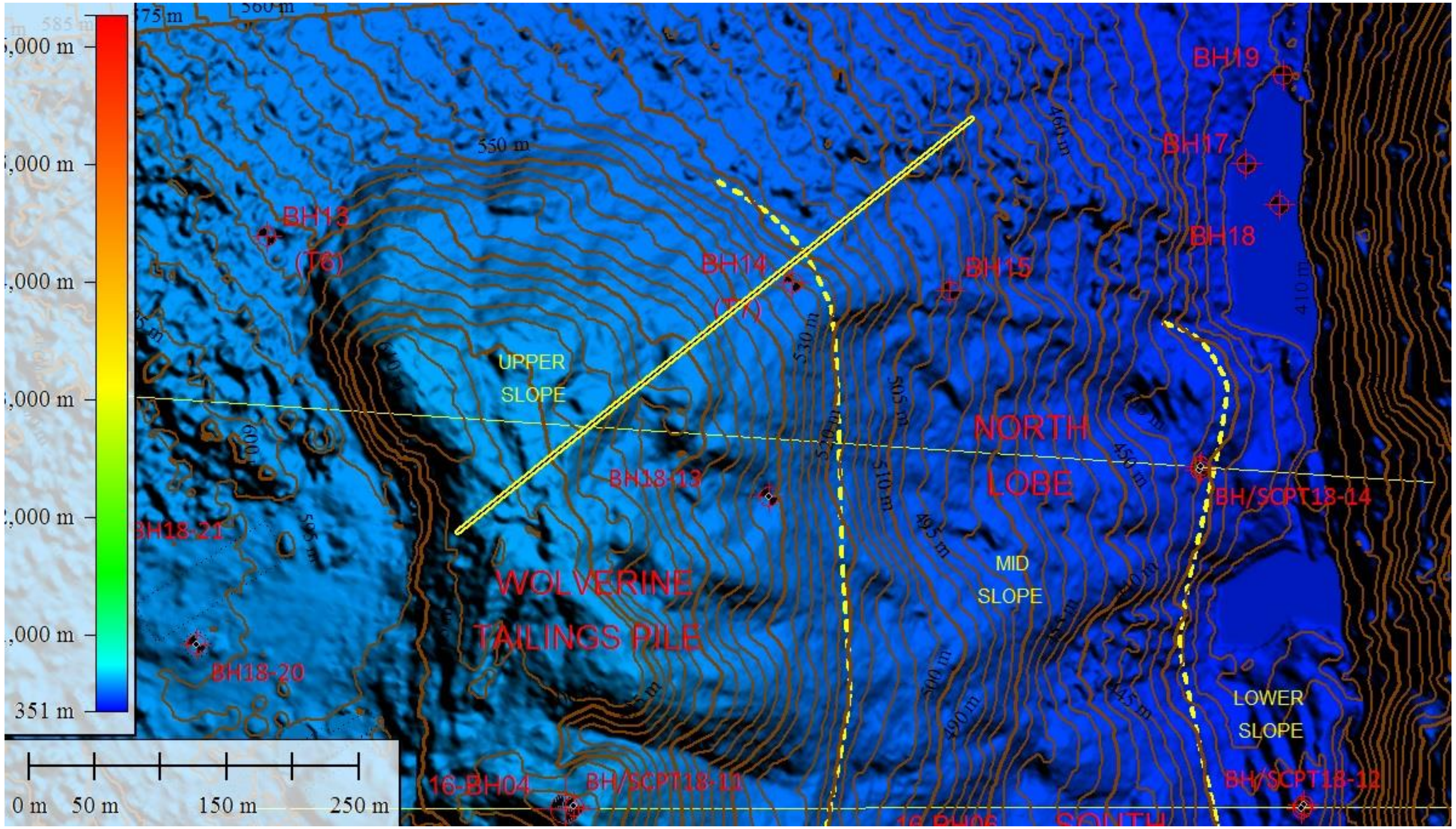
- Colluvium USR c/p’ = 0.20
- Added Waste Material over base until FoS = 1.00
- Phreatic surface 1 m above tailings/colluvium boundary
- WM area: 9,800 m^2
- ~450 m of WM placed from mid-slope to valley

FoS = 1.00



Color	Name	Unit Weight (kN/m^3)	Cohesion' (kPa)	Phi' (°)	Tau/Sigma Ratio
	Alluvium	20	0	25	
	Bedrock				
	Colluvium (unfrozen)	20			0.2
	Tailings	20	0	33	
	Waste Material	20	0	25	
	Weathered Bedrock	20	0	25	

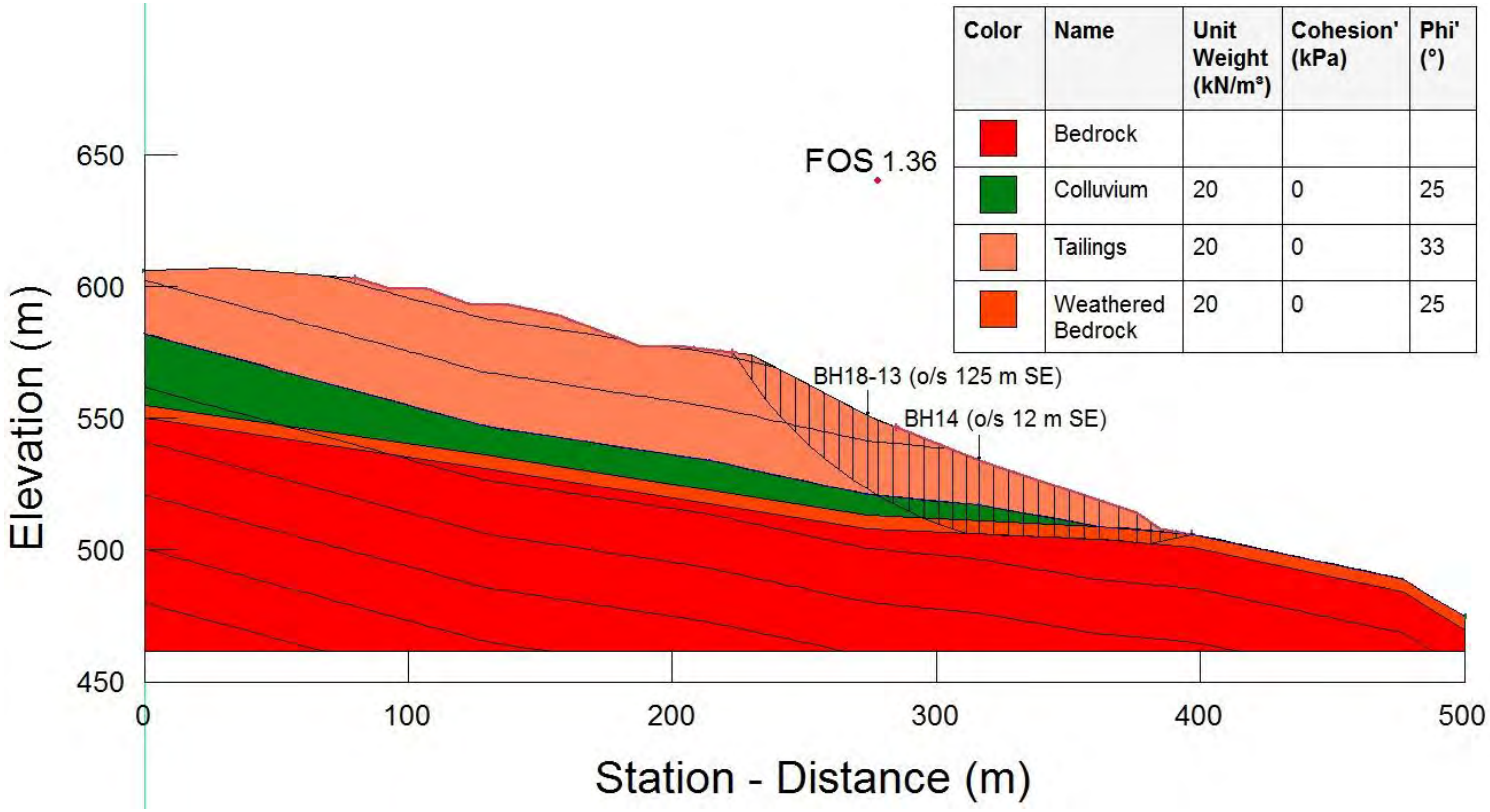
TS3 – Running SW – NE on Upper Slope



TS3 – Base State (Existing Slope)

- Colluvium, weathered bedrock use ϕ=25°, best estimate
- Tailings (unsaturated) use ϕ=33°
- Bedrock is impenetrable
- Phreatic surface follows the tailings/colluvium boundary
- Does not include any high ice content frozen material
- No waste material berm required for static case since FoS > 1.2

FoS = 1.36



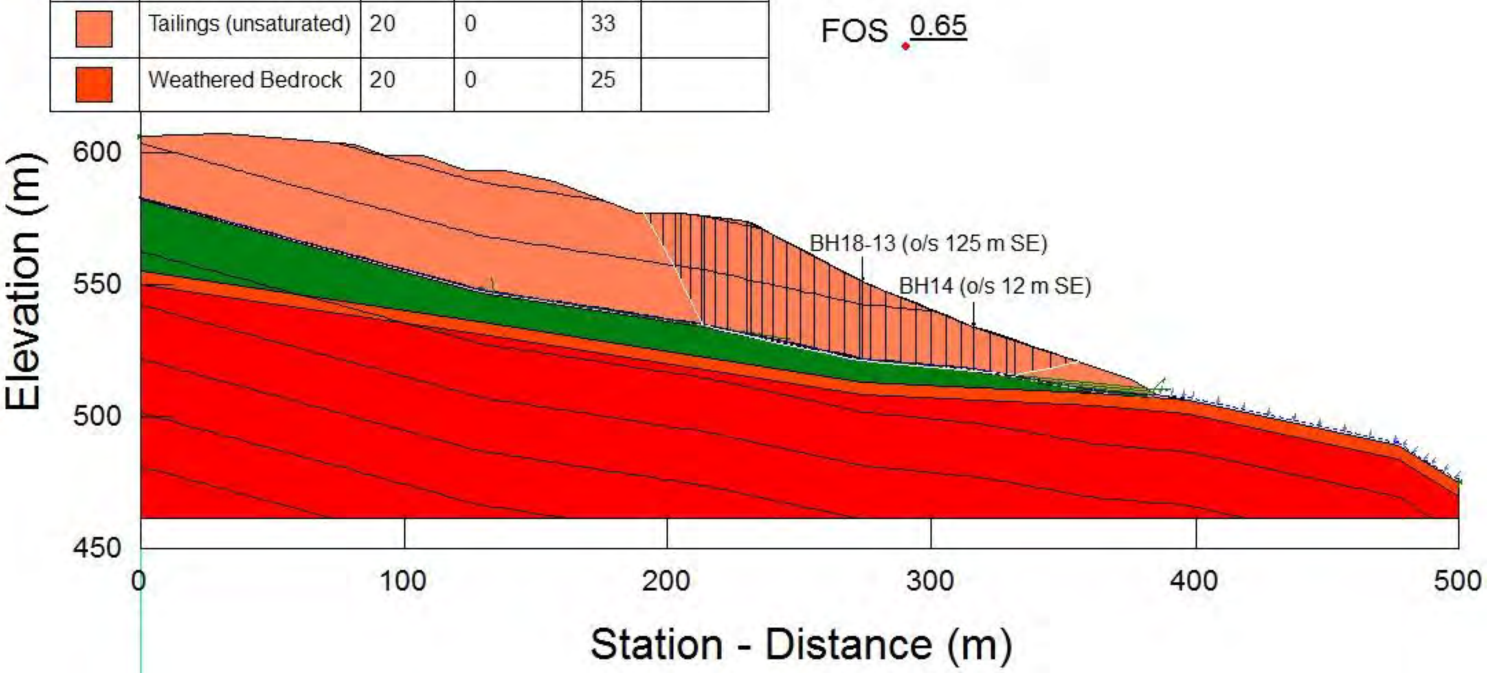
Color	Name	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)
	Bedrock			
	Colluvium	20	0	25
	Tailings	20	0	33
	Weathered Bedrock	20	0	25

TS3 – Option WC-2: Tailings Liquefied

- Tailings (saturated) c/p’ = 0.08
- Phreatic surface 1 m above tailings/colluvium boundary
- Berm required in tailings liquefaction scenario since FoS < 1.0

FoS = 0.65

Color	Name	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Tau/Sigma Ratio
	Bedrock				
	Colluvium	20	0	25	
	Tailings (saturated)	20			0.08
	Tailings (unsaturated)	20	0	33	
	Weathered Bedrock	20	0	25	



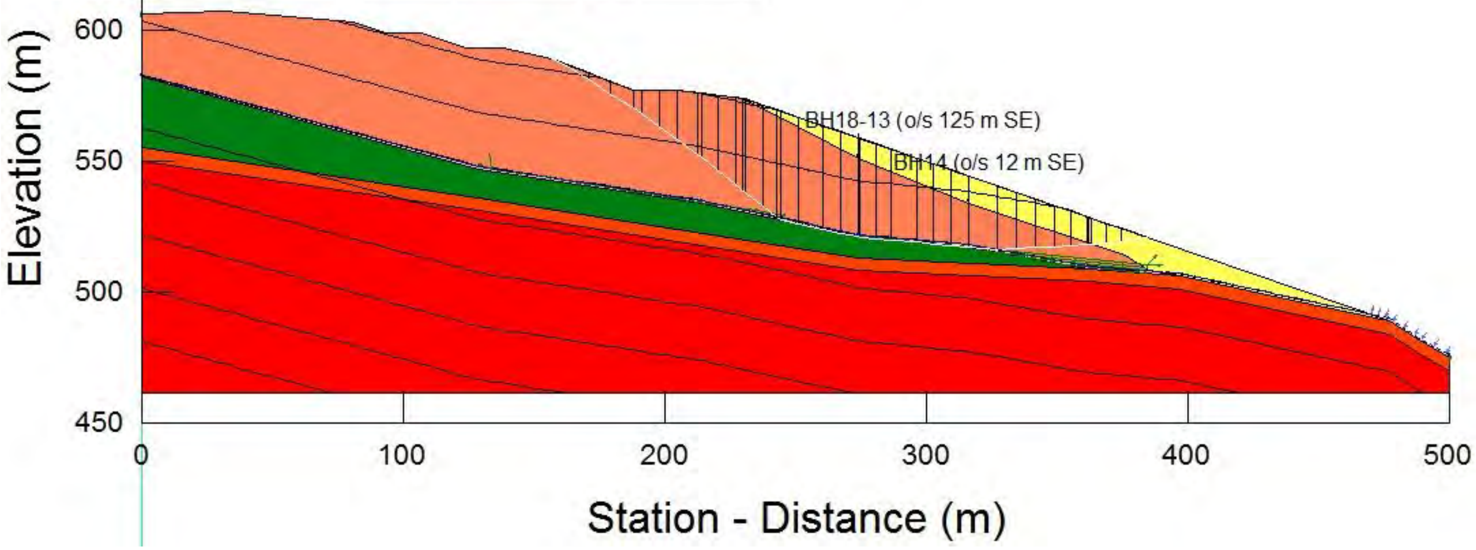
TS3 – Option WC-2: Tailings Liquefied, berm added

- Tailings (saturated) c/p’ = 0.08
- Phreatic surface 1 m above tailings/colluvium boundary
- WM area: 1800 m2
- ~220 m of WM placed from mid-slope to end

Color	Name	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Tau/Sigma Ratio
	Bedrock				
	Colluvium	20	0	25	
	Tailings (saturated)	20			0.08
	Tailings (unsaturated)	20	0	33	
	Waste Material	20	0	25	
	Weathered Bedrock	20	0	25	

FoS = 1.21

FOS 1.21

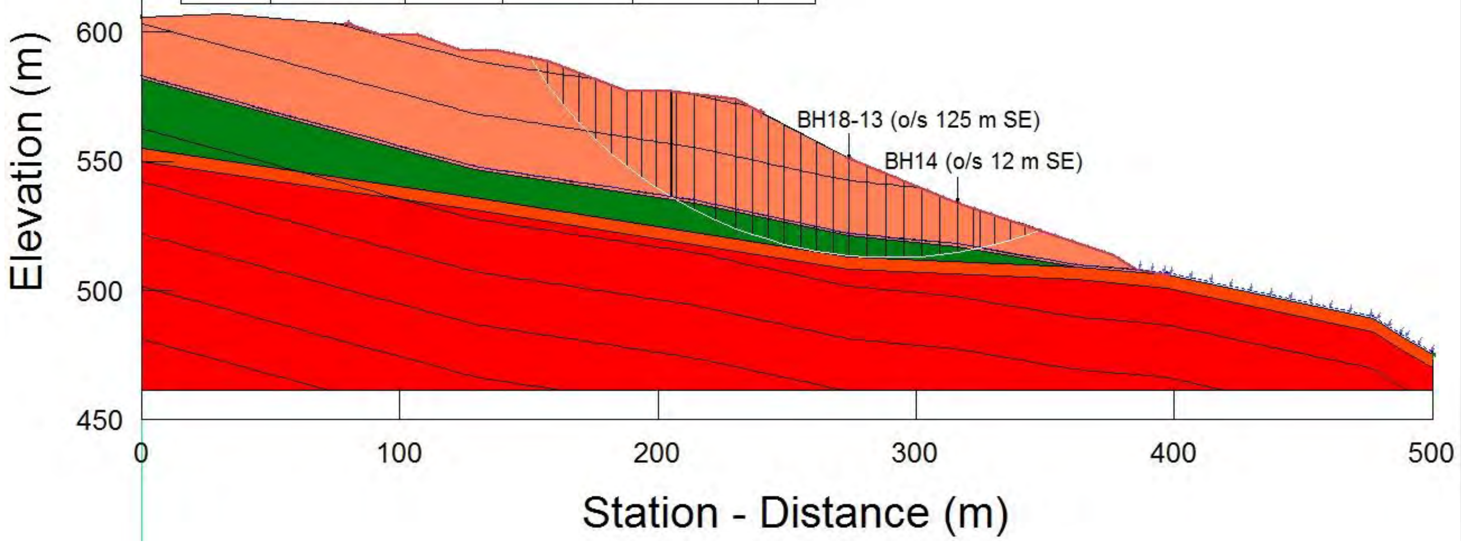


TS3 – Option WC-2: Colluvium undrained

- Colluvium c/p’ = 0.20
- Phreatic surface 1 m above tailings/colluvium boundary
- FoS > 1.0 in undrained colluvium scenario, no berm required

FoS = 1.05

Color	Name	Unit Weight (kN/m³)	Tau/Sigma Ratio	Cohesion' (kPa)	Phi' (°)
	Bedrock				
	Colluvium	20	0.2		
	Tailings	20		0	33
	Weathered Bedrock	20		0	25



Appendix III
Wolverine Creek Stability Assessment:
LCCA Option 3 E

Wolverine Creek Stability Assessment LCCA Option 3 E

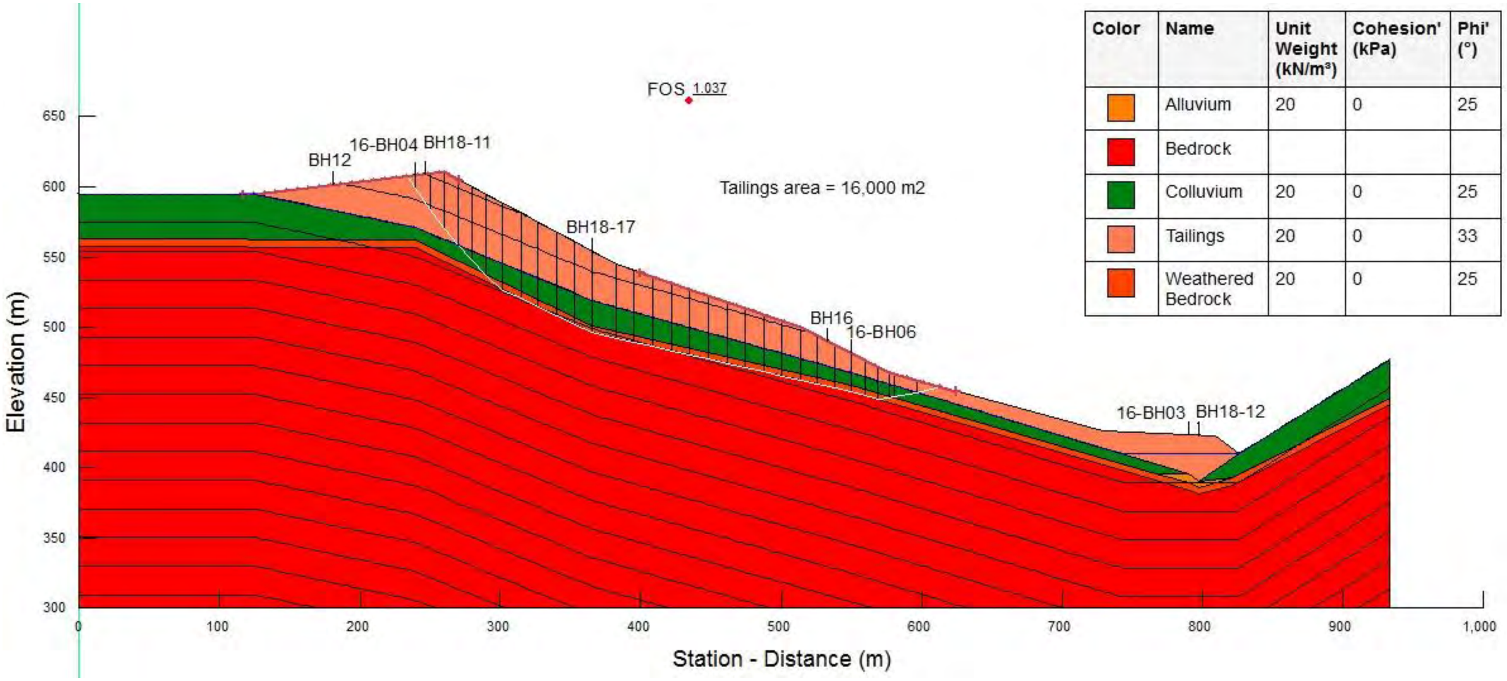
LCCA Option 3(E) requires cut/fill of the tailings to a slope of 3.75H:1V, with 1 m of waste material laid on top.

The 3.75H:1V slope toe is centered on Wolverine Creek, allowing the water conveyance to a location between the toe and the east slope of the valley wall.

North Lobe: TS1 – Base State (Existing Slope)

- Model from cross-section TS-1
- Colluvium, alluvium, weathered bedrock use ϕ=25°, best estimate
- Tailings use ϕ=33°
- Bedrock is impenetrable
- Phreatic surface follows the tailings/colluvium boundary
- Does not include any high ice-content frozen material

FoS = 1.04

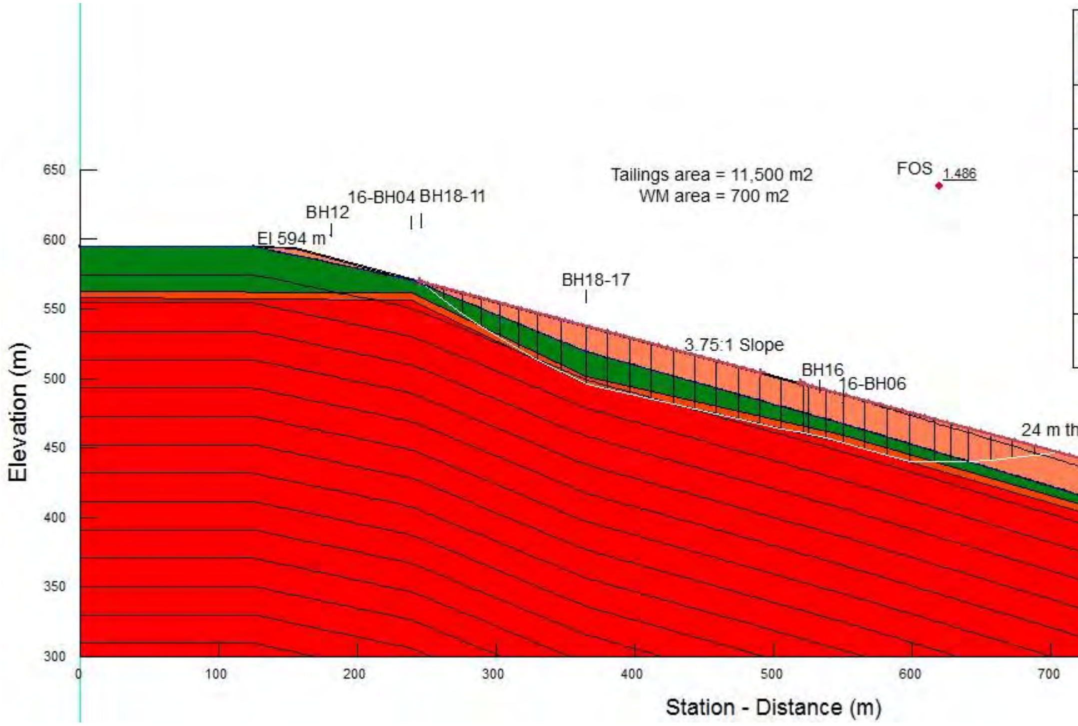


Color	Name	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)
	Alluvium	20	0	25
	Bedrock			
	Colluvium	20	0	25
	Tailings	20	0	33
	Weathered Bedrock	20	0	25

TS1 – Static Analysis, Slope adjusted

- Tailings smoothened out to constant 3.75:1 slope
- Toe built to east slope at El 422 m
- Phreatic surface follows tailings/colluvium boundary
- Tailings area: 11,500 m2
- 1 m of waste material overlaying tailings

FoS = 1.49



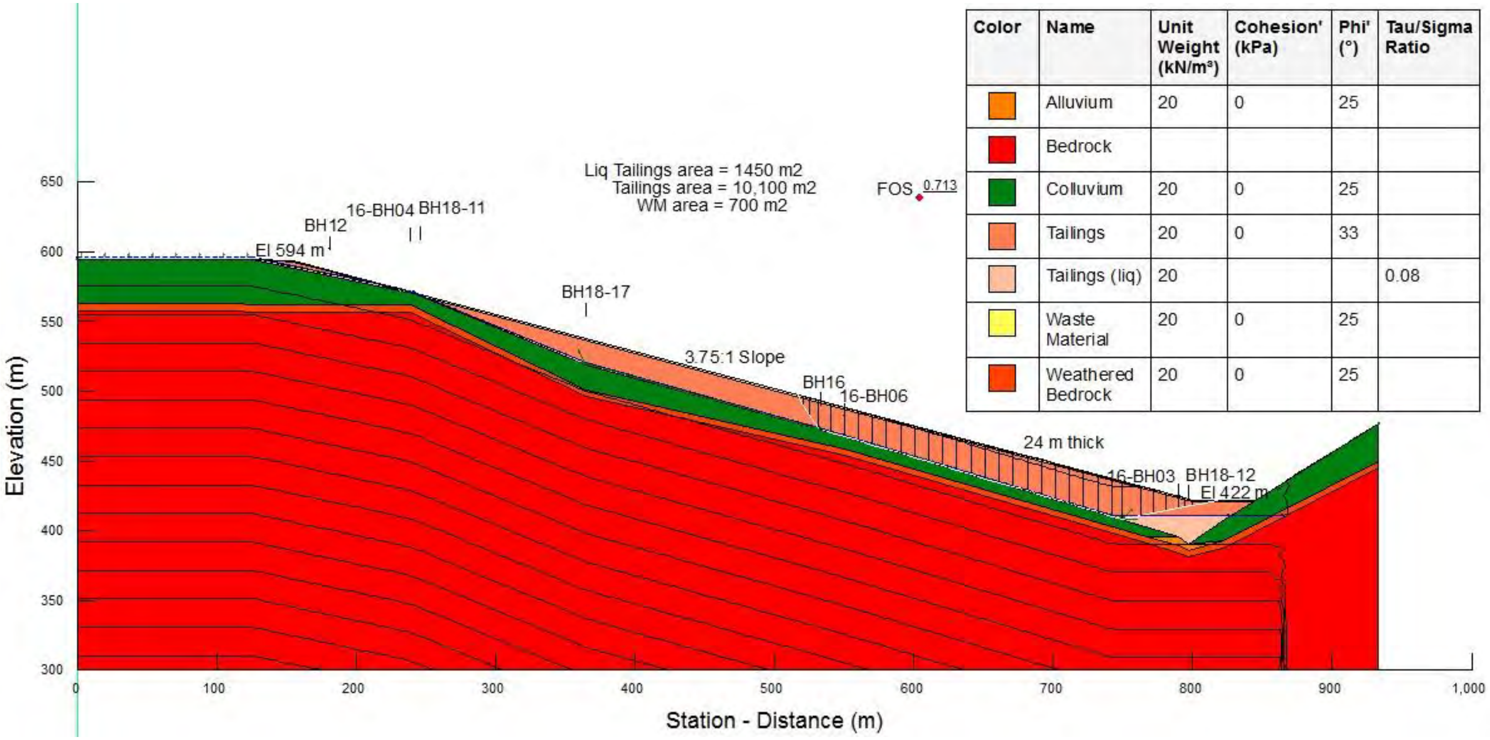
Color	Name	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)
	Alluvium	20	0	25
	Bedrock			
	Colluvium	20	0	25
	Tailings	20	0	33
	Waste Material	20	0	25
	Weathered Bedrock	20	0	25

Static stability satisfactory

TS1 –Tailings Liquefied, Slope adjusted

- Tailings liquefied with c/p’ = 0.08
- Phreatic surface 1 m above tailings/colluvium boundary
- Liq tailings area: 1450 m2
- Dry tailings area: 10,100 m2
- Block failure along liquefied tailings zone

FoS = 0.71



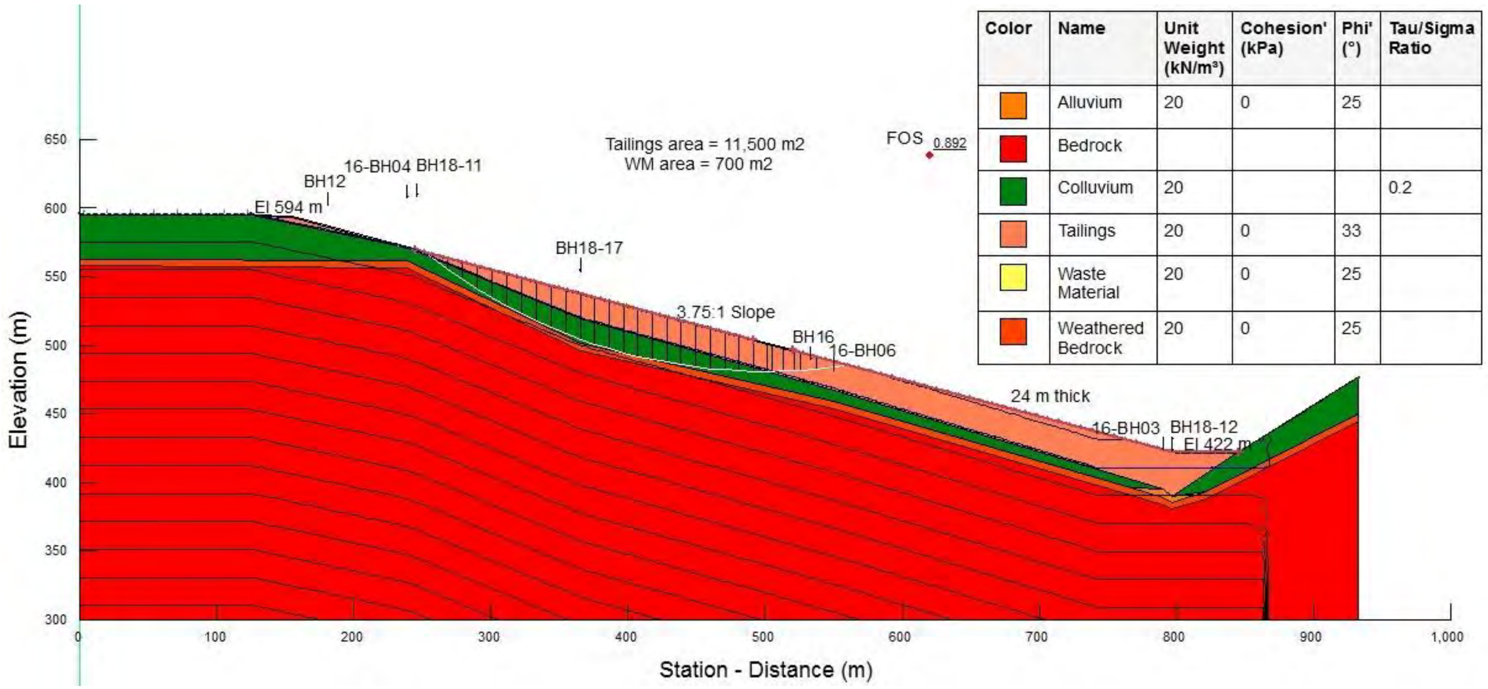
Color	Name	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Tau/Sigma Ratio
	Alluvium	20	0	25	
	Bedrock				
	Colluvium	20	0	25	
	Tailings	20	0	33	
	Tailings (liq)	20			0.08
	Waste Material	20	0	25	
	Weathered Bedrock	20	0	25	

If water table rises into tailings post seismic unstable

TS1 – Colluvium undrained, Slope adjusted

- Colluvium c/p’ = 0.20
- Phreatic surface 1 m above tailings/colluvium boundary

FoS = 0.89

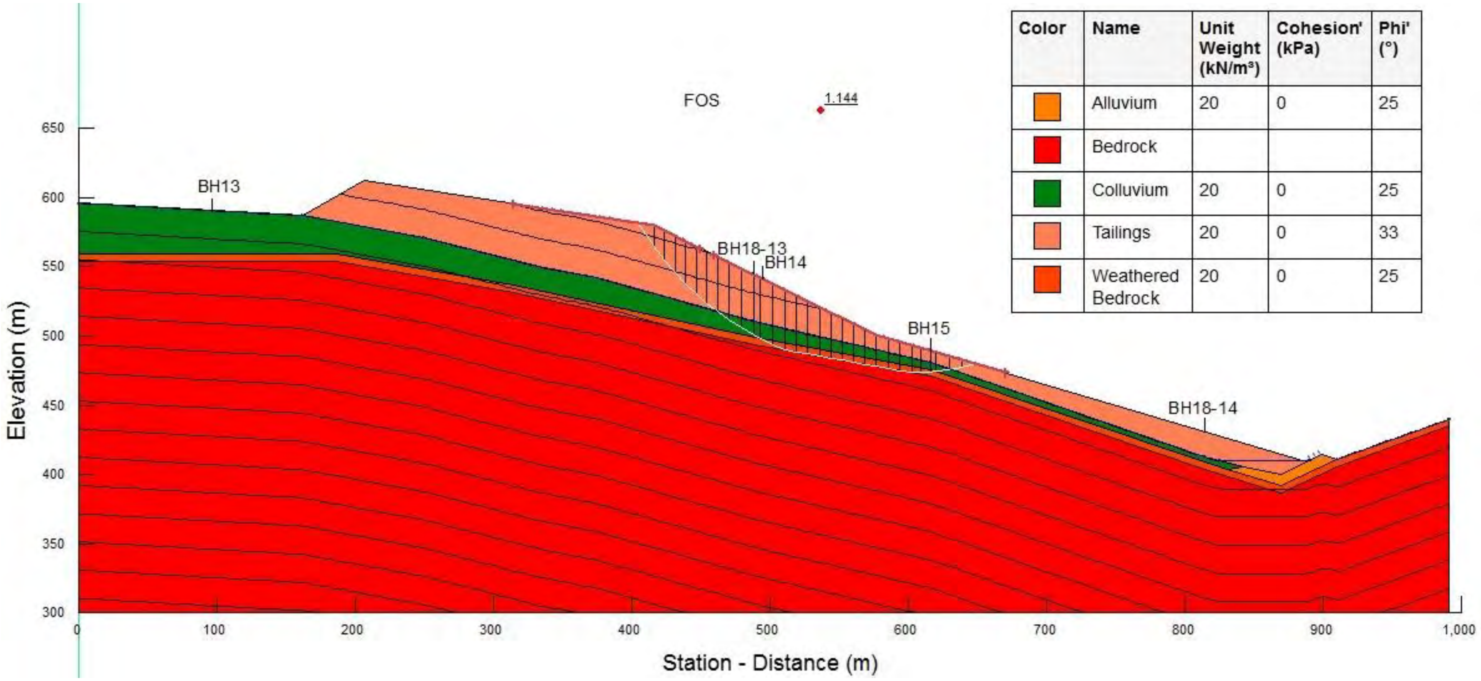


Color	Name	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Tau/Sigma Ratio
	Alluvium	20	0	25	
	Bedrock				
	Colluvium	20			0.2
	Tailings	20	0	33	
	Waste Material	20	0	25	
	Weathered Bedrock	20	0	25	

South Lobe: TS2 – Base State (Existing Slope)

- Model from cross-section TS-2
- Colluvium, alluvium, weathered bedrock use $\phi=25^{o}$, best estimate
- Tailings use $\phi=33^{o}$
- Bedrock is impenetrable
- Phreatic surface follows the tailings/colluvium boundary
- Does not include any high ice-content frozen material

FoS = 1.14

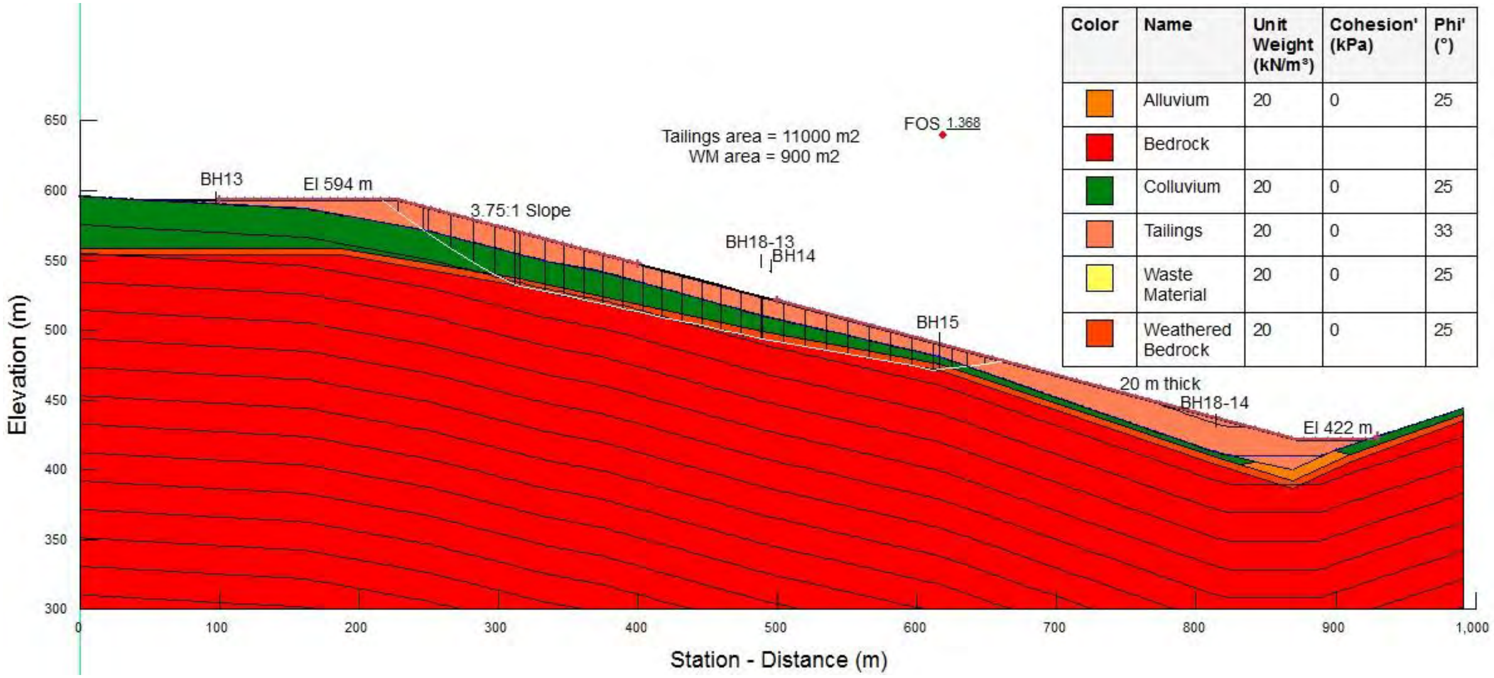


Color	Name	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)
	Alluvium	20	0	25
	Bedrock			
	Colluvium	20	0	25
	Tailings	20	0	33
	Weathered Bedrock	20	0	25

TS2 – Static Analysis, Slope adjusted

- Tailings smoothened out to constant 3.75:1 slope
- Toe built to east slope at El 422 m
- Phreatic surface follows tailings/colluvium boundary
- Tailings area: 11,000 m2
- 1 m of waste material overlaying tailings

FoS = 1.37

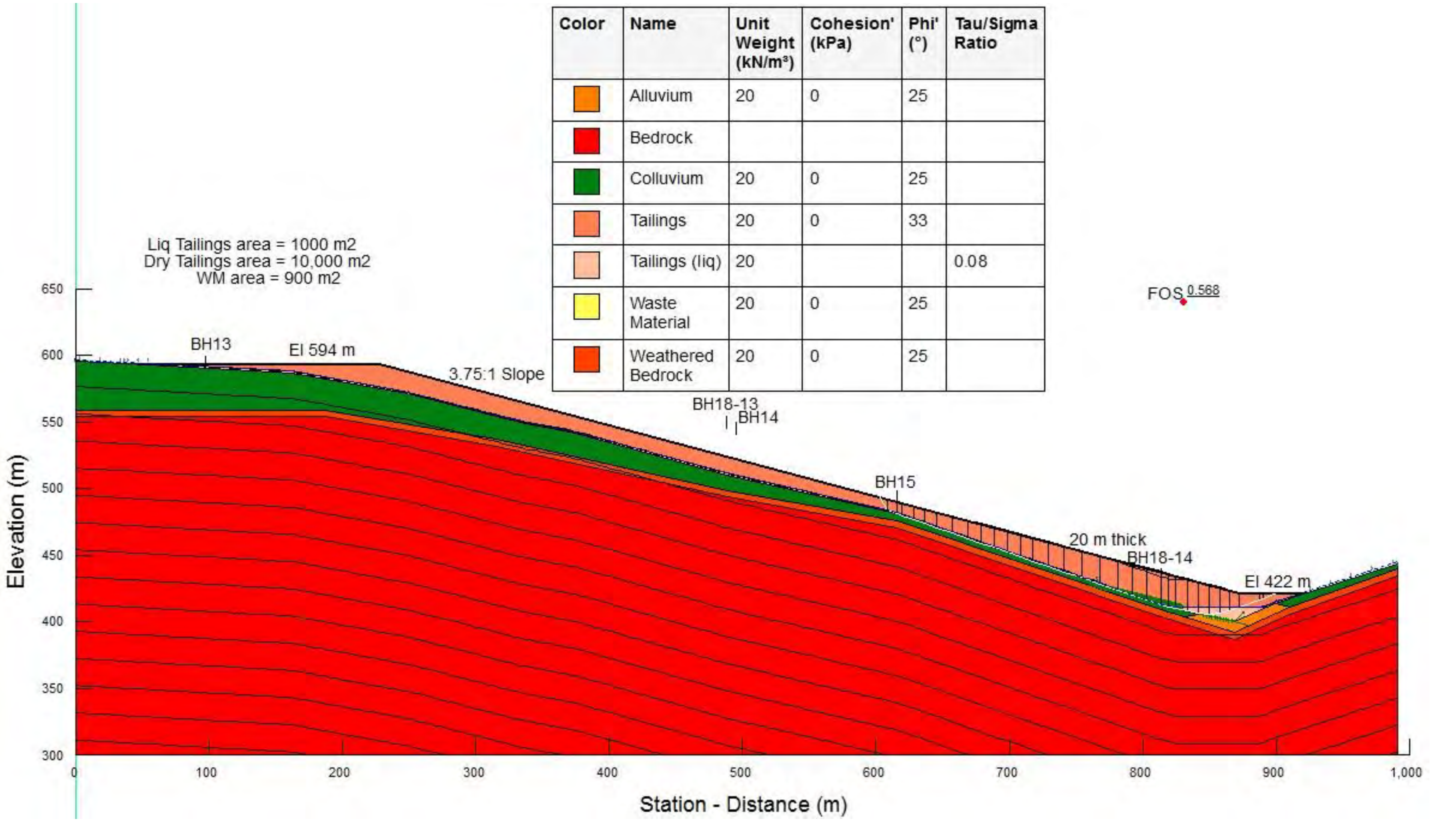


Static stability satisfactory

TS2 – Tailings Liquefied, Slope adjusted

FoS = 0.57

- Tailings liquefied with c/p’ = 0.08
- Phreatic surface 1 m above tailings/colluvium boundary
- Liq tailings area: 1,000 m2
- Dry tailings area: 10,000 m2
- Block failure along liquefied tailings zone



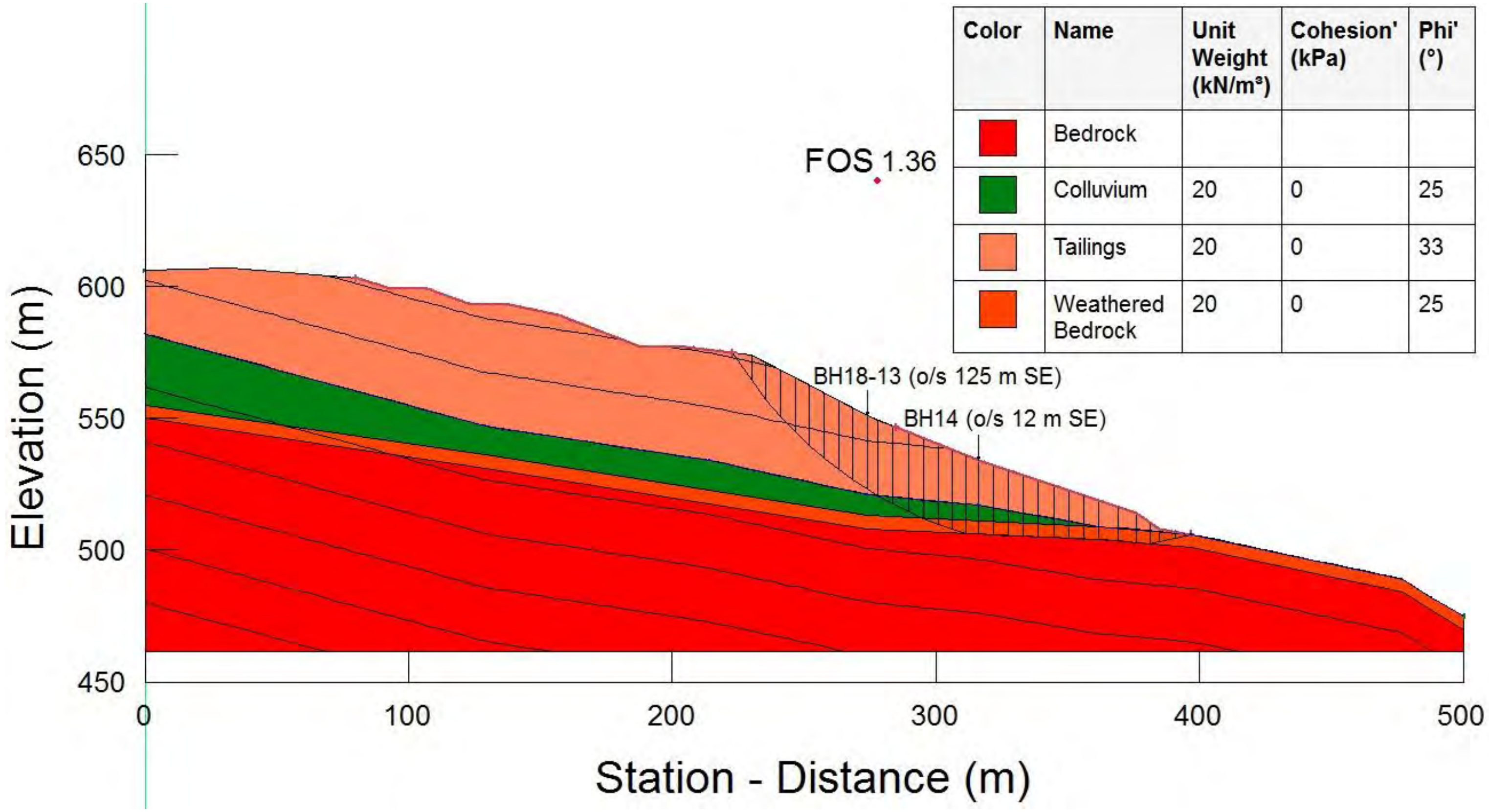
Color	Name	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Tau/Sigma Ratio
	Alluvium	20	0	25	
	Bedrock				
	Colluvium	20	0	25	
	Tailings	20	0	33	
	Tailings (liq)	20			0.08
	Waste Material	20	0	25	
	Weathered Bedrock	20	0	25	

If water table rises into tailings post seismic unstable

TS3 – Base State (Existing Slope)

- Colluvium, weathered bedrock use ϕ=25°
- Tailings use ϕ=33°
- Bedrock is impenetrable
- Phreatic surface follows the tailings/colluvium boundary
- Does not include any high ice-content frozen material

FoS = 1.36

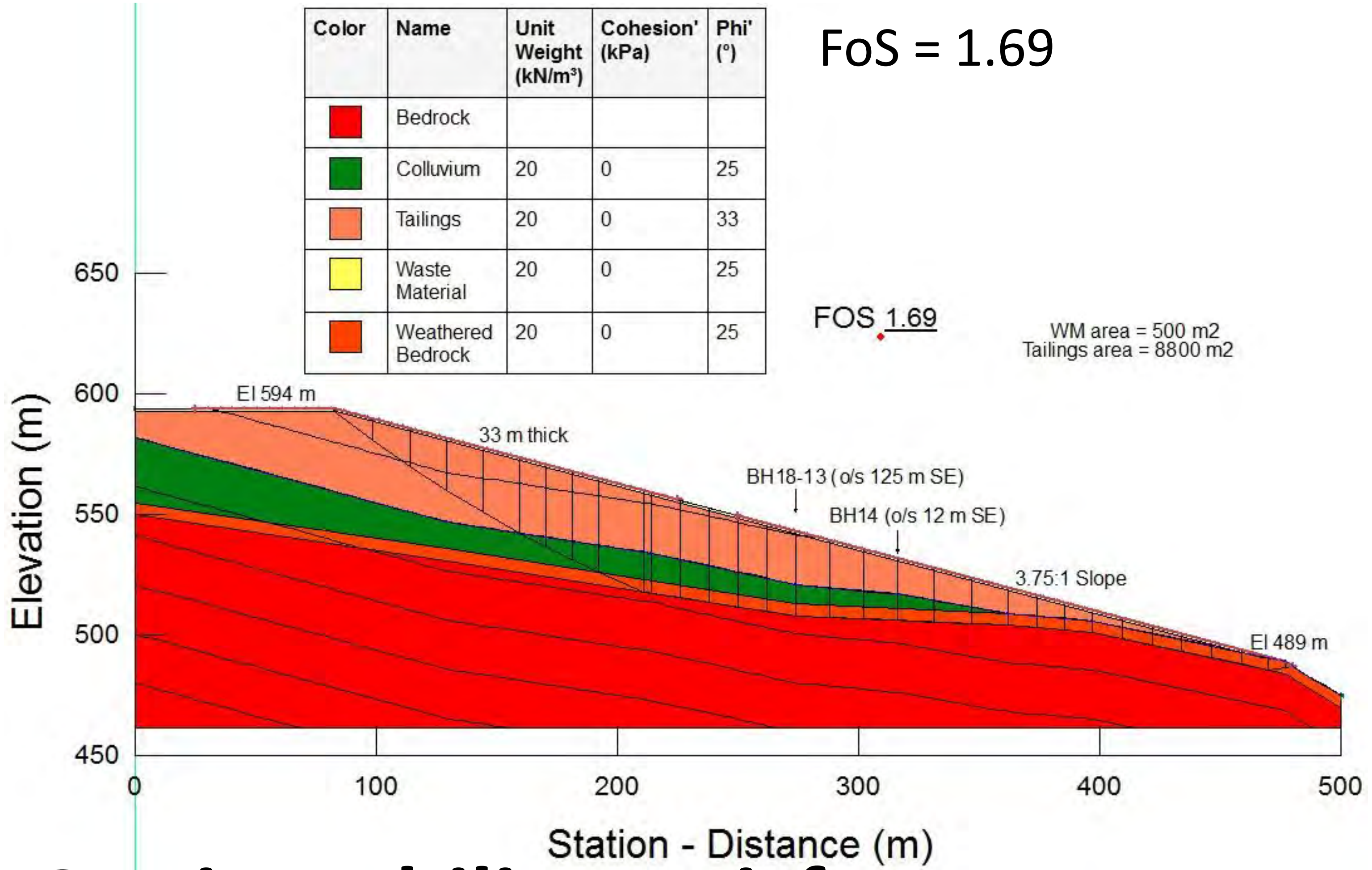


Color	Name	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)
	Bedrock			
	Colluvium	20	0	25
	Tailings	20	0	33
	Weathered Bedrock	20	0	25

TS3 – Static Analysis, Slope adjusted

- Tailings smoothened out to constant 3.75:1 slope
- Crest at El 594 m
- Phreatic surface follows tailings/colluvium boundary
- Tailings area: 8,800 m2
- 1 m of waste material overlaying tailings

FoS = 1.69



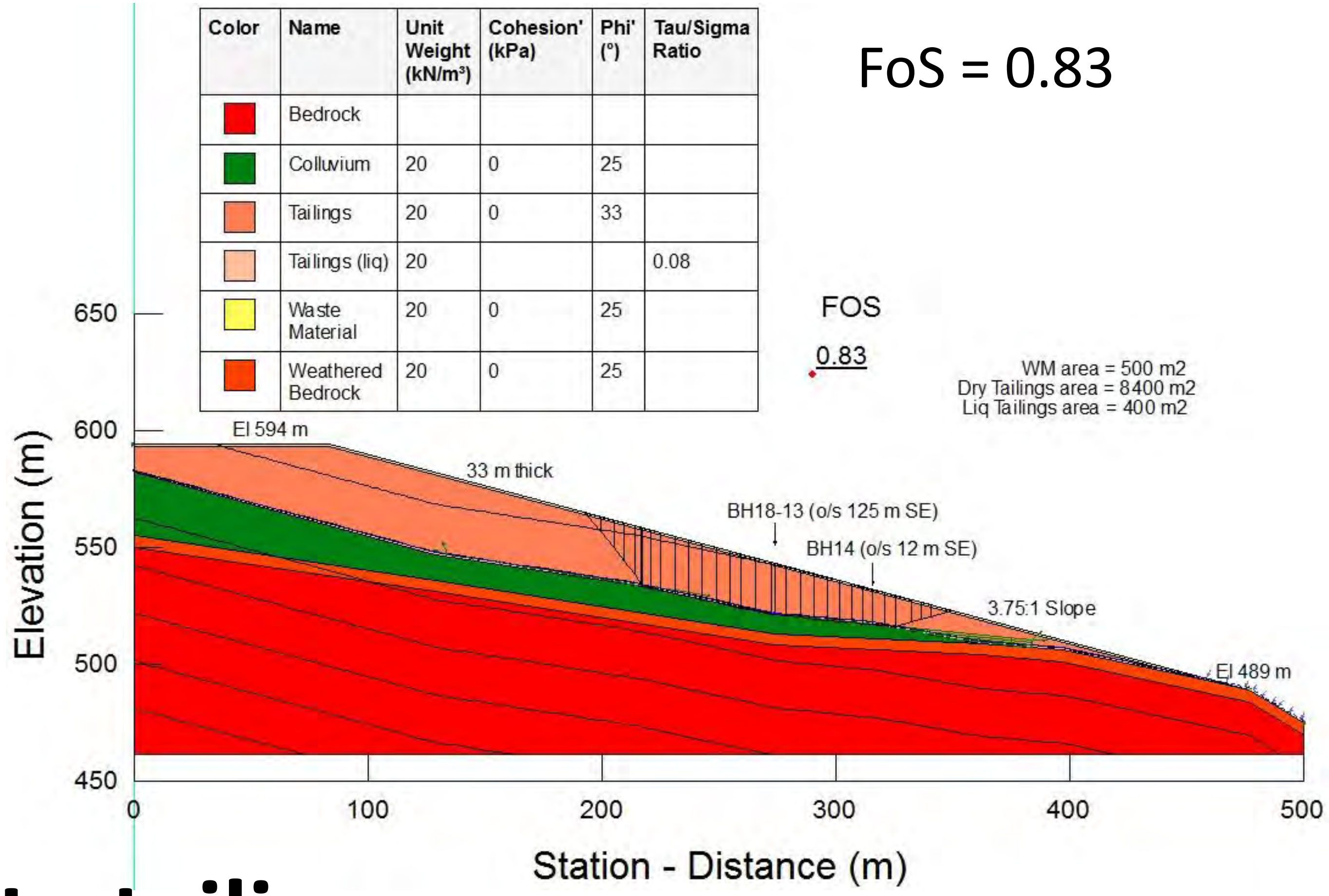
Color	Name	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)
	Bedrock			
	Colluvium	20	0	25
	Tailings	20	0	33
	Waste Material	20	0	25
	Weathered Bedrock	20	0	25

Static stability satisfactory

TS3 – Tailings Liquefied, Slope adjusted

- Tailings liquefied with c/p’ = 0.08
- Phreatic surface 1 m above tailings/colluvium boundary
- Block failure along liquefied tailings zone near toe

FoS = 0.83



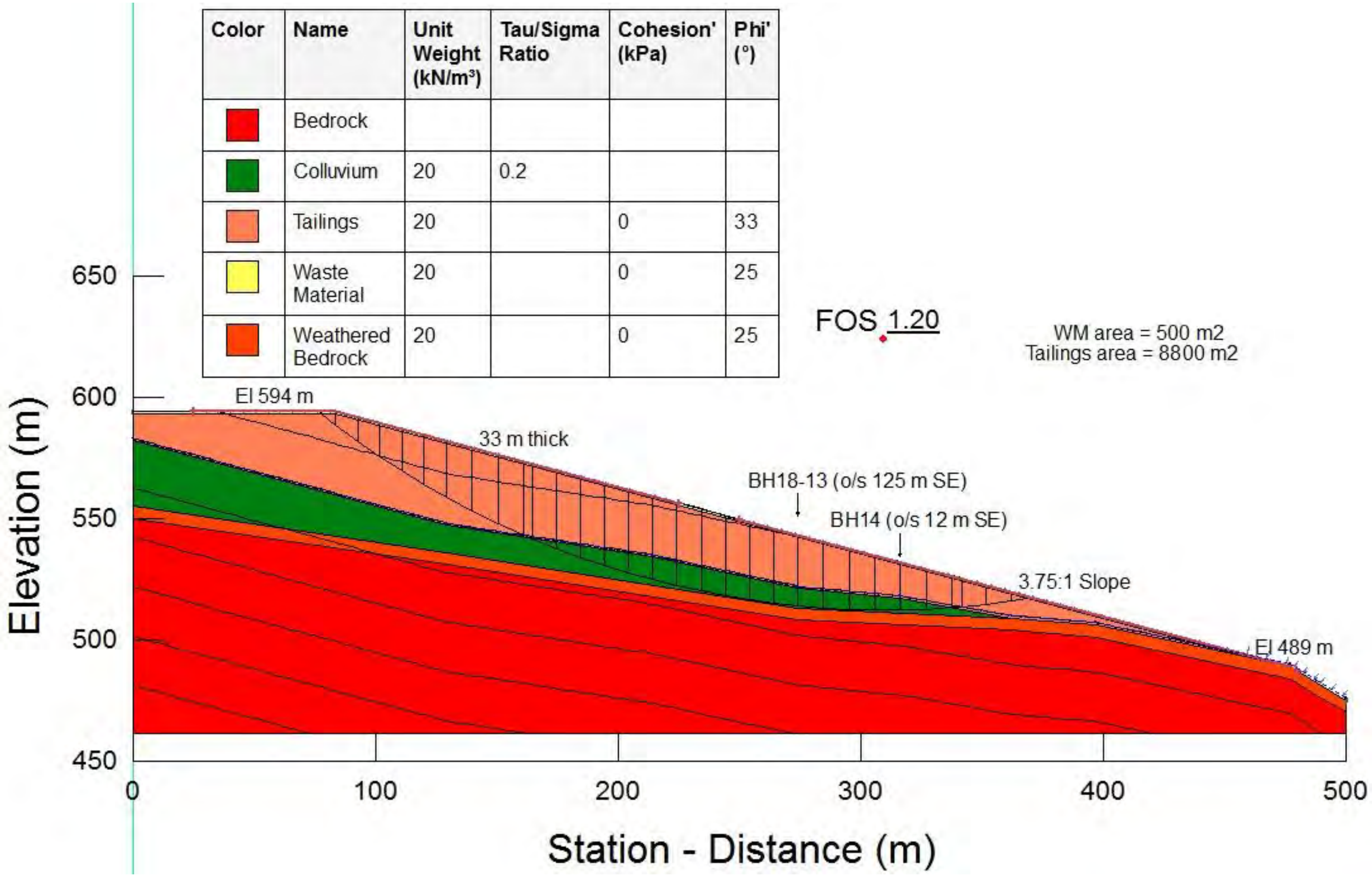
Color	Name	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Tau/Sigma Ratio
	Bedrock				
	Colluvium	20	0	25	
	Tailings	20	0	33	
	Tailings (liq)	20			0.08
	Waste Material	20	0	25	
	Weathered Bedrock	20	0	25	

If water table rises into tailings post seismic unstable

TS3 – Colluvium undrained, Slope adjusted

- Colluvium c/p’ = 0.20
- Phreatic surface 1 m above tailings/colluvium boundary

FoS = 1.20



Color	Name	Unit Weight (kN/m³)	Tau/Sigma Ratio	Cohesion' (kPa)	Phi' (°)
	Bedrock				
	Colluvium	20	0.2		
	Tailings	20		0	33
	Waste Material	20		0	25
	Weathered Bedrock	20		0	25

Appendix IV
Wolverine Creek Stability Assessment Yield Accelerations of Static Stability Cases for Options WC2 & LCCA Option E

Wolverine Creek Stability Assessment Yield Accelerations of Static Stability Cases for Options WC2 & LCCA Option E

These analyses are required to undertake seismic movement analysis

TS1 (South Lobe)

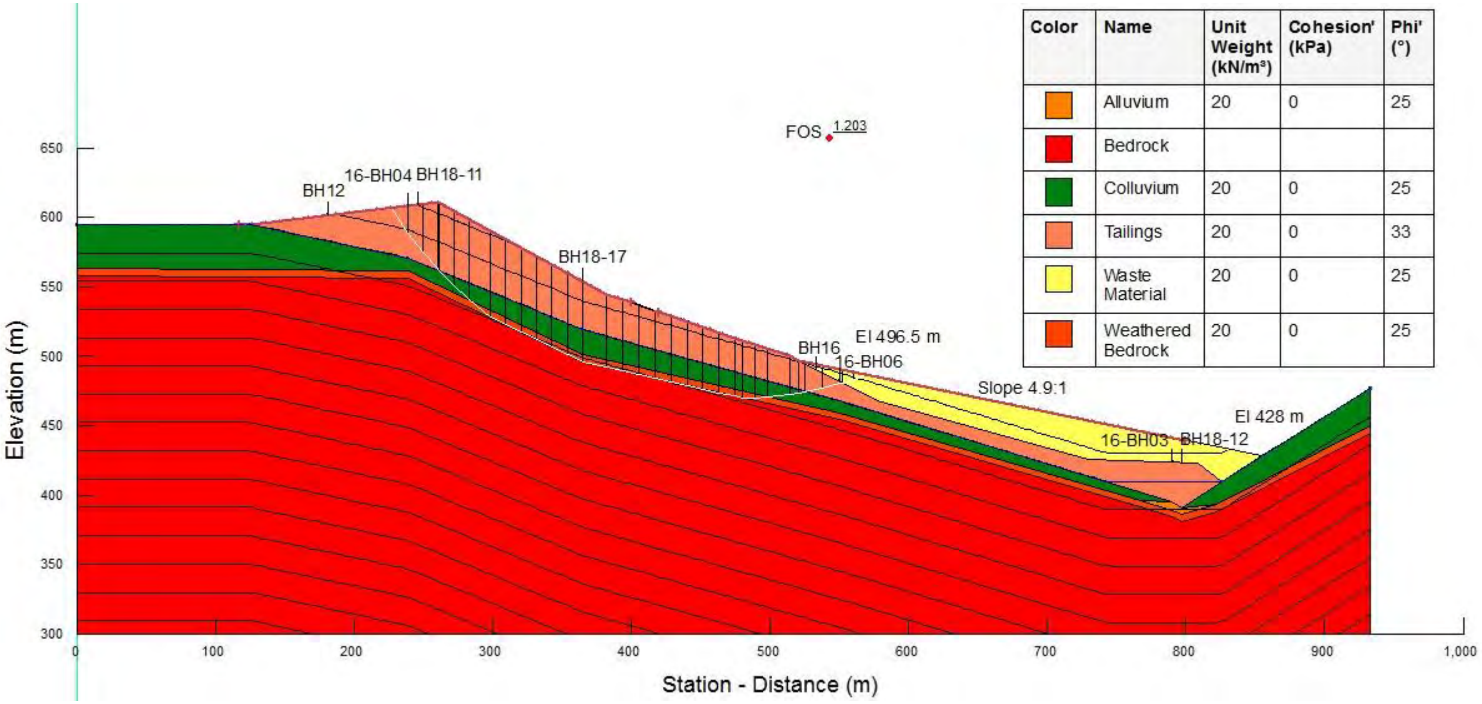
Option WC2 involves storing Waste Material berm over the toe of the tailings slope until reaching a FoS = 1.2 in the static scenario

LCCA Option E involves adjusting the tailings to a slope of 3.75 H to 1 V, with 1 m of waste material laid on top

TS1 – Option WC2: Static Analysis, berm added

- Added Waste Material over base until FoS= 1.20
- Phreatic surface follows the tailings/colluvium boundary
- WM area: 6240 m2
- ~330 m of WM placed from mid-slope to valley

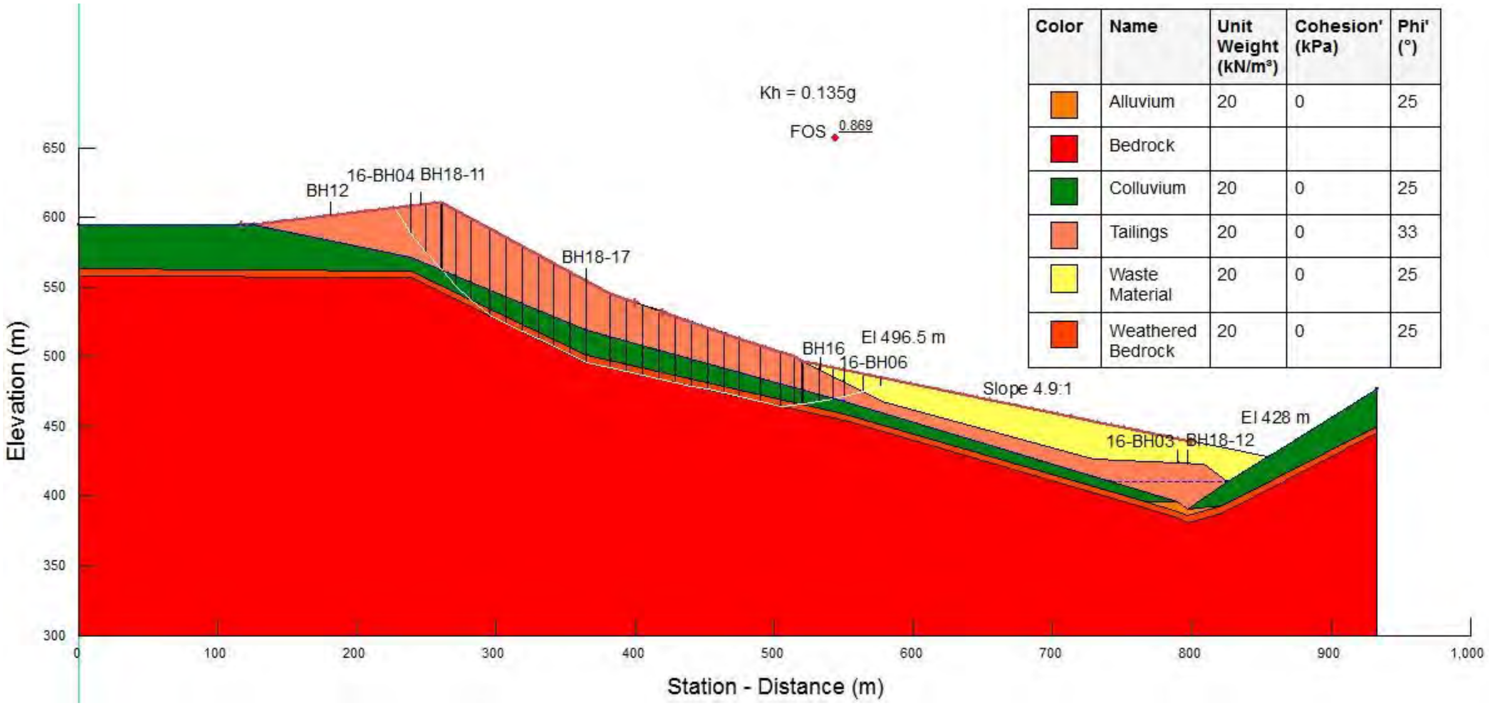
FoS = 1.20



TS1 – Option WC2: Seismic Analysis

- No liquefaction
- Phreatic surface follows tailings/colluvium boundary
- Using Griffen and Franklin (1984) method with Kh = 0.135g (50% of PGA = 0.27)
- FOS = 0.87 less than acceptable for this method.

FoS = 0.87

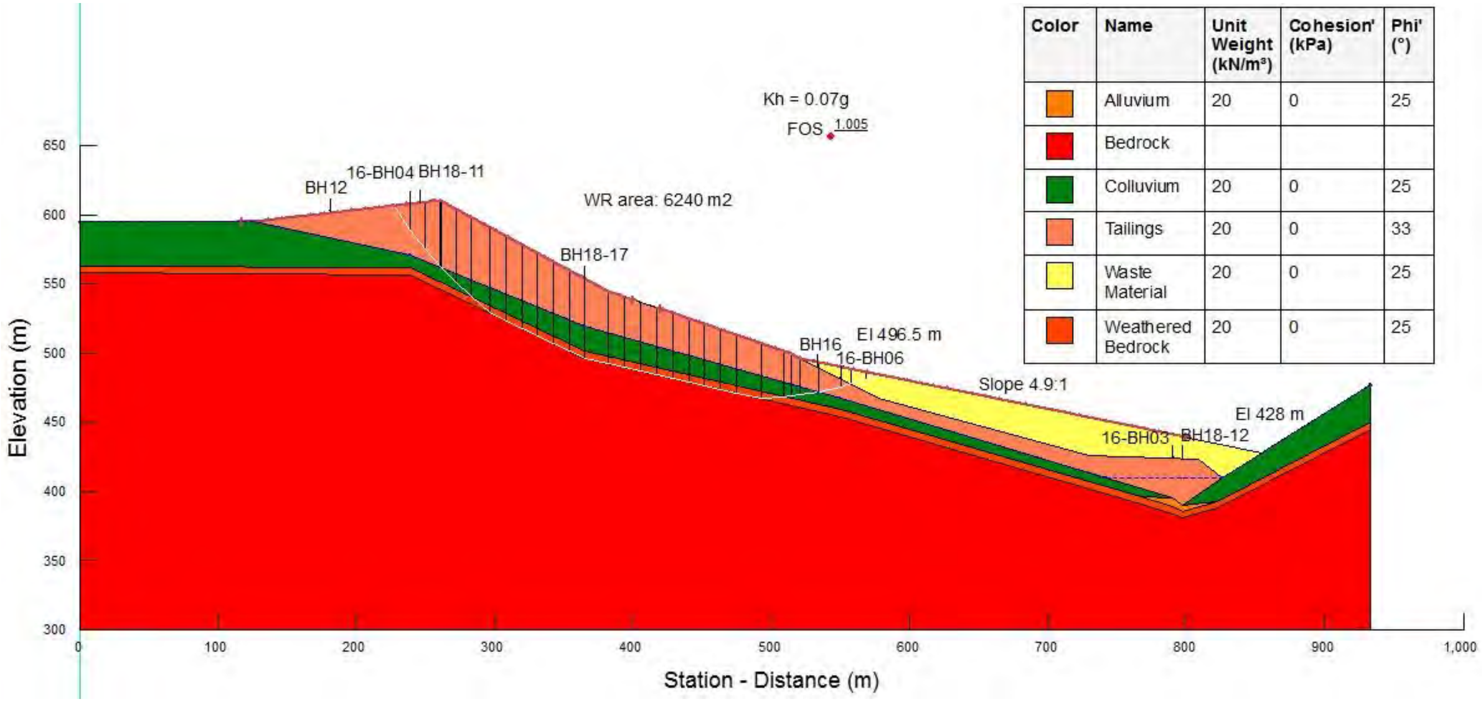


Color	Name	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)
	Alluvium	20	0	25
	Bedrock			
	Colluvium	20	0	25
	Tailings	20	0	33
	Waste Material	20	0	25
	Weathered Bedrock	20	0	25

TS1 – Option WC2: Seismic Analysis, Unity

- Kh adjusted until reaching FoS => 1.0
- Kyield = 0.07g for input into Bray and Tvarasarou (2007) model.
- See memo for movement prediction.

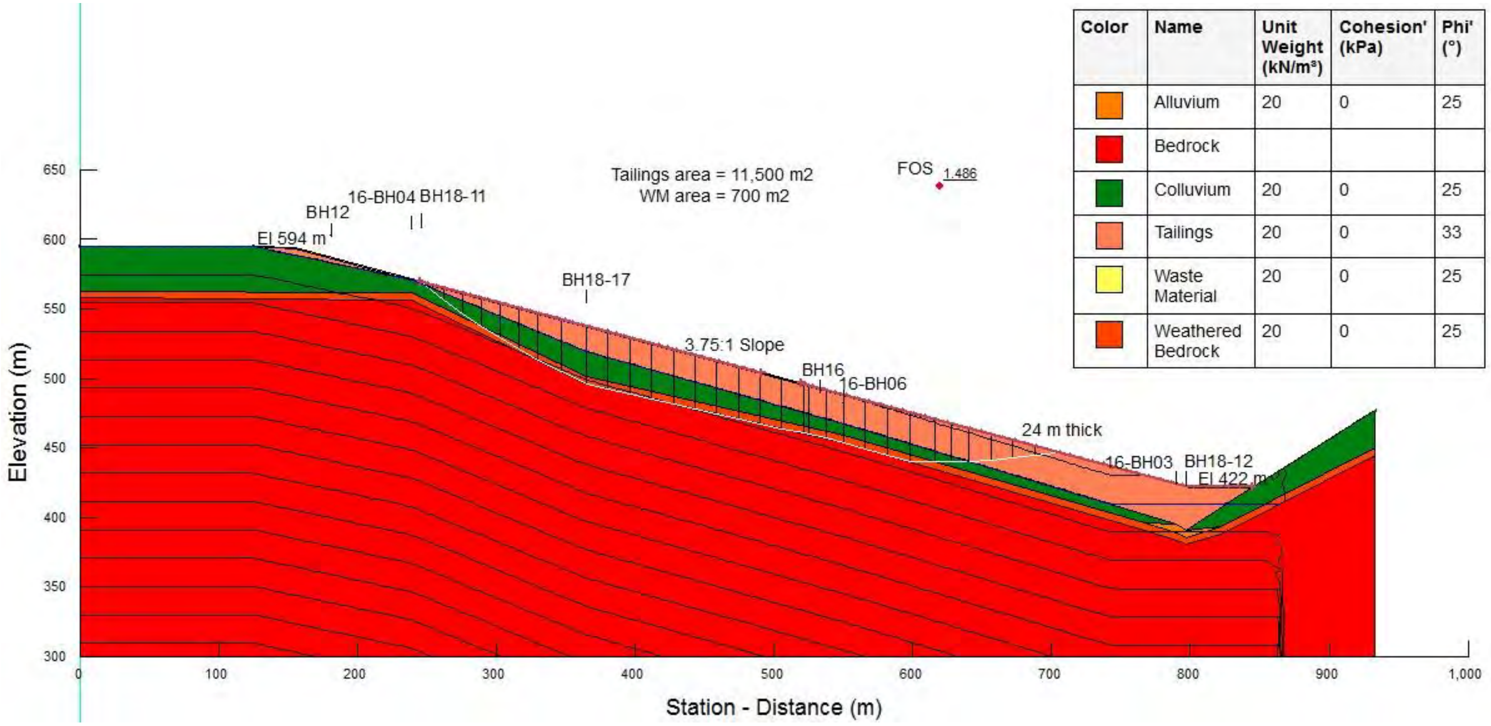
FoS = 1.01



TS1 – LCAA Option E: Static Analysis

- Tailings smoothened out to constant 3.75:1 slope
- 1 m of waste material overlaying tailings
- Toe extended to east slope at El 422 m
- Phreatic surface follows tailings/colluvium boundary
- Tailings area: 11,500 m2

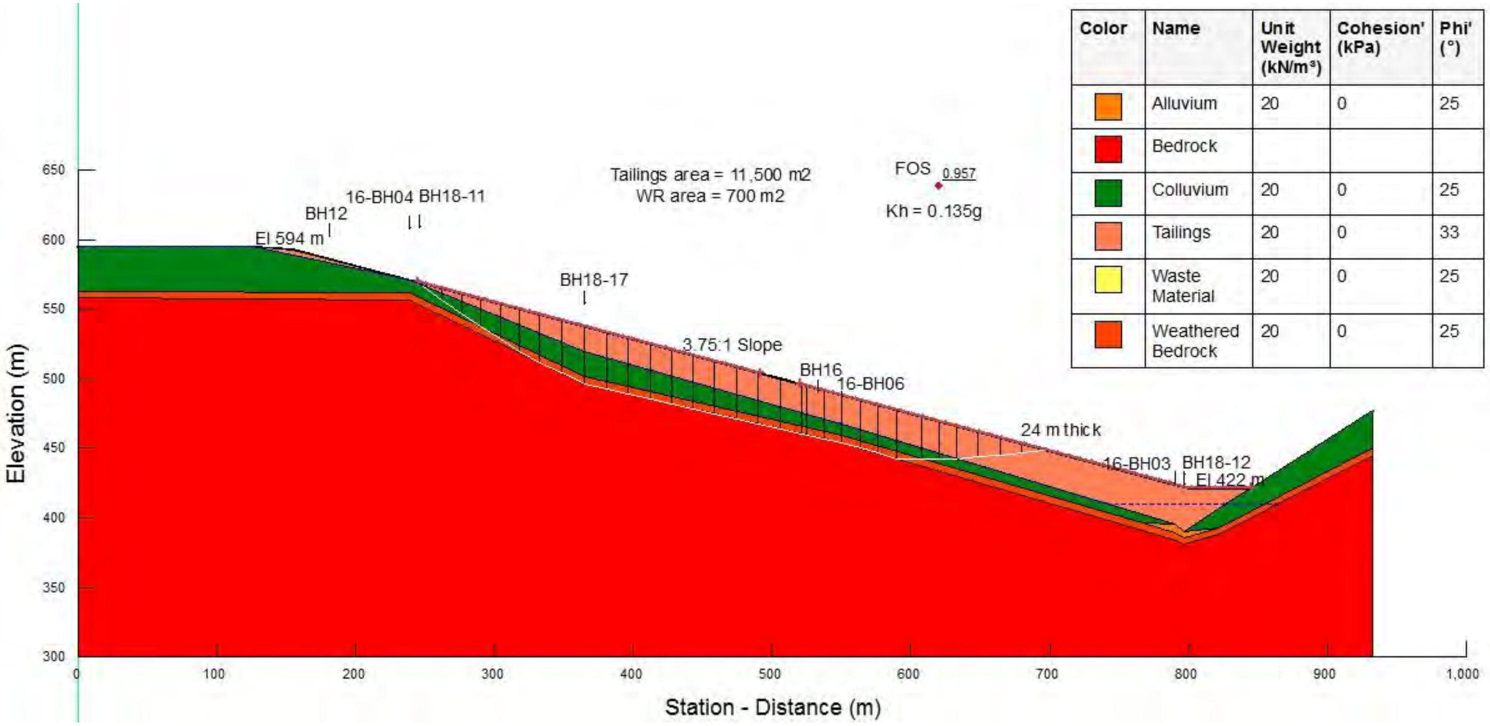
FoS = 1.49



TS1 – LCAA Option E: Seismic Analysis

- No liquefaction
- Phreatic surface follows tailings/colluvium boundary
- Using Griffen and Franklin (1984) method with Kh = 0.135g (50% of PGA = 0.27)
- Kh = 0.135g results in FoS of 0.96, rounds off to movement less than 1m.

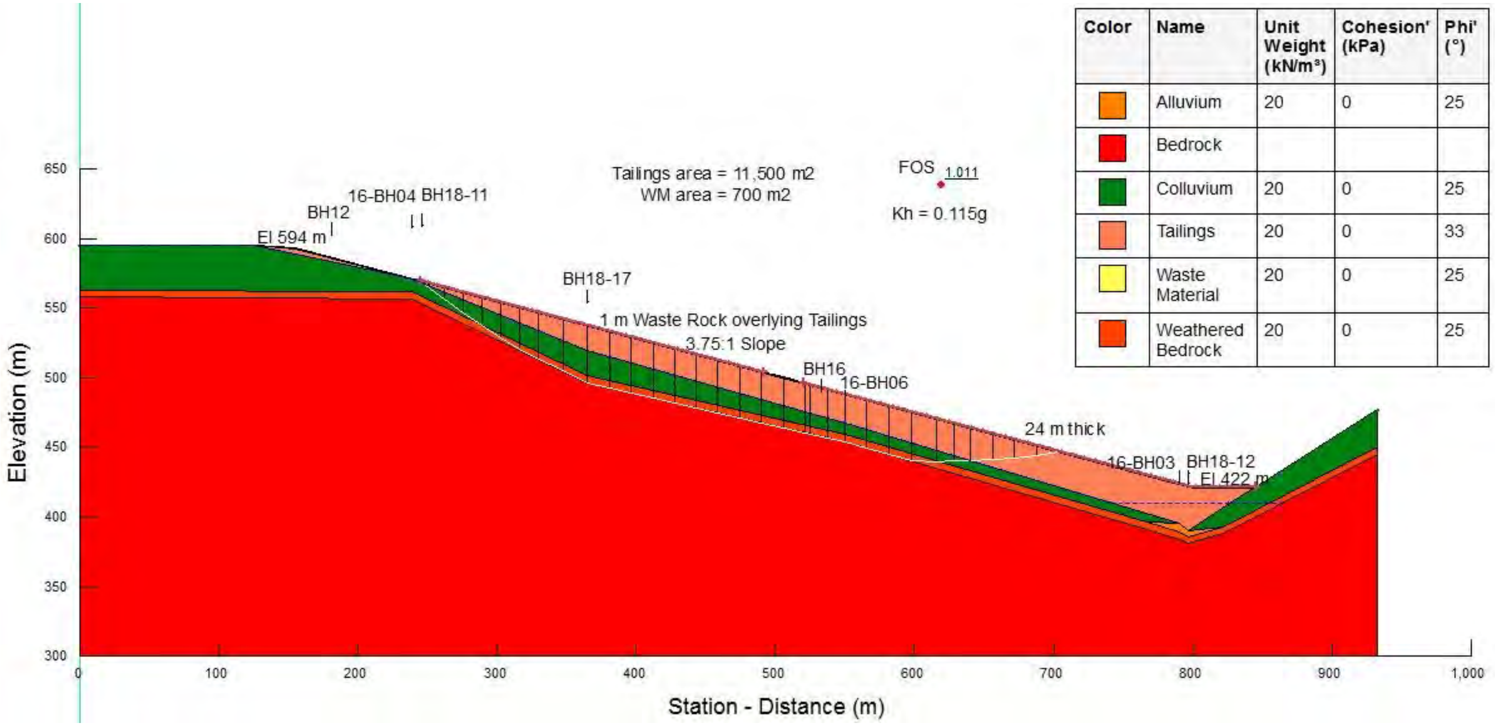
FoS = 0.96



TS1 – LCAA Option E: Seismic Analysis

- Kh adjusted until reaching FoS => 1.0
- Kyield = 0.115g
- See memo for movement prediction

FoS = 1.01



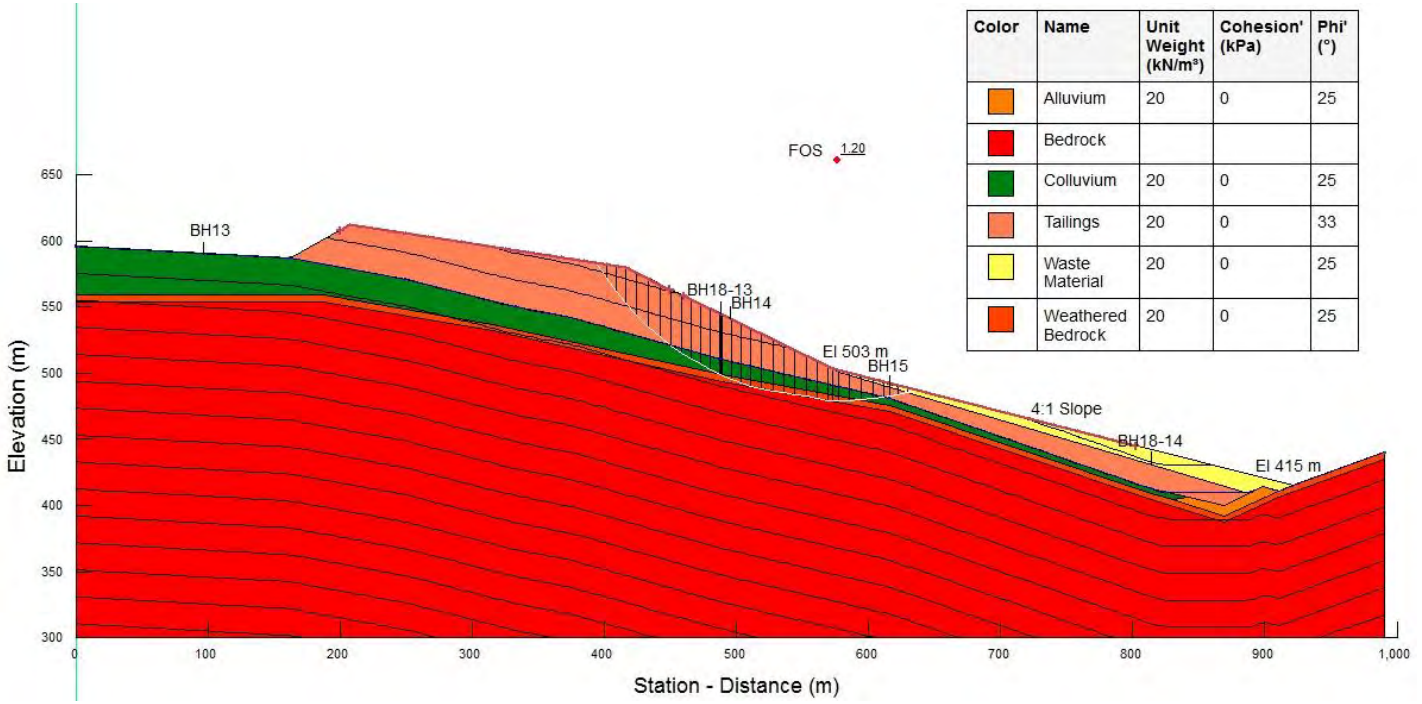
Color	Name	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)
	Alluvium	20	0	25
	Bedrock			
	Colluvium	20	0	25
	Tailings	20	0	33
	Waste Material	20	0	25
	Weathered Bedrock	20	0	25

TS2 (North Lobe)

TS2 – Option WC2: Static Analysis, berm added

- Added Waste Material over base until FoS = 1.20
- Phreatic surface follows the tailings/colluvium boundary
- WM area: 2600 m2
- ~300 m of WM placed from mid-slope to valley

FoS = 1.20

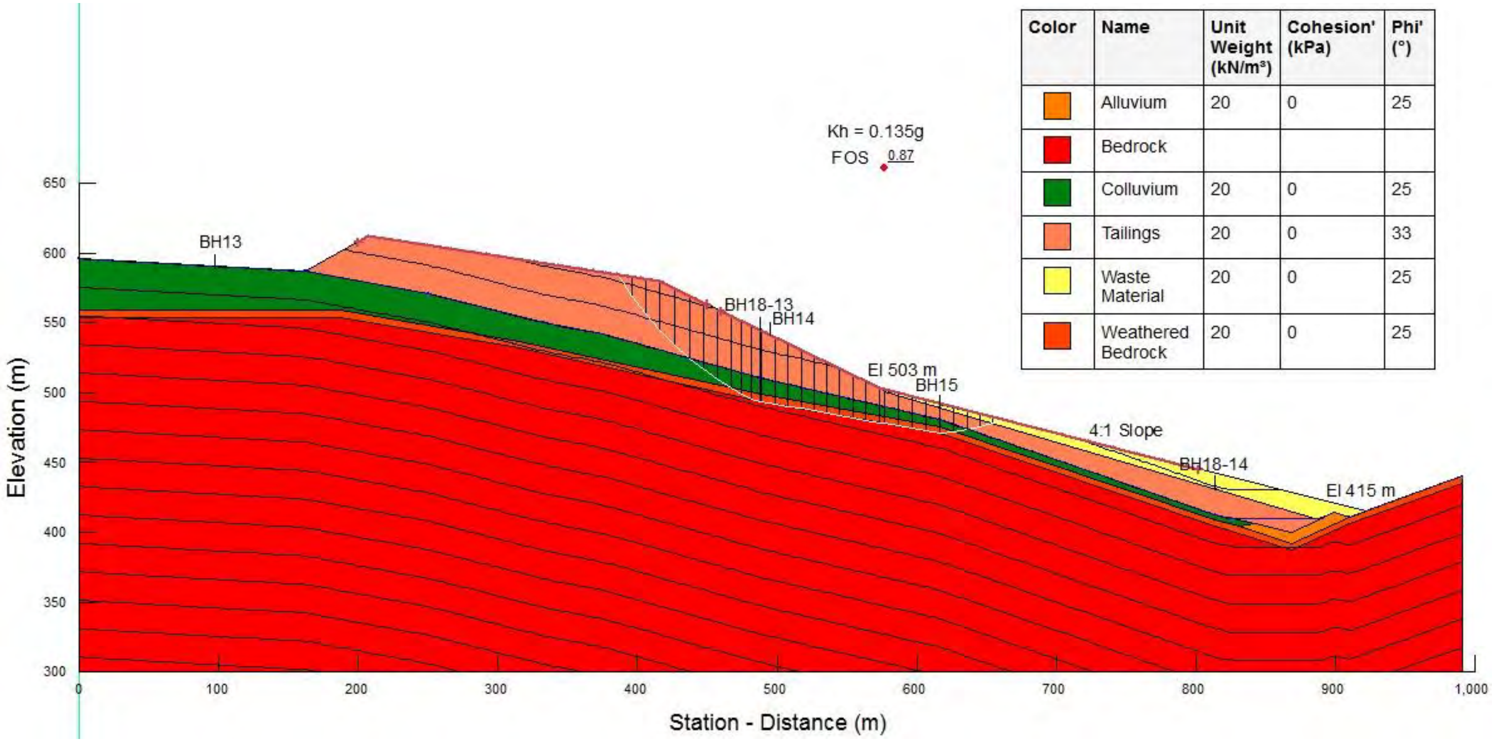


Color	Name	Unit Weight (kN/m^3)	Cohesion' (kPa)	Phi' (°)
	Alluvium	20	0	25
	Bedrock			
	Colluvium	20	0	25
	Tailings	20	0	33
	Waste Material	20	0	25
	Weathered Bedrock	20	0	25

TS2 – Option WC2: Seismic Analysis

- No liquefaction
- Phreatic surface follows tailings/colluvium boundary
- Using Griffen and Franklin (1984) method with Kh = 0.135g (50% of PGA = 0.27)
- FOS = 0.87 less than acceptable for this method.

FoS = 0.87

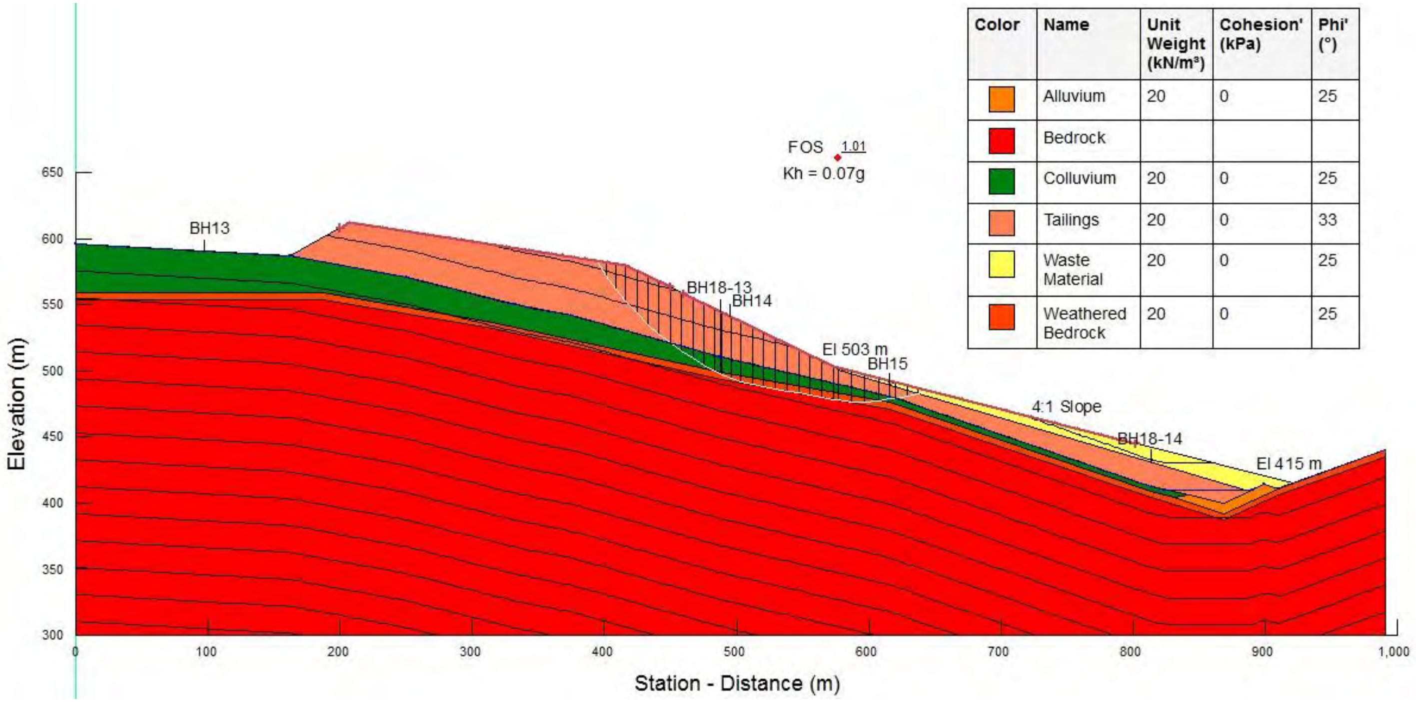


Color	Name	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)
	Alluvium	20	0	25
	Bedrock			
	Colluvium	20	0	25
	Tailings	20	0	33
	Waste Material	20	0	25
	Weathered Bedrock	20	0	25

TS2 – Option WC2: Seismic Analysis, Unity

- Kh adjusted until reaching FoS => 1.0
- Kyield = 0.07g for input into Bray and Tvarasarou (2007) model.
- See memo for movement prediction.

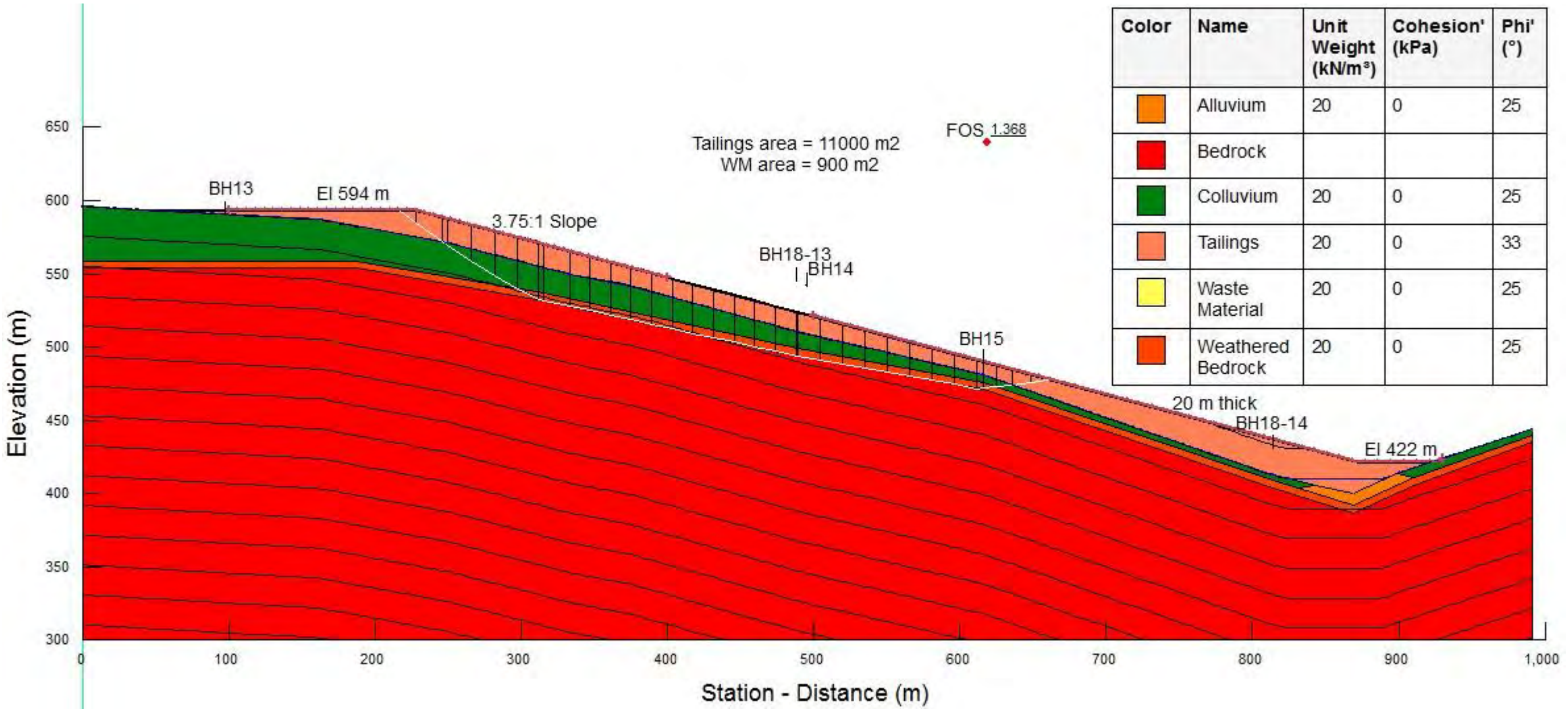
FoS = 1.01



TS2 – LCAA Option E: Static Analysis

- Tailings smoothened out to constant 3.75:1 slope
- 1 m of waste material overlaying tailings
- Toe extended to east slope at El 422 m
- Phreatic surface follows tailings/colluvium boundary
- Tailings area: 11,000 m2

FoS = 1.37

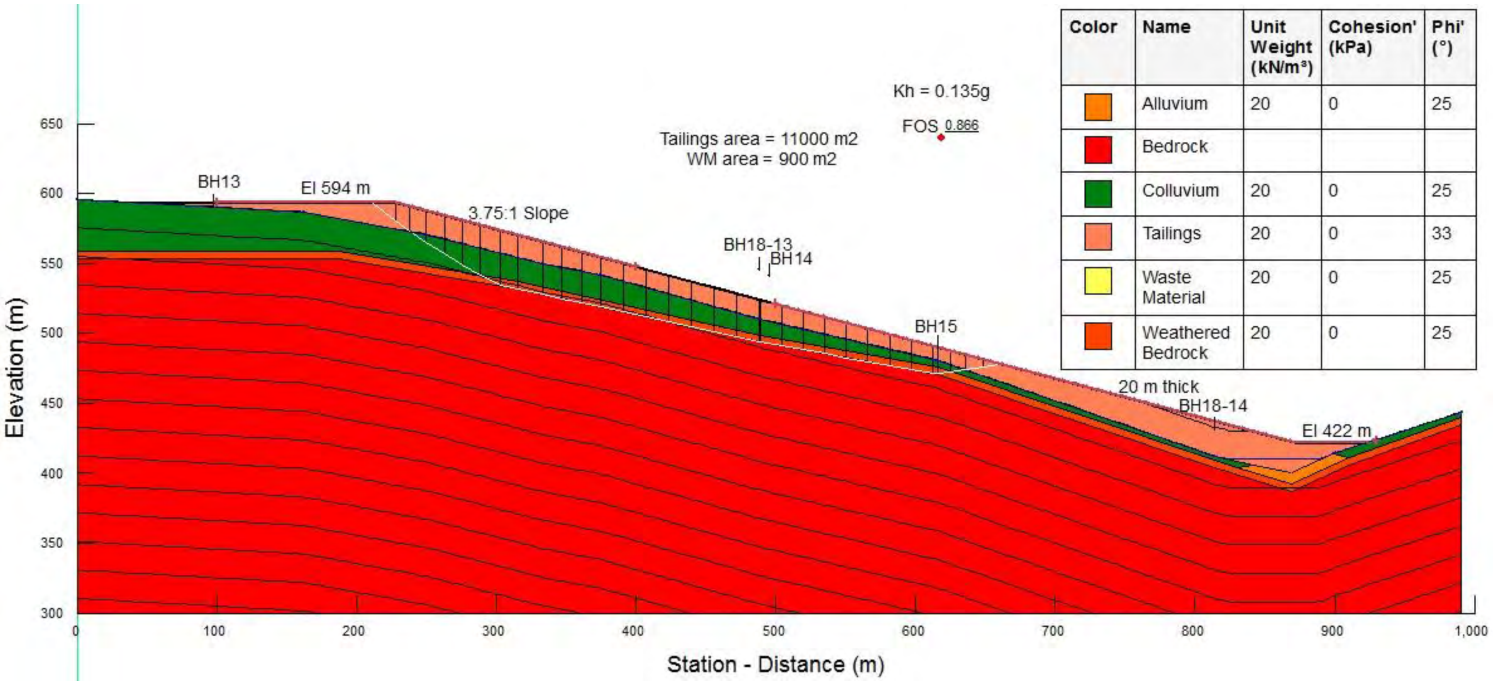


Color	Name	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)
	Alluvium	20	0	25
	Bedrock			
	Colluvium	20	0	25
	Tailings	20	0	33
	Waste Material	20	0	25
	Weathered Bedrock	20	0	25

TS2 – LCAA Option E: Seismic Analysis

- No liquefaction
- Phreatic surface follows tailings/colluvium boundary
- Using Griffen and Franklin (1984) method with Kh = 0.135g (50% of PGA = 0.27)
- Kh = 0.135g results in FoS of 0.87, rounds off to movement less than 1m.

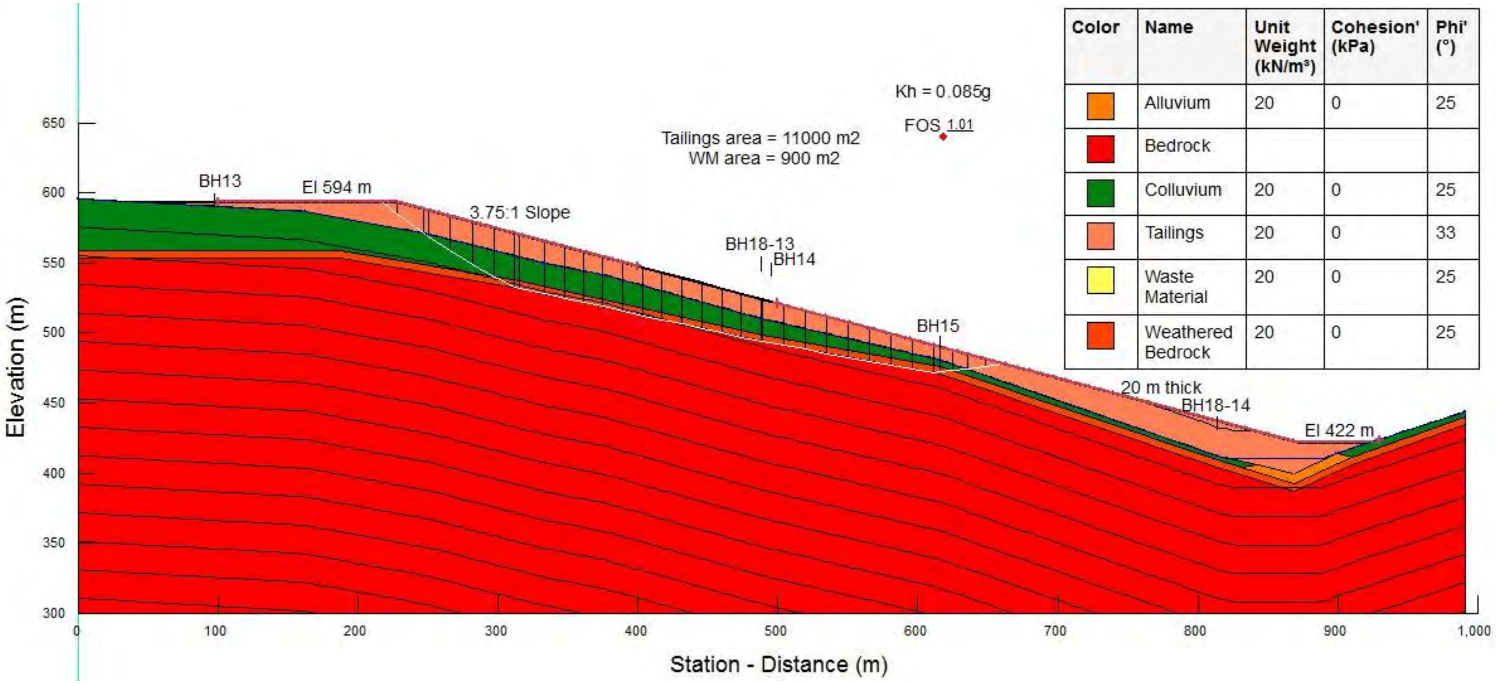
FoS = 0.87



TS2 – LCAA Option E: Seismic Analysis

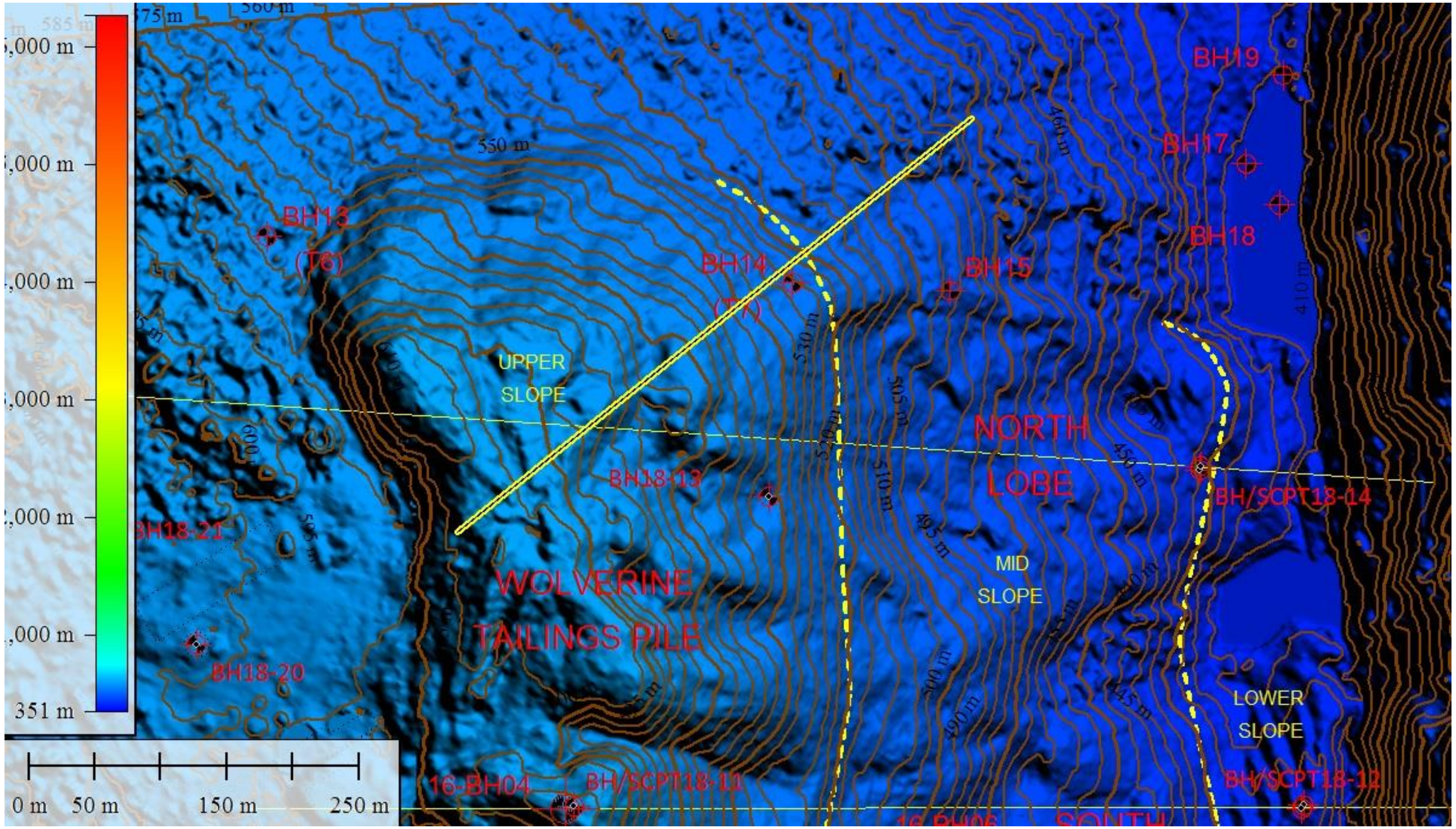
- Kh adjusted until reaching FoS => 1.0
- Kyield = 0.085g
- See memo for movement prediction

FoS = 1.01



Color	Name	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)
	Alluvium	20	0	25
	Bedrock			
	Colluvium	20	0	25
	Tailings	20	0	33
	Waste Material	20	0	25
	Weathered Bedrock	20	0	25

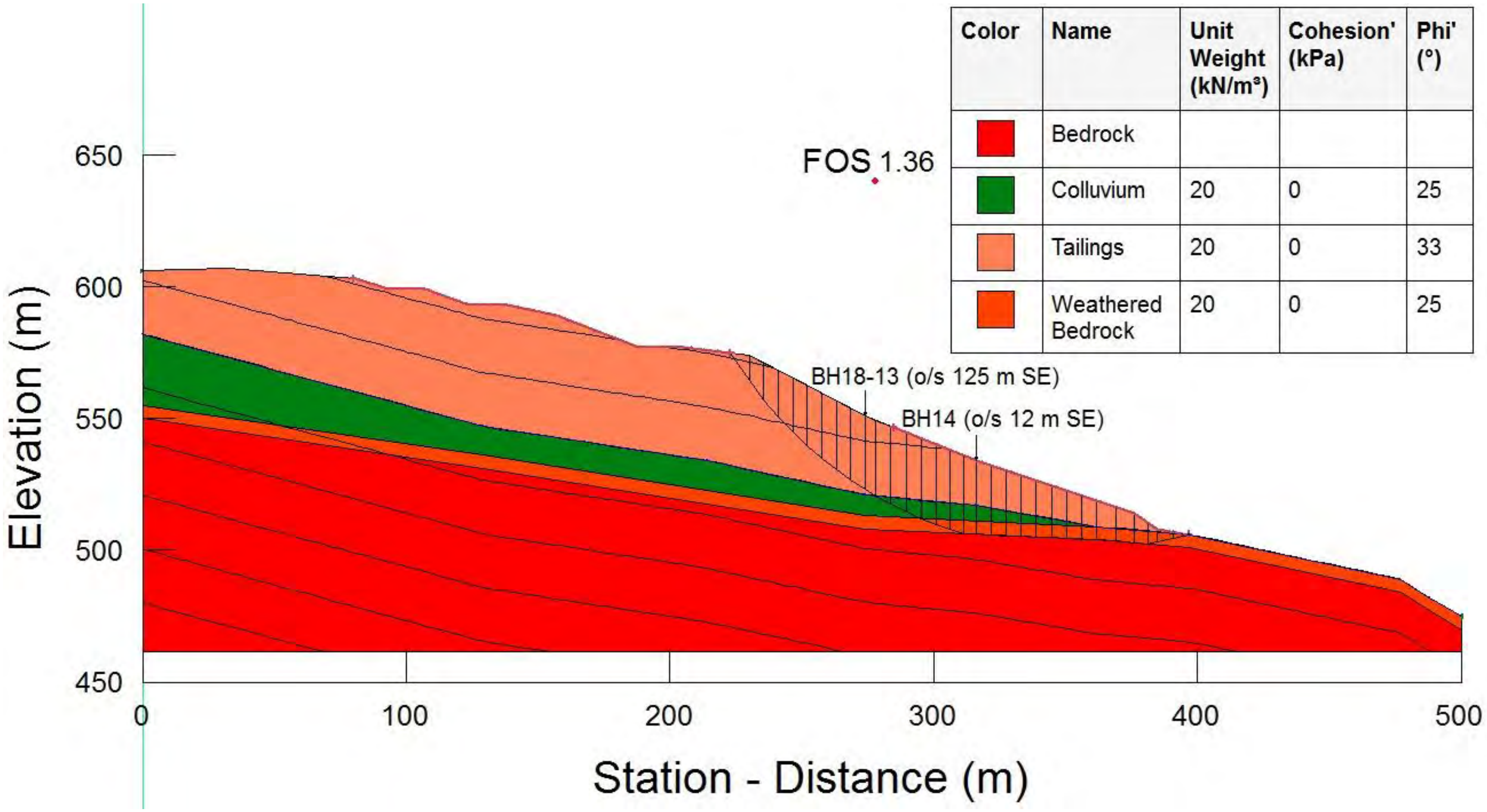
TS3 – Running SW – NE on Upper Slope



TS3 – Option WC2: Static Analysis, no berm added

- No berm added since FoS already > 1.2
- Phreatic surface follows the tailings/colluvium boundary

FoS = 1.36

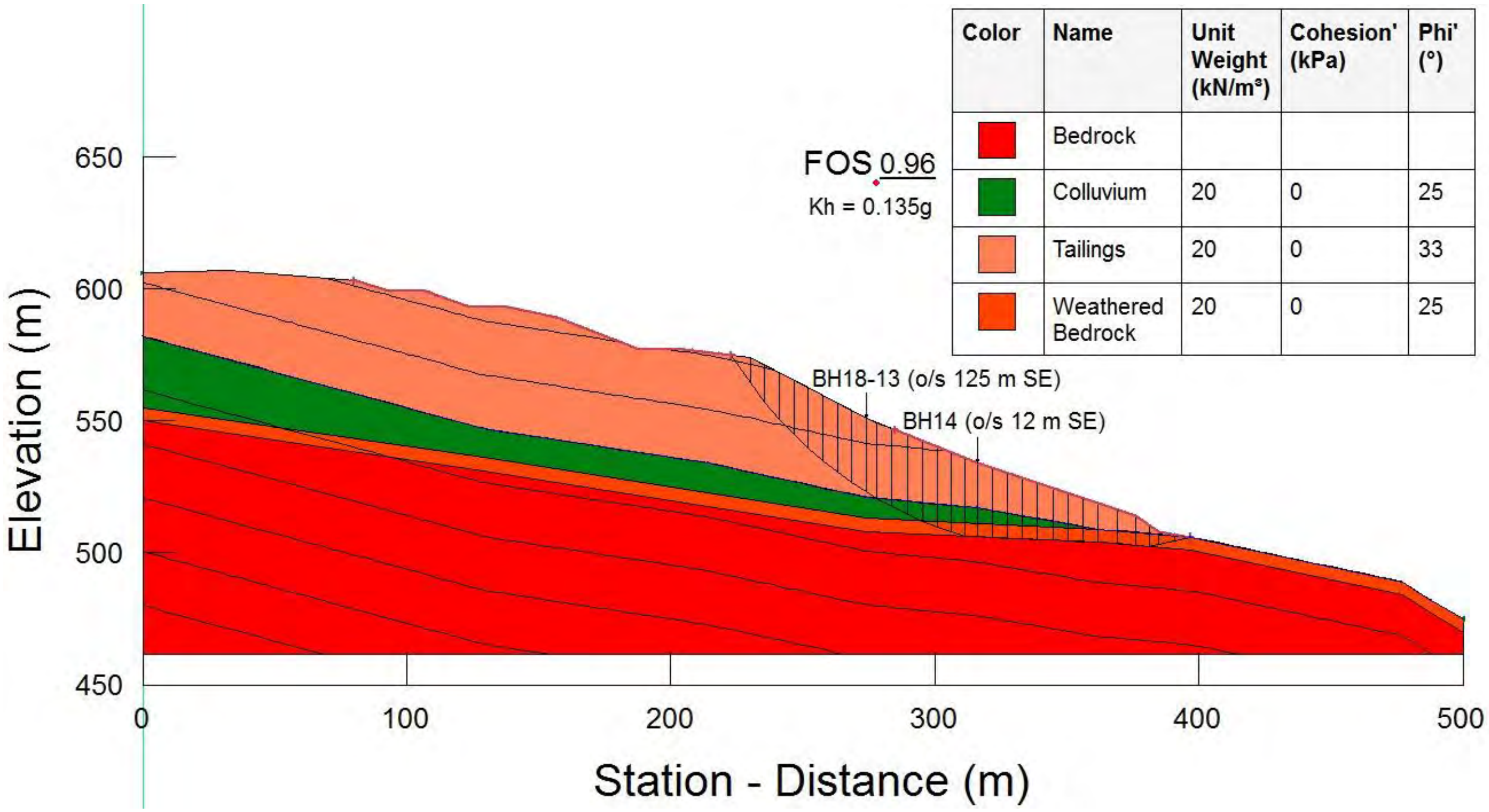


Color	Name	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)
	Bedrock			
	Colluvium	20	0	25
	Tailings	20	0	33
	Weathered Bedrock	20	0	25

TS3 – Option WC2: Seismic Analysis, no berm added

- No liquefaction
- Phreatic surface follows tailings/colluvium boundary
- Using Griffen and Franklin (1984) method with Kh = 0.135g (50% of PGA = 0.27)
- FOS = 0.96 less than acceptable for this method.

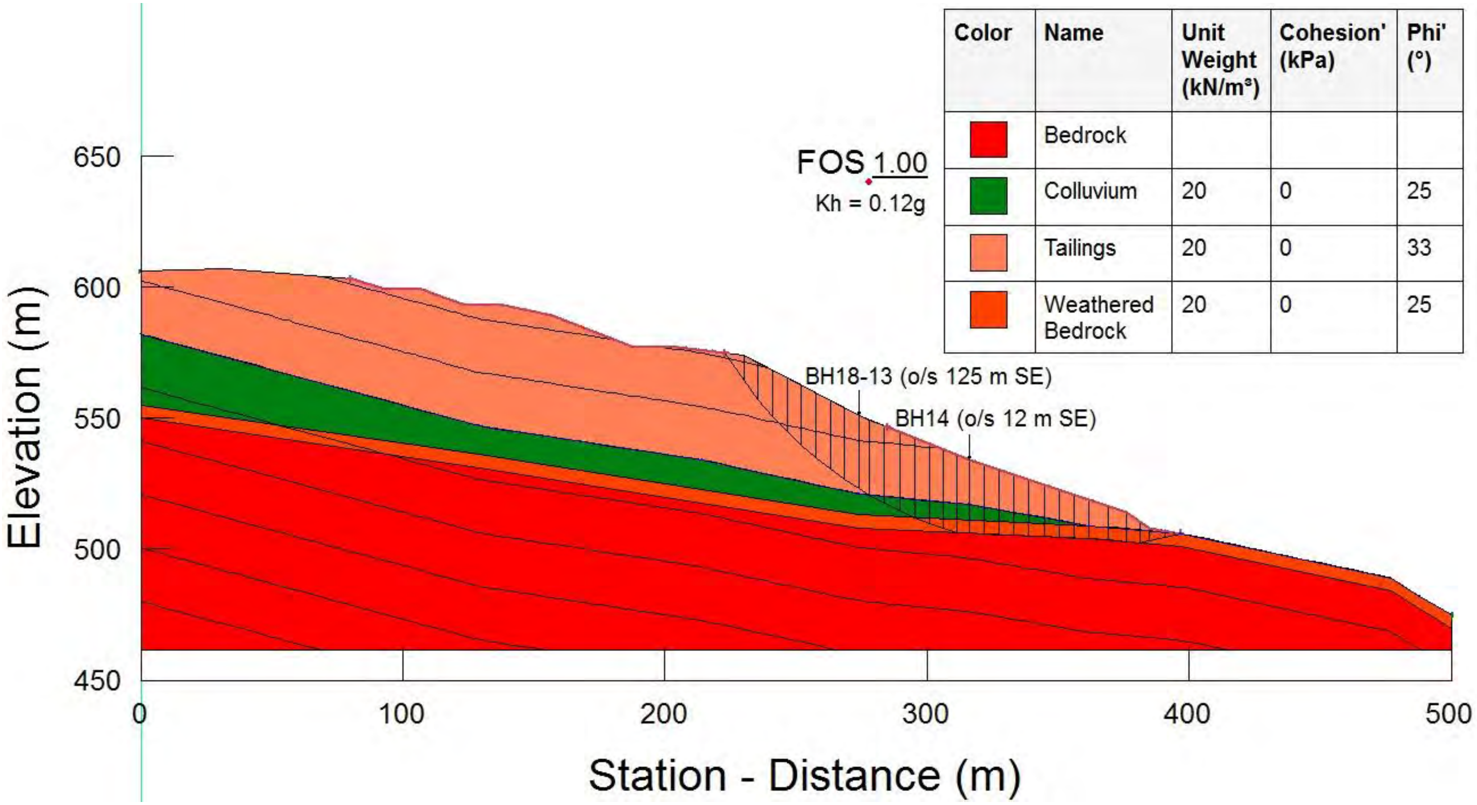
FoS = 0.96



TS2 – Option WC2: Seismic Analysis, Unity

- Kh adjusted until reaching FoS => 1.0
- Kyield = 0.12g for input into Bray and Tvarasarou (2007) model.
- See memo for movement prediction.

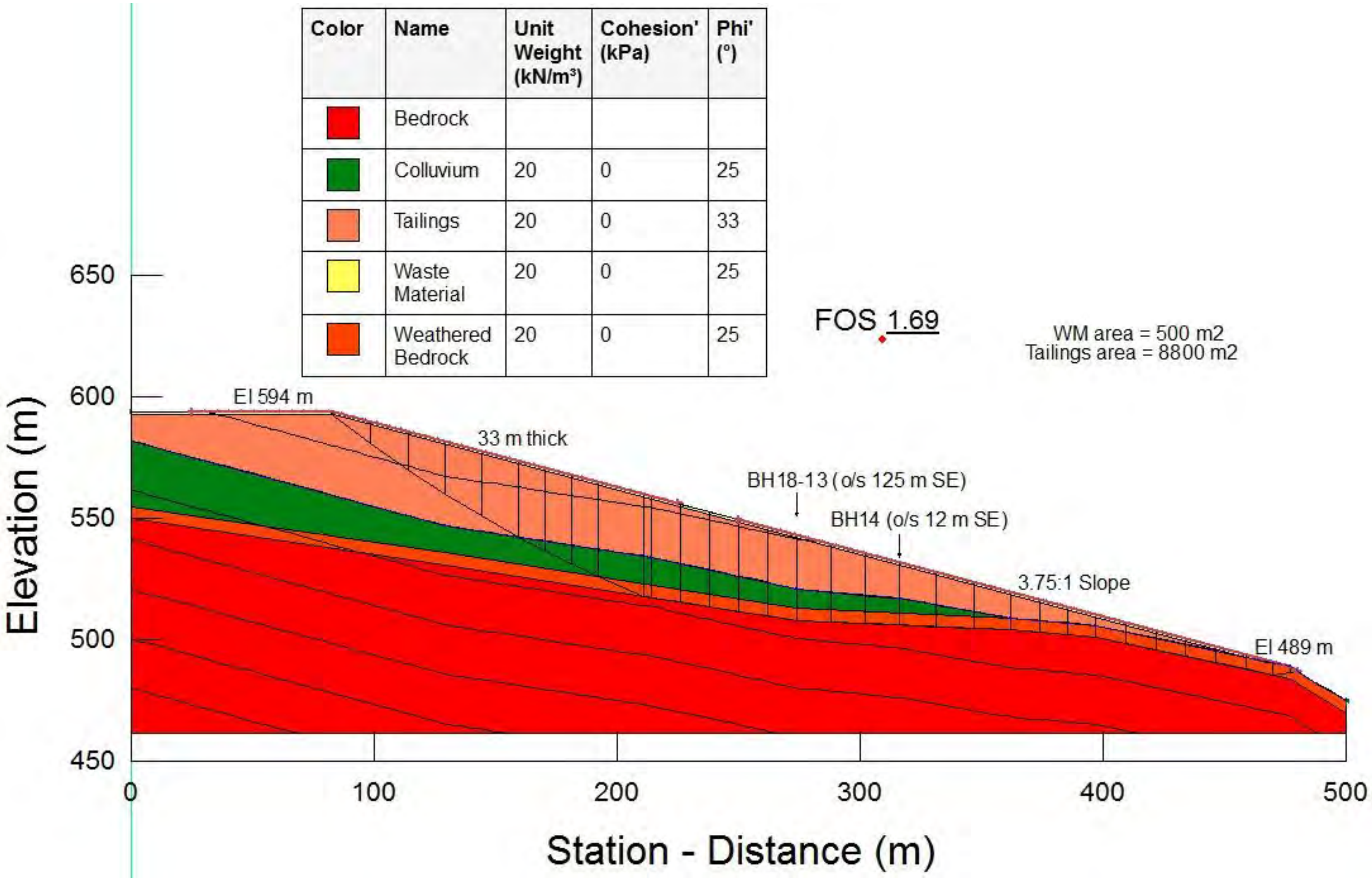
FoS = 1.0



TS3 – LCAA Option E: Static Analysis

- Tailings smoothened out to constant 3.75:1 slope
- 1 m of waste material overlaying tailings
- Crest reaches to El 594 m
- Phreatic surface follows tailings/colluvium boundary
- Tailings area: 8,800 m2

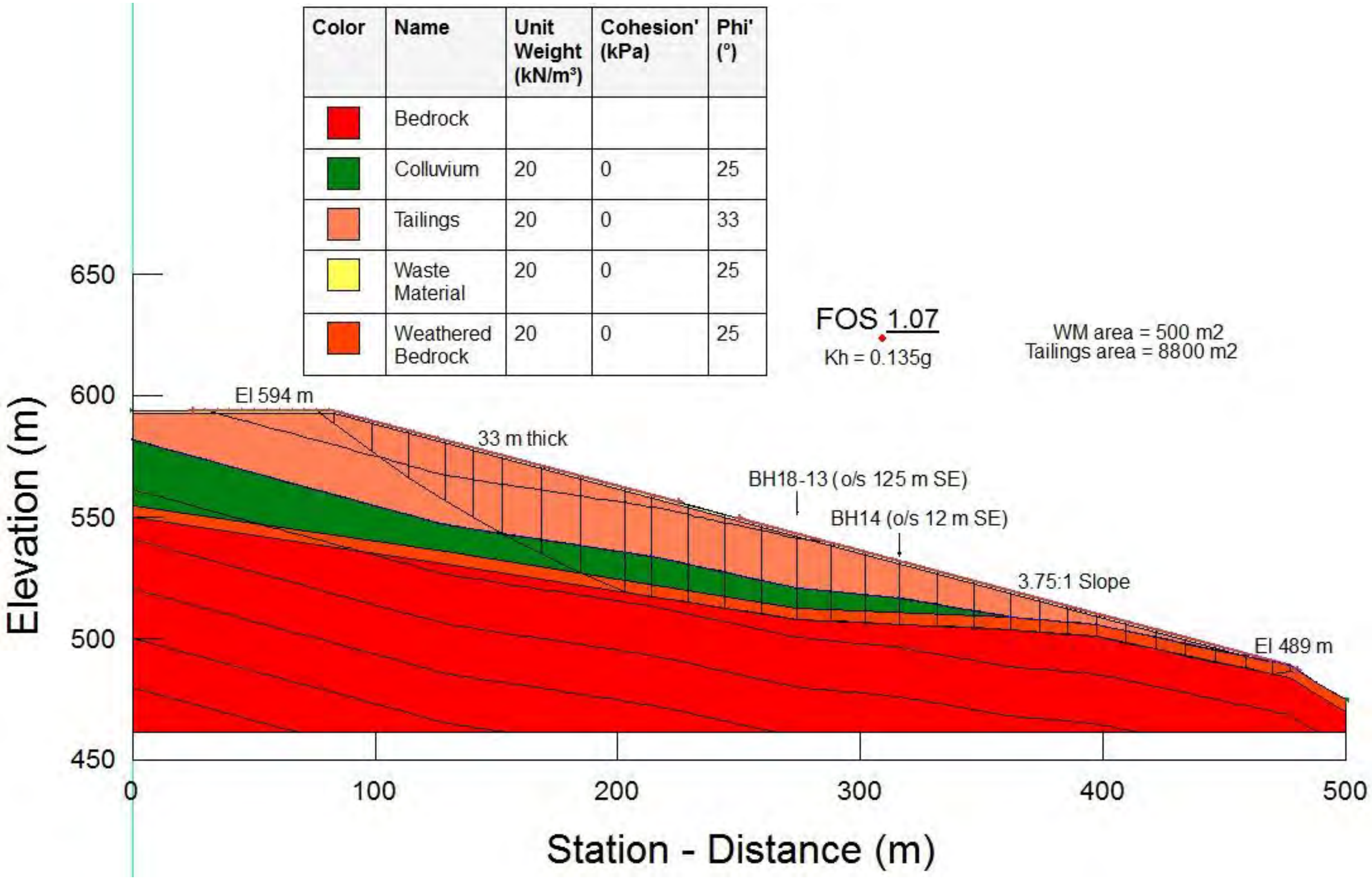
FoS = 1.69



TS3 – LCAA Option E: Seismic Analysis

- No liquefaction
- Phreatic surface follows tailings/colluvium boundary
- Using Griffen and Franklin (1984) method with Kh = 0.135g (50% of PGA = 0.27)
- Kh = 0.135g results in FoS of 1.07, rounds off to movement less than 1m.

FoS = 1.07



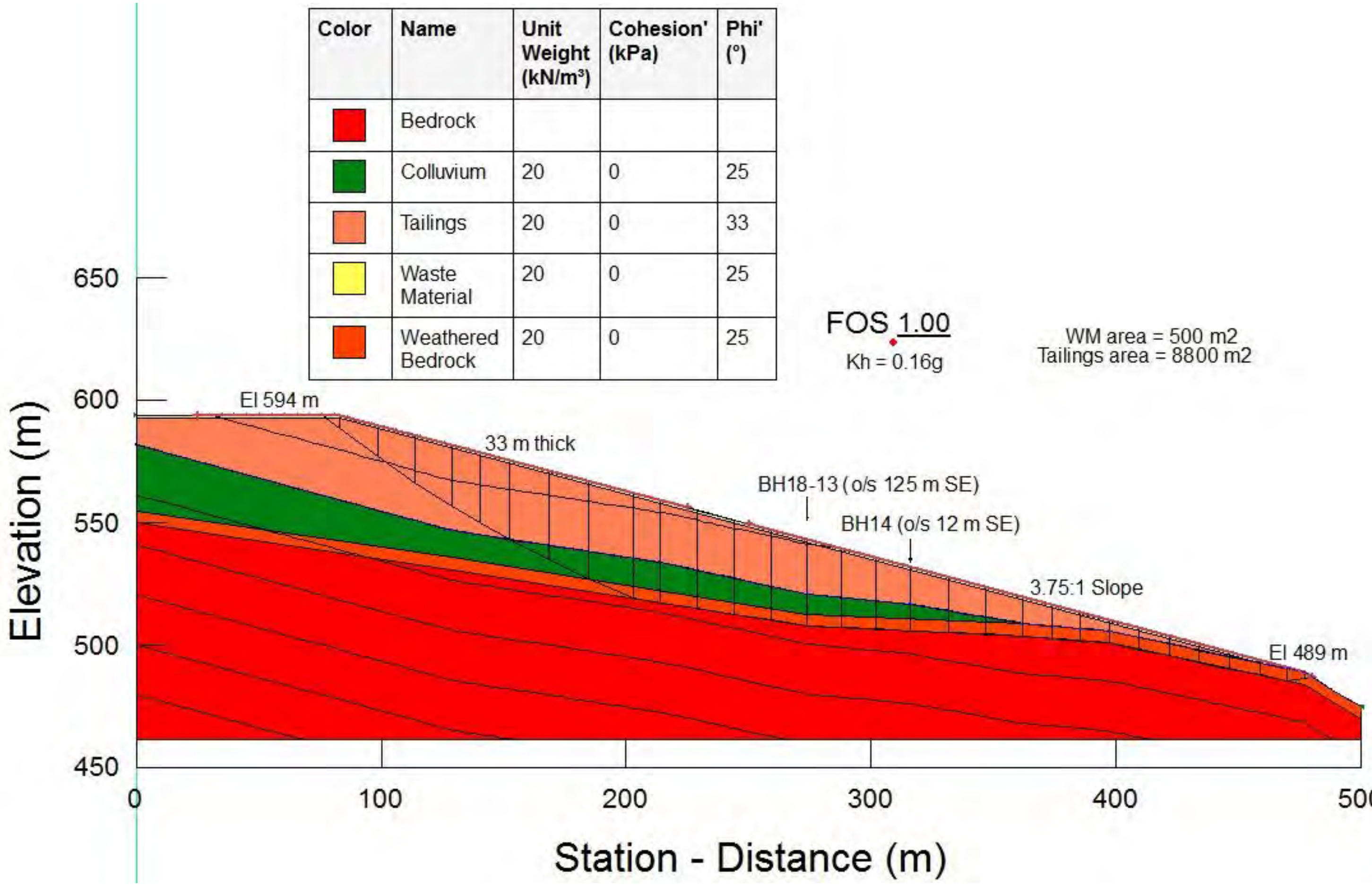
Color	Name	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)
	Bedrock			
	Colluvium	20	0	25
	Tailings	20	0	33
	Waste Material	20	0	25
	Weathered Bedrock	20	0	25

TS2 – LCAA Option E: Seismic Analysis

- Kh adjusted until reaching FoS => 1.0
- Kyield = 0.16g
- See memo for movement prediction

FoS = 1.00

Color	Name	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)
	Bedrock			
	Colluvium	20	0	25
	Tailings	20	0	33
	Waste Material	20	0	25
	Weathered Bedrock	20	0	25



Appendix G
Clinton Creek – Storage and Excavation Volumes

Wood Environment & Infrastructure Solutions,
a Division of Wood Canada Limited
600 – 4445 Lougheed Highway
Burnaby, BC V5C 0E4
Canada
T: 604-294-3811

Memo

To:	File		
From:	Hamid Yousefbeigi / Surinder Garewal	**Reviewer:**	Brian Ross, PEng (AB)
cc:	Ed McRoberts / Karen Hincks	**Wood File No.:**	VE52705D.100.3
Date:	26 August 2019		
Re:	**Clinton Creek – Storage and Excavation Volumes – REV 01**		

1.0 Introduction

This memo provides an overview and methodology used to determine mining and associated waste (waste and tailings) volumes produced during the active mining period (1968-1978) at the abandoned Clinton Creek Mine in the Yukon Territory and estimates available storage volumes in the former pits and other potential areas for site closure concepts. These volumes include:

- An estimate of the available storage in Porcupine Pit, Snowshoe Pit and in the former mill area,
- The volume of material included in the Clinton Creek Waste Dump, Porcupine Creek Waste Dump, Snowshoe Pit Waste Dump, and Wolverine Tailings Dump.
- The required volume of excavation for the three Project Partners Closure Concepts for the Clinton Creek Waste Dump.

Other volumes, such as the volume of water in Hudgeon lake volume and the volume of tailings sediments that were transported and deposited downstream of the tailings area, are not included as part of this memorandum.

2.0 Scope

The scope of work that is summarized in this memo is as follows:

1. Obtain relevant volumes of the pits, waste dumps and tailings from available data, including the Digital Elevation Models (DEMs), and cross check the calculated volumes with those provided in previous studies;
2. Determine the available volumes for waste or tailings storage available in the existing pits and alternative areas based on the DEM and preliminary waste storage designs;
3. Determine volumes of the material that would need to be removed for each of the closure concepts (CC1, CC2, CC3 and WC3) based on available DEM and the preliminary (10%) design concepts;
4. Prepare a memo detailing methodology and procedures. Appendix A includes a selection of applicable figures and sketches.

Volumes of excavation required for option WC2 is presented in a separate memorandum, under the same overall report cover (Appendix H).

3.0 Historic Information

A desktop study was undertaken to find available information on previously reported mine operation volumes and previously assumed or calculated volumes. A summary of the reviewed documents is included in Table 3-1 along with a summary of the estimated volumes provided in reviewed documents. The reported volumes in most of these documents are very similar; however, none of the reports explained how they calculated the reported volumes. It has been assumed that the volumes were estimated based on simple geometry assumptions (ie width of the channel and slopes of the side hills).

Table 3-1: Reference Documents

Reference Document Title	Reference	Reported Volume
UBC Report on Clinton Creek Geology (2016). Page 23	UBC (2016)	Average waste-to-ore ration of about 5.5-1
R14 Former Clinton Creek Asbestos Mine Overview Report - AECOM (2009), Page 13/102 section 3.1 and 3.2 Page 15/102 section 3.3	AECOM (2009)	Total of 12 million tonnes of serpentine ore extracted from three pits. The total volume of waste is estimated to be 60 million tonnes (Roach 1998). The ratio of waste to ore was 4 to 1 About 12 million tonnes of asbestos tailings were deposited over the west slope of the Wolverine Creek valley.
R47 Conceptual Design Report - UMA Engineering (2002) Page 32/101 section 6.2 Reference plan drawings on page 98/101	UMA Engineering Ltd. (2002)	Valley width of 100 m was used to estimate waste excavation of 10,000,000 m^3 to achieve a stable geometry as shown on drawing 5 (2.5:1 slope). Also, volume 0.6 M m^3, and 3 M m^3 of waste removal options were provided to stabilize the Clinton creek waster rock and to provide meandering pattern in this report.
R056 - Lifecycle Cost Anal Remediation Options - Worley Parsons (2014), Page 43/248 section 3.4.6 (No lake option) Reference plan drawing on page 104/248 and 105/248	Worley Parsons (2014)	9.5 M m^3 of waste would be removed to stabilize the dump and expose a 100 m-wide section of the valley floor. Total of 5 M m^3 relocate to Porcupine Pit and 4.5 M m^3 would be placed along the south side of the Hudgeon Lake.
R097 - Clinton Creek Technical Options Assessment - AECOM (2011), Page 5/99 Summary & 64/99 section 8.4.1	AECOM (2011)	Over 60 million tonnes of Clinton Creek Wasterock Dump and over 10 to 12 million tonnes of tailing
R120 - Long-term Monitoring Program - Worley Parsons (2015) – Redu, Page 15/166 section 1.1.2.4	Worley Parsons (2015)	approximately 60 M tonnes of waste and OB deposited over the south slope of the Clinton Creek valley (Clinton Creek Waste Dump) with an additional 3 M tonnes of waste and OB placed to the southeast of Porcupine Pit in the Porcupine Pit waste dump (R01, UMA 2000)
Stepanek and McAlpine paper, Page 1	Stepanek and McAlpine	A total of 13 M tons of ore was milled, producing 1.1 M tones of asbestos fiber and 12 M tones of tailings. Waste about 35 M m^3 and some 7 M m^3 deposited over the slopes adjacent to the open pit and the mill area
Gap Analysis & Site Investigation Plan Rev. 0,	Amec Foster Wheeler (2016)	Waste (60 M tons) was placed along the south valley wall of Clinton Creek while tailings from the milling operation (11 M tons) were placed along the west valley wall of Wolverine Creek.
Field Program Summary Report Rev.0 (See note below) Page 17 & 18/321 section 5.6.1, 5.6.2, 5.6.3	Amec Foster Wheeler (2017)	- Based on the survey boundary of the pile, Porcupine Creek Waste Dump contains an estimated volume of 280,000 m^3 of material. - Snowshoe Pit waste pile volume of approximately 950,000 m^3. - Porcupine Pit storage capacity estimated to be 12 to 13 M m^3. - Snowshoe Pit storage capacity estimated to be 1.1 M m^3. - Creek Pit storage capacity estimated to be 2 M m^3.

4.0 Methodology

4.1 Digital Elevation Models

Three DEM surfaces, a 1949 DEM (with three different vertical adjustments), a 2012 DEM, and an Arctic Digital Elevation Model (ArcticDEM[1]) were used to calculate the volumes reported in this memo. It is understood that a 1949 pre-mining topographic surface was created from aerial images through stereoscopic viewing by TetraTech and provided to Wood (formerly Amec Foster Wheeler) in 2016. Wood was also provided a 2012 Digital Globe DEM and Hudgeon Lake bathymetry data. The ArcticDEM, which is a collaborative project completed in 2001 by the University of Minnesota to produce a 2-meter resolution DEM, was only used to calculate tailings volume in the upper reaches of the Wolverine Creek, described in section 8.2.1 Option E2. Prior to the 2018 site investigation, due to the 1949 DEM not matching well with available 2016 borehole data, a comparison was made between the 2012 DEM, bathymetry data, available borehole information, and the 1949 DEM using Global Mapper. As a result, a -13 m vertical adjustment to the 1949 DEM was adopted for 2018 site investigation planning purposes and preliminary design use.

Subsequently, and in reviewing the results of the 2018 Site Investigation Program, a vertical adjustment to the 1949 DEM of -20 m was adopted for the Clinton Creek valley, in particular along the valley bottom. A vertical adjustment of -25 m was adopted for the upper portion of the Wolverine Creek valley, while the previous adjustment of -13 m was retained for the lower portion of the Wolverine Creek valley, for design and volume calculations. The -20 m vertical adjustment was used for the Porcupine Pit analyses. It should be noted that none of the adjustments appear to be completely reliable. The -13 m original adjustment matched well with the boreholes in the valley bottom of Wolverine Creek and with select boreholes in the Clinton Creek valley but did not have a good fit along the valley walls of Wolverine Creek or along the north valley wall of Clinton Creek.

4.2 Software Program

The methodology used in modeling and calculating volumes in this study was the adoption of civil design software, RoadEng, developed by Softree. RoadEng can produce 3D modeling and site design, road and corridor design, and corridor optimization. The focus of this program is mainly on engineering

4.3 Calibration and checks

A comparison exercise was conducted between several software programs to check the accuracy of the obtained volumes and quantities from the RoadEng program. Comparisons were made between various software programs including Global Mapper, Eagle Point, RoadEng, and Civil 3D. Table 4-1 presents the results of the comparisons from the different software programs.

Table 4-1: Comparison of Volumes Obtained from Different Software.

Software Name	Calculated Volume (m^3)	Difference (%) VS. RoadEng
Eagle Point	11,555,410	-0.0001%
Global Mapper	11,477,452	0.6746%
RoadEng	11,555,403	0.0000%
Civil 3D CAD	11,555,741	-0.0029%

[1] ArcticDEM is a National Geospatial-Intelligence Agency (NGA) and National Science Foundation (NSF) public-private initiative to automatically produce a high-resolution, high-quality digital surface model (DSM) of the Arctic using optical stereo imagery, high-performance computing, and open source photogrammetry software. The product is a collection of time-dependent DEM strips and a seamless terrain mosaic that can be distributed without restriction.

A cross-section average end area calculation was also performed to compare the resulting volume produced on one of the Clinton Creek closure concept options. The difference between the two calculated volumes was 0.14%. Finally, a back of the envelope calculation check was conducted by a senior engineer to provide an additional level of comfort against the software calculated volumes.

5.0 Available Storage Volumes in Existing Pits

5.1 Porcupine Pit Storage

The estimated volume of material removed from the Porcupine Pit was determined by outlining the pit using a poly line on the 2012 DEM, with the aid of the available aerial imagery. This poly line was used to calculate the volume of material removed from the pit by comparing the 1949 -20 m DEM adjustment surface and the 2012 DEM surface. The estimated volume of material removed from the Porcupine Pit is estimated to be about 27,386,000 Bench Cubic Meter (BCM).

According to the 2012 DEM data, the elevation of the standing water in the pit is approximately 375 m and the elevation of the lowest portions of the pit rim is at 418 m. The volume of material that can be placed between the water surface and the lowest point of the pit rim is 2,585,000 m^3, as presented in Figure 5.1. A storage elevation curve for the Porcupine Pit was then produced, based on a partial shell constructed with an overall 3H:1V slope extending from the pit rim (Figure 5.2 & Figure 5.3). The toe of this shell is assumed to be founded on the original ground at the lowest portions of the pit rim at the north end of the Porcupine Pit. The overall storage capacity of the Porcupine Pit, not including volumes below the existing water level (375 m El), is 23.9 M m^3 based on a 3H:1V constructed shell shown in Figure 5.2 and shown in the Storage Elevation Curve presented in Figure 5.3.

The current model indicates that it is possible to construct the final Porcupine Pit backfill to about 600 m elevation. However, an ultimate elevation of 570 m was considered for this analysis, since the construction of the last 30 m (ie from 570 m to 600 m El) would result in a small volume increase with less economical construction. To support the design slope of 3H:1V, a shell wherever the backfill would not be confined by the pit walls is assumed to be required. The shell has been assumed to be a 50 m wide (measured on the horizontal plane) zone of compacted waste.

The first stage of the construction of the shell would have to be completed against the north valley wall of the pit, from the bottom of the pit to the shortest pit rim elevation at 418 m. The slope of this portion of the shell is designed at 2H:1V. The volume of this initial shell construction is estimated to be 395,000 m^3 and is shown in Figure 5.4. The total volume of a 50 m wide compacted shell at Porcupine Pit is estimated to be about 4,733,000 $^{m3.}$ This volume, which includes the in pit portion, is calculated based on neat line quantities and configuration presented in Figure 5.5. It should be noted that the actual construction of the shell would be based on an upstream raise construction methodology, which would require additional volume compared with the neat line quantities presented in this memo.

5.2 Snowshoe Pit and Creek Pit Storage

The current Snowshoe Pit volume was calculated by outlining the perimeter using a poly line on the 2012 DEM with help from the available aerial imagery. This poly line was used to compare the pit in-situ volume by using the 1949 -20 m DEM adjustment surface and the 2012 DEM surface. The total excavated volume at Snowshoe Pit and Creek Pit was estimated to be about 1,914,040 Bench Cubic Meter (BCM).

The geometry of the Snowshoe Pit is very non-typical, and the Snowshoe Pit does not have a closed pit rim as it has been cut at the side of the hill. There are not many options for backfilling this pit at the proposed 3H:1V slopes. The overall capacity of the Snowshoe Pit for excavated material in any of the closure concepts is estimated to be 806,000 m^3, based on the configuration shown in Figure 5.6 and Figure 5.7.

The available storage volume in Creek Pit has not been calculated at this stage as there is no information about the geometry of the pit below the current water surface.

6.0 Waste and Tailings Volume Calculations

6.1 Waste Dumps

Individual waste dumps in Clinton Creek, Porcupine Creek and around the Snowshoe Pit were outlined with a poly line. Then the in-situ volume between the 2012 DEM and the 1949 DEM -20 m adjustment was calculated. However, the 1949 extent does not cover the entire area occupied by the waste dump in Porcupine Creek and at the east side of the Snowshoe Pit, as shown in Figure 6.1. The red and green dashed lines in Figure 6.1 indicate the extent of the 1949 and 2012 DEM surfaces, respectively. Since the volume calculations were based on comparison between the two DEM surfaces, some of the waste dump at the Porcupine Creek and Snowshoe Pit Waste Dumps had to be accounted for using different methods.

The missing portion of the Porcupine Creek Waste Dump area outside of the 1949 DEM was reconstructed using surrounding 2012 DEM contours and available borehole information, so the volume could be calculated. The volume of the Porcupine Creek Waste Dump in the overlap area of both DEMs was about 1,219,000 m^3 whereas the volume of the waste outside of the overlapped zone was estimated to be about 1,829,000 m^3. Adding the two, the total volume of the Porcupine Creek Waste Dump is estimated to be 3,050,000 m^3. The two area east of Snowshoe Pit was simply estimated (too small) and added to the total volumes.

A 3D model of the current Clinton Creek topography (based on the 2012 DEM) showing the pits in green (with the 1949 DEM -20 m adjustment contours shown) and the waste dumps in red is presented in Figure 6.2. The total estimated volume of material removed from the pits (Porcupine, Snowshoe and Creek Pits shown in green) is 29,300,000 Bench Cubic Meter (BCM), while the total estimated volume of material in the waste dumps (Clinton Creek, Porcupine Creek and Snowshoe Pit) is 25,205,000 Compacted Cubic Meter (CCM)

6.2 Tailings

As previously identified, the 1949 DEM was not completely reliable and various adjustments were made to best fit the 1949 DEM to available data. The 1949 DEM data had two different adjustments applied to an area of approximately 700 m by 700 m in the Wolverine Creek valley, due to the topographic difference along the Wolverine Creek slopes. The -13 m adjustment matched well with the boreholes in the valley bottom of Wolverine Creek but did not have a good fit along the valley walls, while the -25 m adjustment fit well with the boreholes drilled along the upper and mid-slopes. As a result, it is anticipated that the -13 m DEM adjustment would underpredict the volume of tailings while the -25 m DEM adjustment would over predict the volume of tailings.

A plan view of the tailings area is presented in Figure 6.3, while a 3-D model of the placed tailings is presented in Figure 6.4. Cross-sections of the north and south lobes of the Wolverine Tailings Dump are presented in Figure 6.5 and Figure 6.6, respectively. Using the -13 m DEM adjustment compared to the 2012 DEM resulted in a calculated volume of tailings of 5,293,000 m^3 and using the -25 m DEM adjustment compared to the 2012 DEM resulted in a calculated volume of 10,082,000 m^3 tailings. If the two values are averaged, the resulting volume would be 7,688,000 m^3.

7.0 Mass Balance

Table 7-1 presents a comparison of the calculated volumes of the total estimated volume of material removed from the pits (Porcupine, Snowshoe and Creek Pits), as reported in Section 5. The calculated volume between the two DEM surfaces indicated that a total of 29,300,016 Banked Cubic Meter (BCM) of overburden, waste, and ore was excavated from the three pits. This volume was then multiplied by a 25% bulking factor, which resulted in an estimated volume of all materials of 36,625,020 Loose Cubic Meter (LCM). Though the waste was end dumped with no compaction, it is assumed that the material properties have been changed due to stresses within the waste dumps, weathering, chemical changes, previous failures, and other types of degradation over the years since mine closure. It has been assumed that this degradation has lowered the void ratio resulting in a minimal "compaction" over time. As a result, a compaction factor of 0.9 has been assumed in the calculations presented in Table 7-1.

Table 7-1: Estimated Volume of Total Material Excavated from Pits

Material	Factor	Quantity	Units
Porcupine Pit (excavated volume)	-	27,385,976	m^3
Snowshoe & Creek Pits (excavated volume)	-	1,914,040	m^3
Total Excavated Volume	-	29,300,016	m^3
Estimated Total Volume with bulking	1.25	36,625,020	m^3
Estimated Total Volume after 40 years of effective compaction	0.90	32,962,518	m^3
Total Estimated Volume from Pits		**32,962,518**	**m^3**

The volume of material contained in the waste dumps and the tailings dump was calculated in Section 6 and are summarized in Table 7-2. In general, a total of 32,893,000 m^3 (25,205,000 m^3 waste and 7,688,000 m^3 tailings) of material is remaining on site.

Table 7-2: Calculated Estimated Volumes of Waste, Tailings and Produced Asbestos

Material	Quantity	Units
Waste (Clinton Creek, Porcupine Creek and Snowshoe Pit Dumps)	25,205,000	m^3
Tailings (Wolverine Tailings Dump)	7,688,000	m^3
Asbestos[1]	940,000	Tones
Asbestos[1]	371,542	m^3
Total Waste, Tailings and Produced Asbestos	**33,264,542**	**m^3**

Note(s)

1. Approximately 940,000 tons of white asbestos (known as chrysotile) was removed from three pits at the mine sit (Source: http://www.emr.gov.yk.ca/aam/clinton_creek.html). A unit weight of 2530 kg/m^3 used for conversion.

The total difference between the estimated total volume of mined material from Table 7-1 is approximately 302,024 m^3 less than the total estimated volume of waste (plus produced asbestos) presented in Table 7-2.

8.0 Calculated Material Removal Volumes for Proposed Closure Concepts

The Project Partners provided Wood with three closure concepts for the Clinton Creek Waste Dump and three closure concepts for the Wolverine Tailings Pile. All three of the closure concepts for the Clinton Creek Waste Dump require excavation, removal and subsequent storage of the waste material, while only two of the three closure concepts for the Wolverine Tailings Dump requires excavation, removal and subsequent storage of the tailings; however, the option for WC2 is addressed in a separate memorandum. Additional details on the specific closure concepts is provided in Wood's Design Basis Memorandum. The following sections provide the required excavation volumes for each of the closure concepts requiring excavation with the exception of WC2.

8.1 Clinton Creek Waste Dump

The Clinton Creek Waste Dump currently contains approximately 21,657,000 m^3 of waste, as calculated by the difference between the 1949 and 2012 DEM surfaces, shown in Figure 8.1. All three closure concepts require the channel to be widened and the slopes in the waste to be cut back, to avoid a catastrophic failure of the dumps that could block the new channel. The width of the modified channel was determined by others, and slope stability analysis was undertaken (and presented in a separate memo) in order to determine stable slopes. The design creek channel is 32 m wide with side slopes of 6.5H:1V. For all three options, the slope into Hudgeon Lake will be cut back at 6H:1V, starting from the Hudgeon Lake water line contact. For all three options, the Clinton Creek alignment will be kept on the north side of the pre-construction Clinton Creek valley, without creating cuts in the north slopes (the north slopes will likely require additional buttressing, but for this assessment are to remain undisturbed).

8.1.1 Clinton Creek Closure Concept 1 – CC1

CC1, as presented by the Project Partners, requires sufficient work on the waste pile to mitigate a catastrophic failure of the pile and construct a water conveyance channel to provide water passage from Hudgeon Lake to Clinton Creek. This option would keep Hudgeon Lake at the current elevation of 412 m and requires widening of the channel and cutting back of the waste slopes to mitigate a catastrophic failure. The design channel kept close to the current profile (detailed hydrotechnical work on the water passage/energy dissipation structures presented under separate cover) and alignment.

A plan view showing the proposed cuts into Hudgeon Lake and along the Clinton Creek channel alignment are presented in Figure 8.2, and a 3-D view of the completed slopes is presented in Figure 8.3. The total volume of cut required for CC1 is 4,822,000 m^3. In this option, a total of 16,835,000 m^3 waste remains in the Clinton Creek valley.

8.1.2 Clinton Creek Closure Concept 2 - CC2

CC2, as presented by the Project Partners, requires sufficient work on the waste pile to mitigate a catastrophic failure, construct a water conveyance channel to provide water passage from Hudgeon Lake to Clinton Creek and lower Hudgeon Lake. Based on work completed by Wood's hydrotechnical team and reported elsewhere, in this option the lake will be lowered by approximately 10 m. The alignment of this option will be shifted to the south, in order to avoid any cuts in the already oversteepened north slope of the Clinton Creek valley, and the vertical profile will be flattened to approximately 3% overall, with detailed design of the vertical profile being completed by others. For the purpose of calculating volumes, the waste cut slope near the existing south slopes have been stepped at 2H:1V to gradually transition into the existing south slopes.

A 3-D model of the completed slope cuts is presented in Figure 8.4. The total volume of cut required for CC2 is 7,667,000 m^3. A total of 13,990,000 m^3 of waste will remain in the Clinton Creek valley with this option.

8.1.3 Clinton Creek Closure Concept 3 – CC3

CC3, as presented by the Project Partners, requires sufficient work on the waste pile to prevent it from acting as a dam on Clinton Creek, to mitigate a catastrophic failure of the waste pile, and construct a water conveyance channel to provide water passage through the site. This option requires complete removal of Hudgeon Lake. The design of this option requires the channel to be realigned to the valley bottom of the pre-construction Clinton Creek valley. For the purpose of calculating volumes, the waste cut slope near the existing south slopes have been stepped at 2H:1V to gradually transition into the existing south slopes.

A 3-D model of the completed slope cuts is presented in Figure 8.5. The total volume of waste removal required to complete CC3 is 13,435,000 m^3. It should be noted that a total of 8,222,000 m^3 of waste will remain in the Clinton Creek valley. This majority of this remaining volume is mostly located at the downstream portions of the Clinton Creek.

8.2 Tailings

The Project Partners have presented three closure concepts for the Wolverine Tailings Dump. Concept WC1 does not involve disturbing the tailings in any way, and considerations for WC2 are presented in a separate memorandum (appendix H), while WC3 involves removing the tailings. Wood has identified two locations where the tailings may be relocated to, the first option is to relocate the tailings to the Porcupine Pit, and the second option would be to build a containment and place tailings in the old mill site.

8.2.1 Relocation to Porcupine Pit

In this option, all tailings will be relocated to the Porcupine Pit. The calculated volume of the tailings, which was estimated to be approximately 7,688,000 m^3, as described in Section 6.2.

8.2.2 Relocation to Plant area

For this option, relocation of the tailings to the relatively flat and previously cleared former mill site was considered. The area of consideration was limited to the area that was formerly cleared, as determined from historical air photos, and used a minimum setback of 50 m from the slope breaks of Clinton and Wolverine Creek valleys. A proposed slope of 4H:1V up to 25 to 30 m high was used to determine the potential storage volume at this location, as shown in Figure 8.6. The total volume available in this area, given the stated assumptions is estimated to be 1,465,000 m^3, which is not enough for the estimated volume of 7,688,000 m^3 of tailings requiring relocation.

9.0 Summary and Conclusions

Table 9-1 presents a summary of the estimated required volume of material that would need to be removed for each of the Clinton Creek Waste Dump options CC1, CC2 and CC3 and the Wolverine Tailings Dump Option WC3. A 10% bulking factor has been applied to the excavated volumes, as a sensitivity; it is not expected that the excavated wastes will bulk.

Table 9-2 presents the volumes of storage available in the Porcupine Pit, Snowshoe Pit and in the former mill site location. Upon observation of these quantities there is adequate storage space for the materials to be excavated and removed from their present locations, even with consideration of bulking.

Table 9-1 : Estimated Volume of Required Excavation for Closure Concepts

Option	Quantity Excavated	Quantity Excavated (bulked 10%)	Quantity Remaining In-place	Units
CC1	4,822,000	5,304,200	16,835,000	m^3
CC 2	7,667,000	8,433,700	13,990,000	m^3
CC 3	13,435,000	14,778,500	8,222,000	m^3
WC3	7,688,000	8,456,800	-	m^3
Max Total (CC 3 + WC 3)	**21,123,000**	**23,235,300**	-	$\mathbf{m^3}$
Min Total (CC 1)	**4,822,000**	**5,304,200**	-	$\mathbf{m^3}$

Table 9-2: Estimated Volume of Storage for Excavated Materials on Site

Option	Quantity	Units
Porcupine Pit	23,900,000	m^3
Snowshoe Pit	806,000	m^3
Mill Site	1,465,000	m^3
Total	**26,171,000**	$\mathbf{m^3}$

It should be noted that if WC3 is selected, and that the tailings will be placed into Porcupine Pit, the tailings should be removed and placed in the pit first. This, however, will require staging, as the required 50-m wide shell of compacted waste should be placed along the north wall of the existing pit, with the tailings placed in behind the shell. However, the available volume up to elevation 418 m (ie the current pit lip) is 2,585,000 m^3 minus the 395,000 m^3 of compacted shell required. Tailings will extend above the current pit lip at elevation 418 m, so staged construction of the shell with placement of tailings in behind the shell, will be required until the total volume of tailings has been removed, then waste should be placed above the tailings, in order to encapsulate them.

Sincerely,

Wood Environment & Infrastructure Solutions
a Division of Wood Canada Limited

Prepared by:

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August 28, 2019

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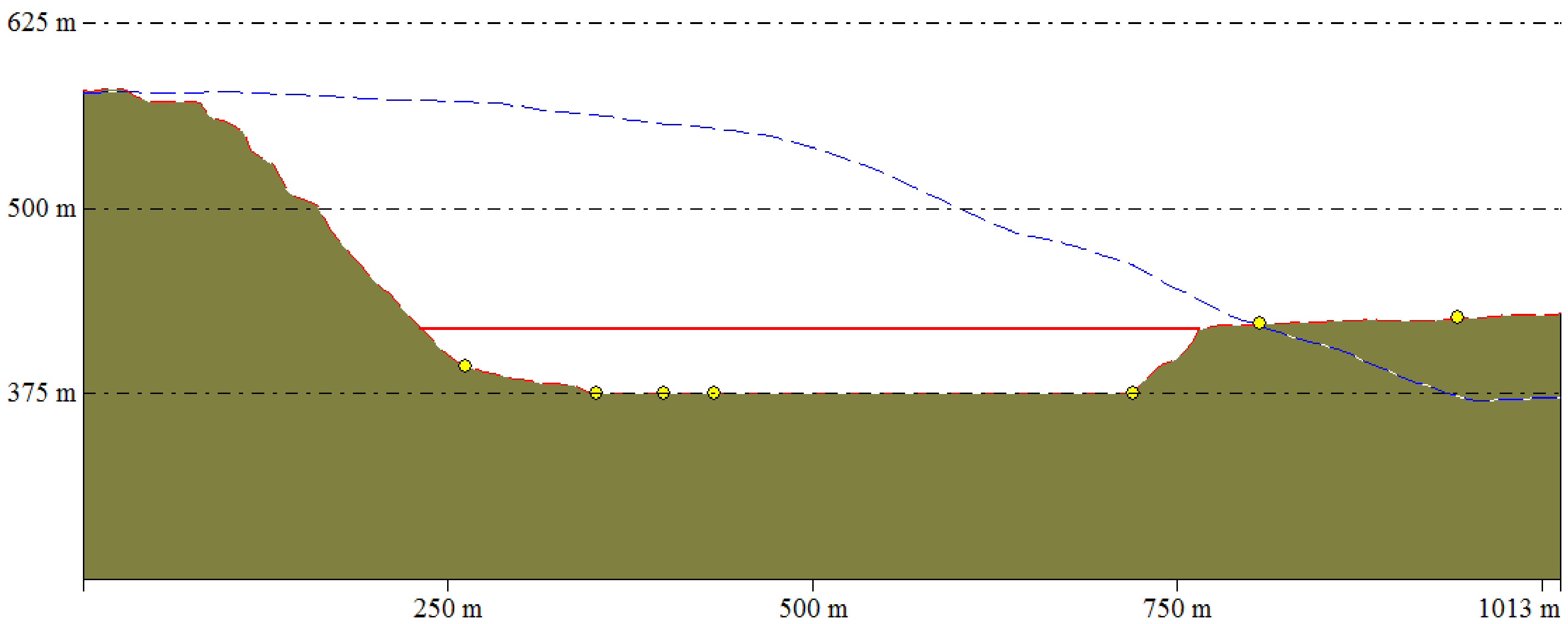
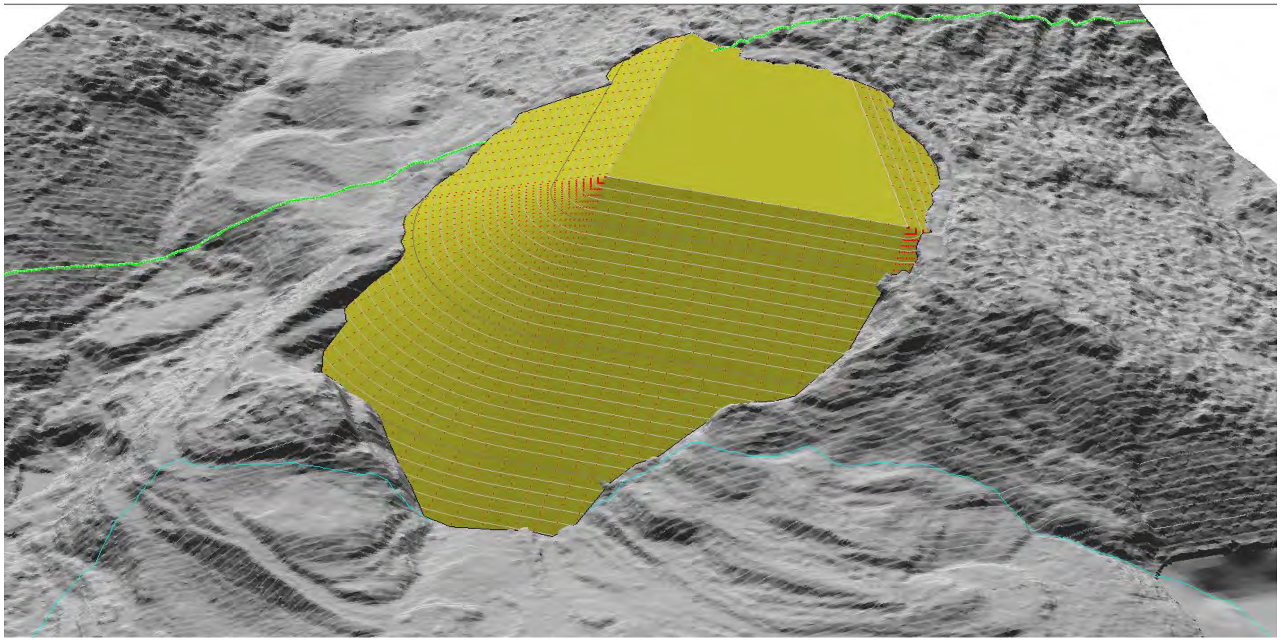
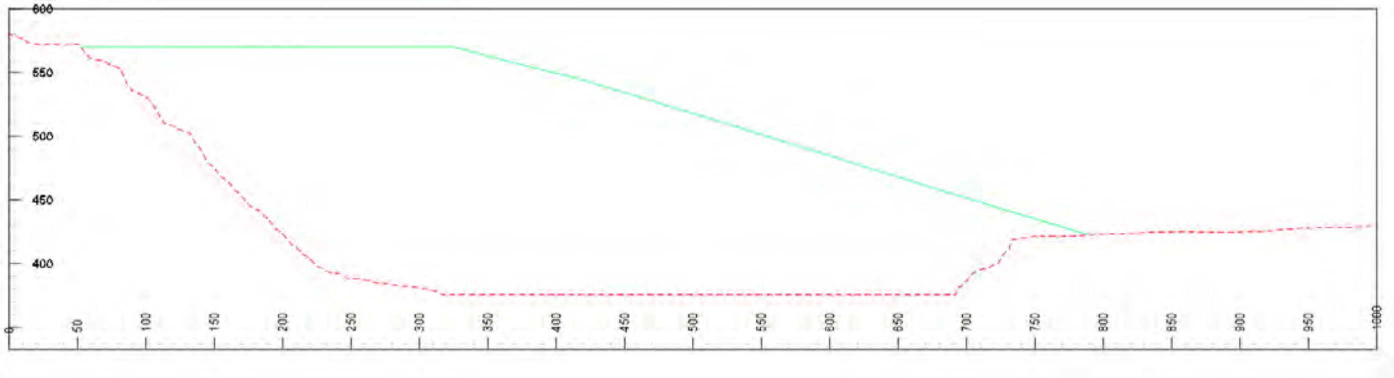


Estimated volume available for fill placement between El. 375 m (top of the Porcupine Pit Lake) and the north rim of the Porcupine Pit at El. 418 m (shown by the red line in the cross-section) is 2.6 M m3

Wood Environment & Infrastructure Solutions	PROJECT:	Clinton Creek Remediation Project
	TITLE:	**Porcupine Pit Backfill Volume between El. 375 m and 418 m**
CLIENT: **Government of Yukon**		

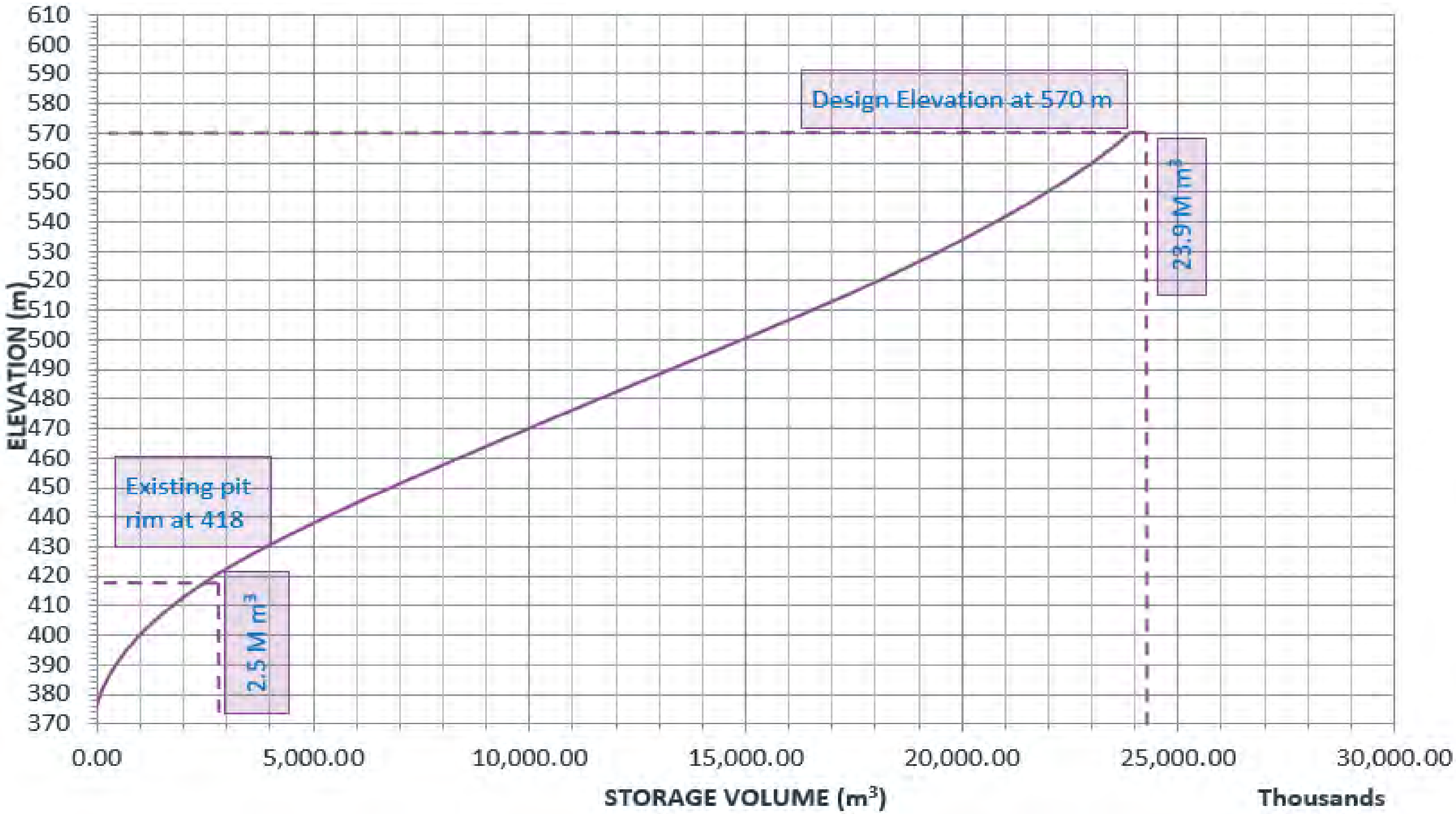
DATE:	JOB No.:	FILE:	FIGURE No.:	REV.
August 2019	VE52705D	Storage & Excavation Figures.Rev-01.xlsx	5.1	A





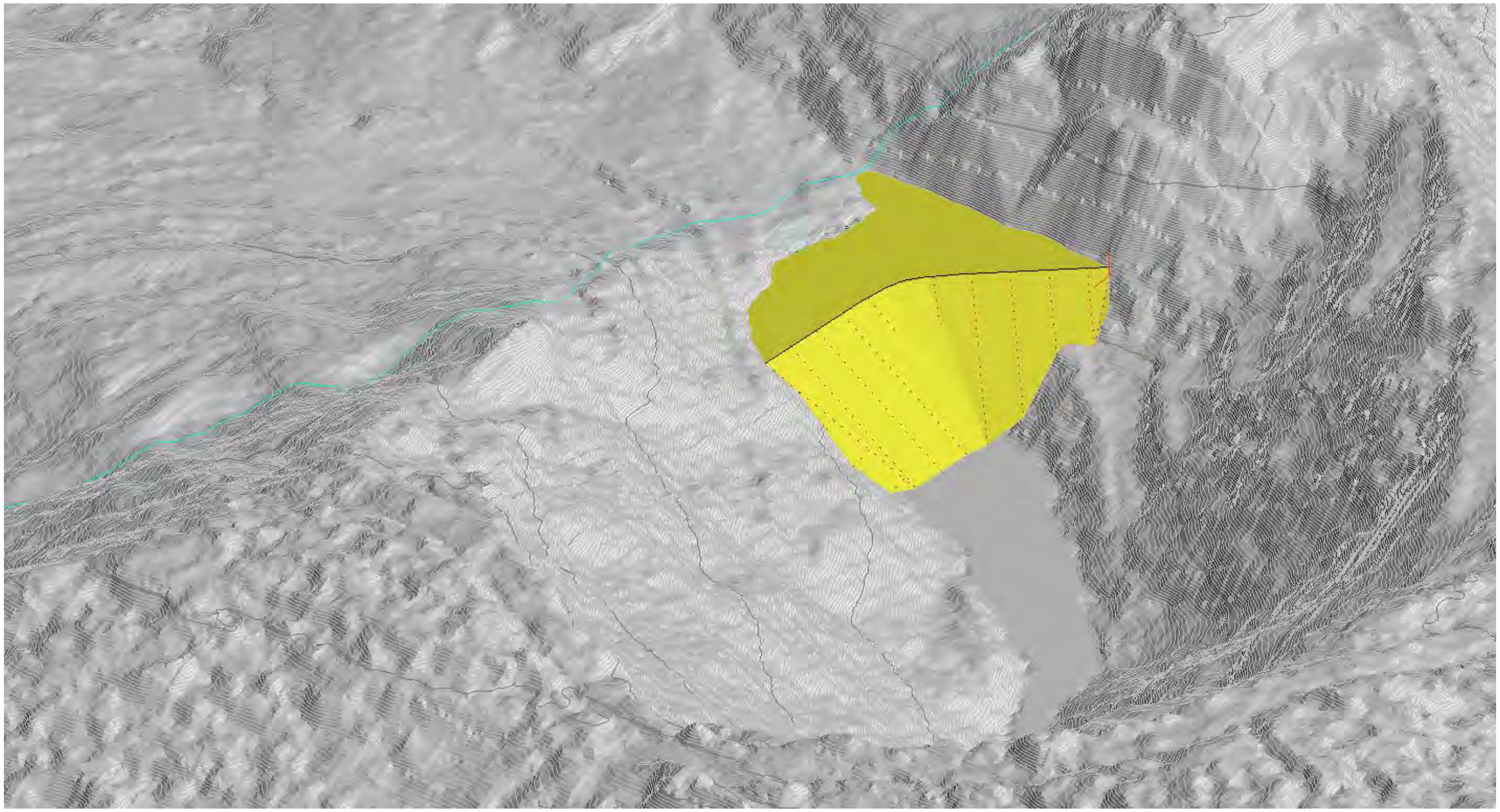
The overall storage capacity of the Porcupine Pit with assumed slopes of 3H:1V above the current topography, as shown, is approximatly 23.9 M m³.

Wood Environment & Infrastructure Solutions — wood.	PROJECT: Clinton Creek Remediation Project				
	TITLE: Porcupine Pit Shell at 3H:1V to El. 570 m				
CLIENT: Government of Yukon	DATE: August 2019	JOB No.: VE52705D	FILE: Storage & Excavation Figures.Rev-01.xlsx	FIGURE No.: 5.2	REV. A



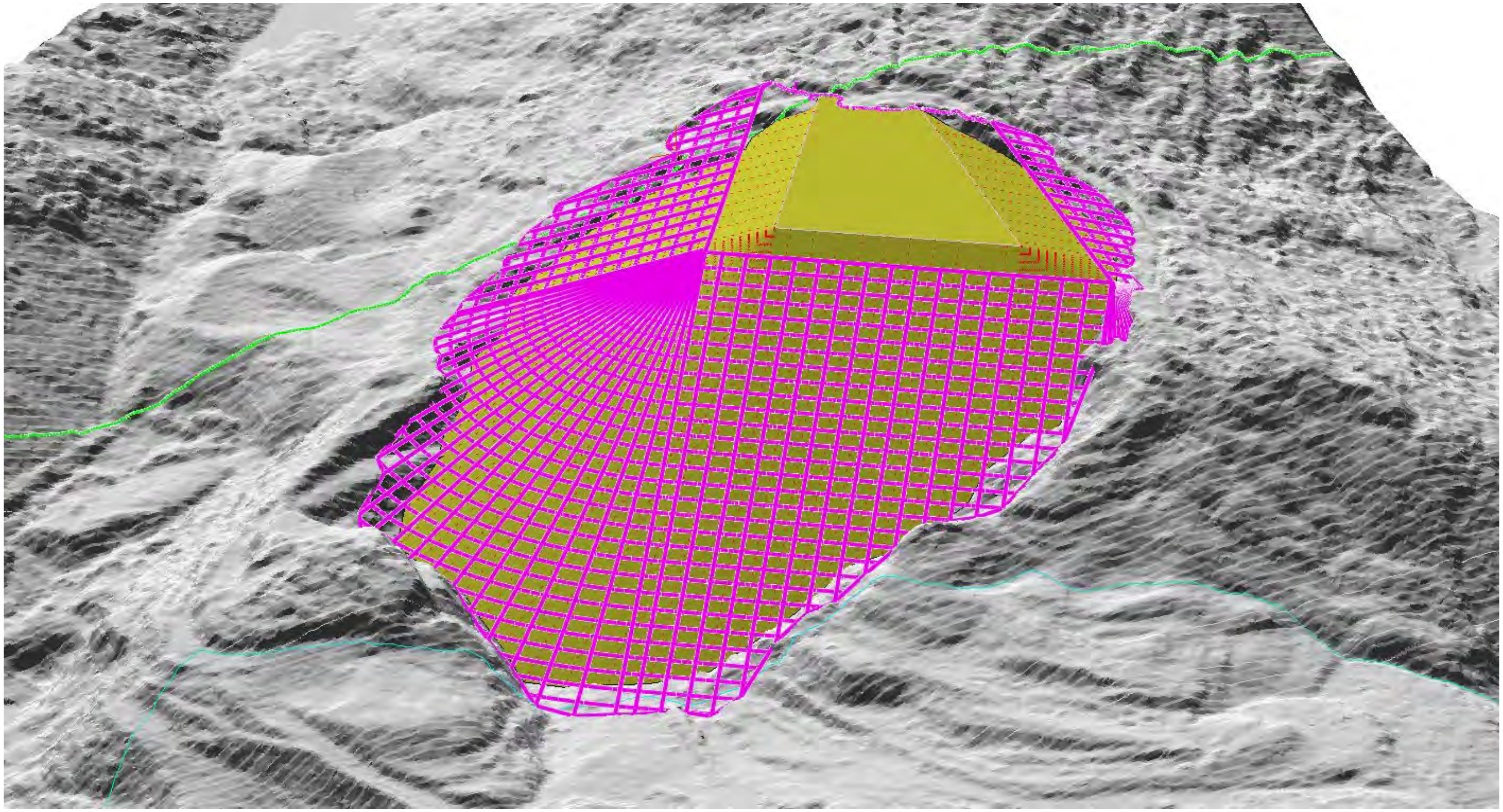
The overall storage capacity of the Porcupine Pit with assumed slopes of 3H:1V above the current topography, as shown, is approximatly 23.9 M m^3.

Wood Environment & Infrastructure Solutions	PROJECT: Clinton Creek Remediation Project				
CLIENT: Government of Yukon	TITLE: Porcupine Pit Storage Elevation Curve				
	DATE: August 2019	JOB No.: VE52705D	FILE: Storage & Excavation Figures.Rev-01.xlsx	FIGURE No.: 5.3	REV. A



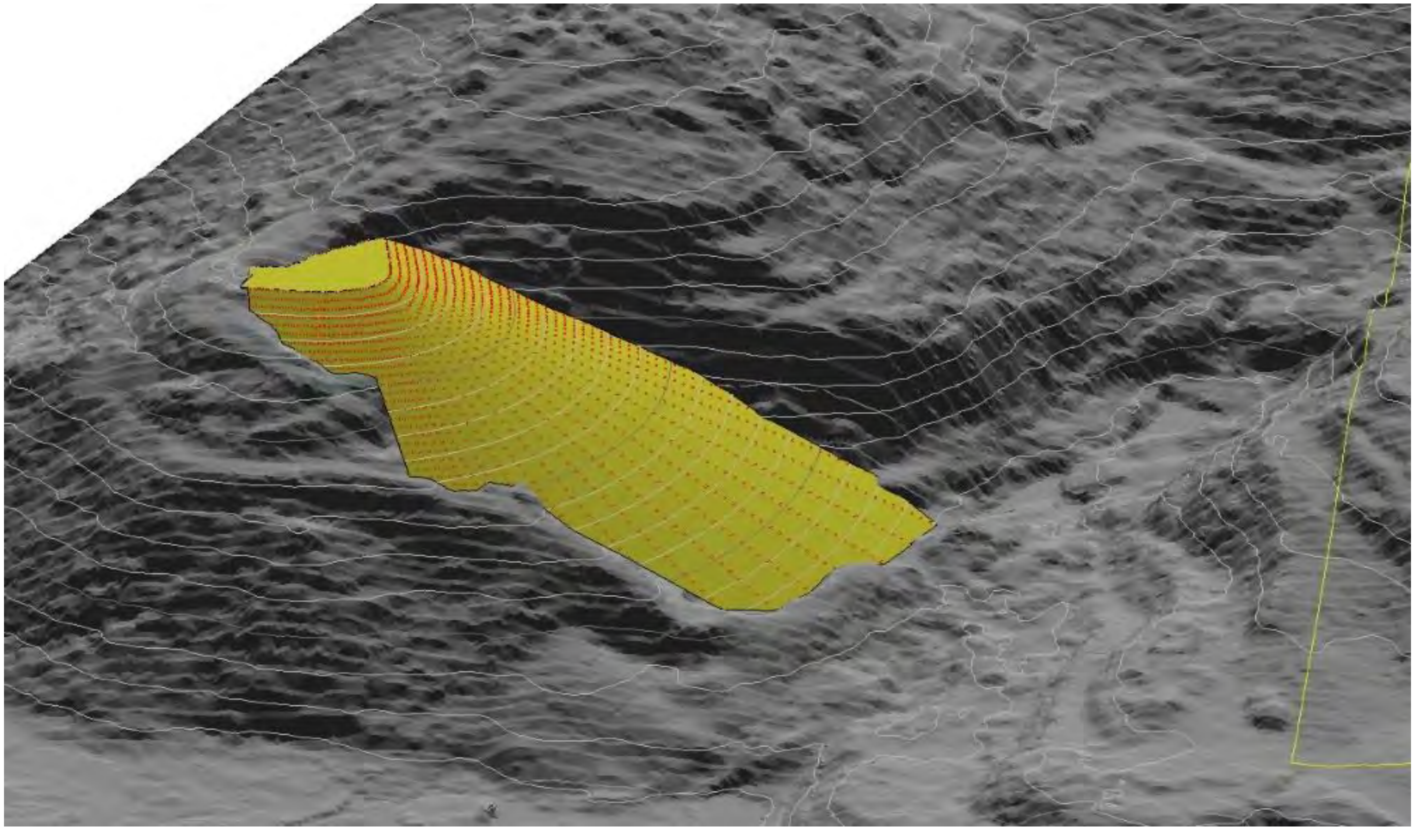
The initial shell required along the north pit wall of Porcupine Pit, with a slope of 2H:1V up to the lip of the pit at El. 418 m.

Wood Environment & Infrastructure Solutions	PROJECT: Clinton Creek Remediation Project				
	TITLE: 3-D Model of the North Wall Shell of Porcupine Pit				
CLIENT: Government of Yukon	DATE: August 2019	JOB No.: VE52705D	FILE: Storage & Excavation Figures.Rev-01.xlsx	FIGURE No.: 5.4	REV. A



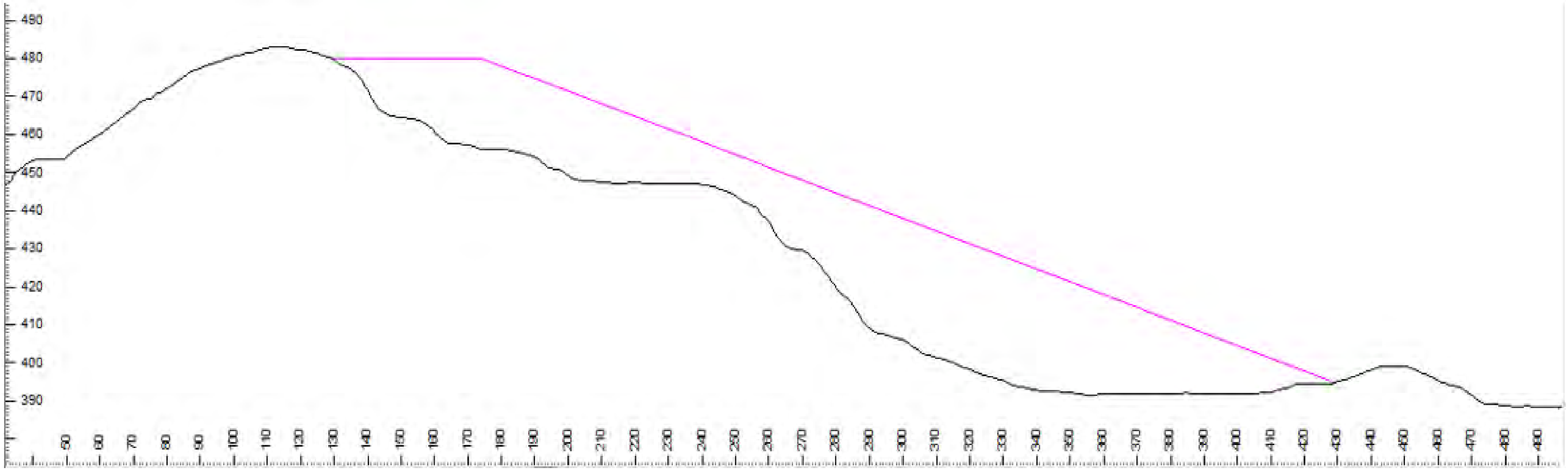
The final shell design to El. 570 m with a 3H:1V slope between El. 418 and El. 570 m

Wood Environment & Infrastructure Solutions	PROJECT: Clinton Creek Remediation Project				
	TITLE: 3-D Model of the shell to 570 m elevation for Porcupine Pit				
CLIENT: Government of Yukon	DATE: August 2019	JOB No.: VE52705D	FILE: Storage & Excavation Figures.Rev-01.xlsx	FIGURE No.: 5.5	REV. A



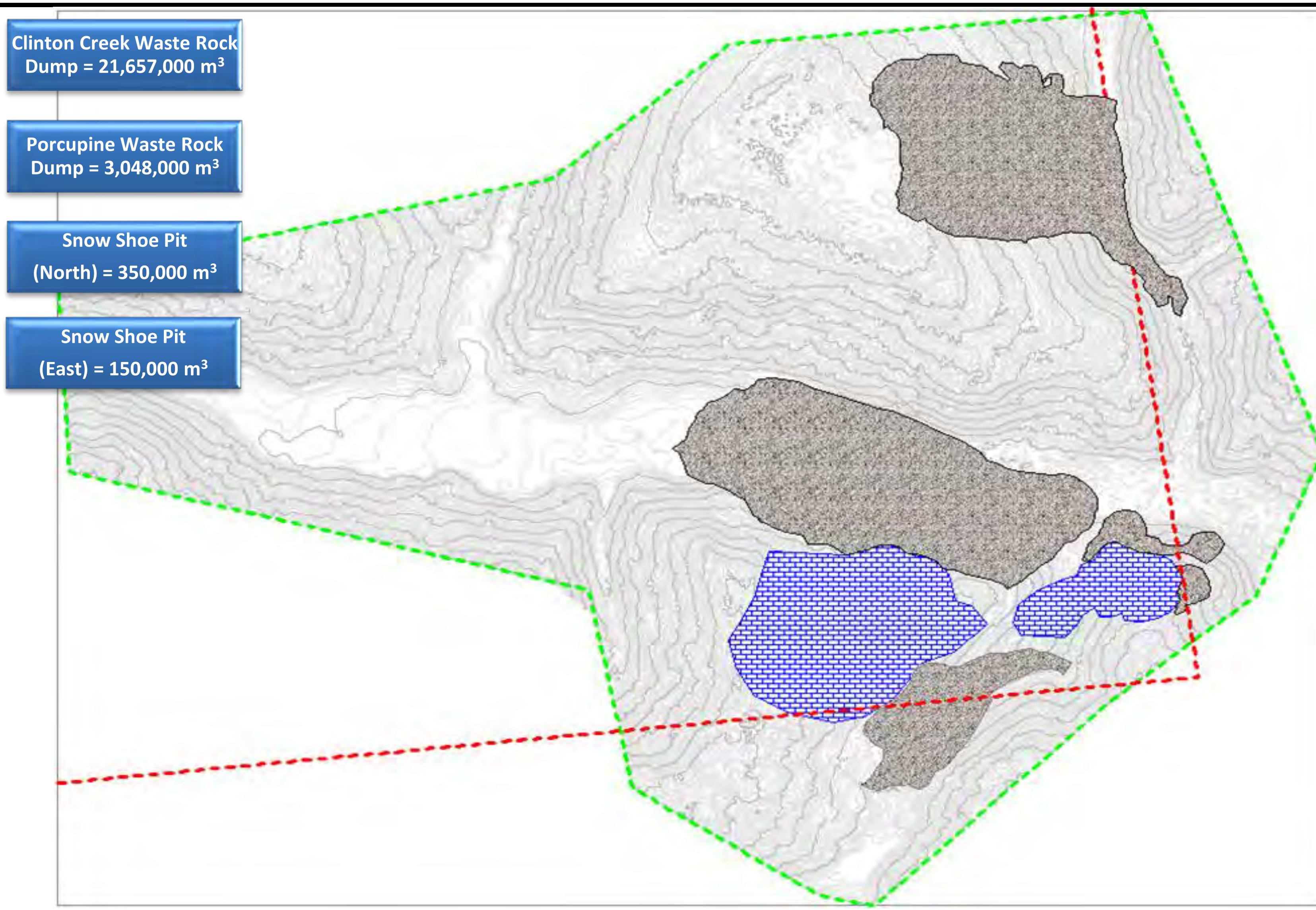
The estimaged potential storage volume available in the Snowshoe Pit is 608,000 m^3, based on the assumed 3H:1V slopes shown.

Wood Environment & Infrastructure Solutions	PROJECT: Clinton Creek Remediation Project				
	TITLE: Snowshoe Pit Storage Volume Configuration				
CLIENT: Government of Yukon	DATE: August 2019	JOB No.: VE52705D	FILE: Storage & Excavation Figures.Rev-01.xlsx	FIGURE No.: 5.6	REV. A



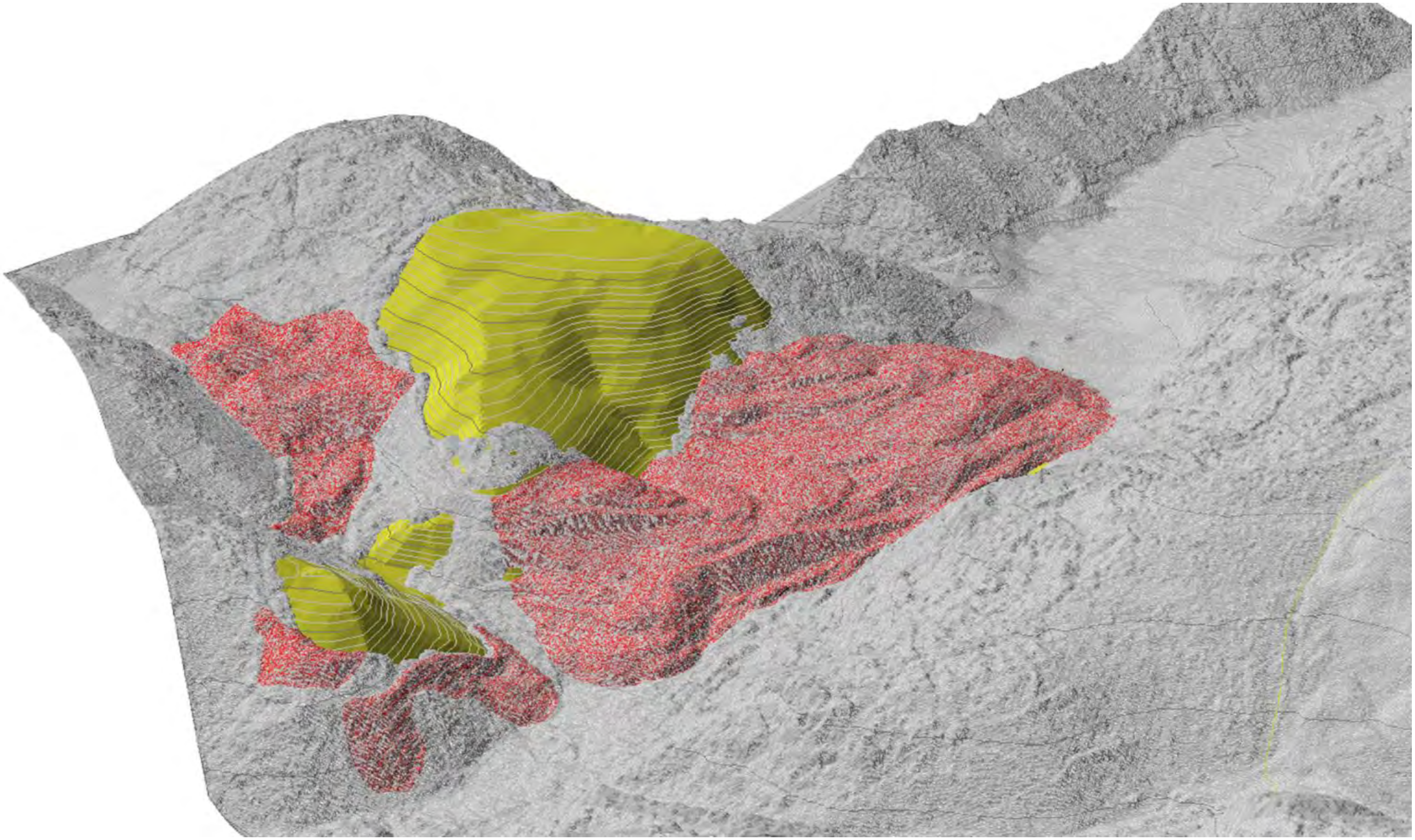
Snowshoe Pit profile view showing the available storage up to El. 480 m, based on the final profile shown in magenta.

Wood Environment & Infrastructure Solutions	PROJECT:	**Clinton Creek Remediation Project**			
	TITLE:	**Snowshoe and Creek Pit Profile to El. 480 m**			
CLIENT: **Government of Yukon**	DATE: August 2019	JOB No.: VE52705D	FILE: Storage & Excavation Figures.Rev-01.xlsx	FIGURE No.: 5.7	REV. A



Waste rock volumes in the Clinton Creek, Porcupine Creek and Snowshoe Pit Waste Rock Dumps

Wood Environment & Infrastructure Solutions (wood.)	PROJECT: Clinton Creek Remediation Project				
CLIENT: Government of Yukon	TITLE: Waste rock volumes in the Waste Rock Dumps				
	DATE: August 2019	JOB No.: VE52705D	FILE: Storage & Excavation Figures.Rev-01.xlsx	FIGURE No.: 6.1	REV. A



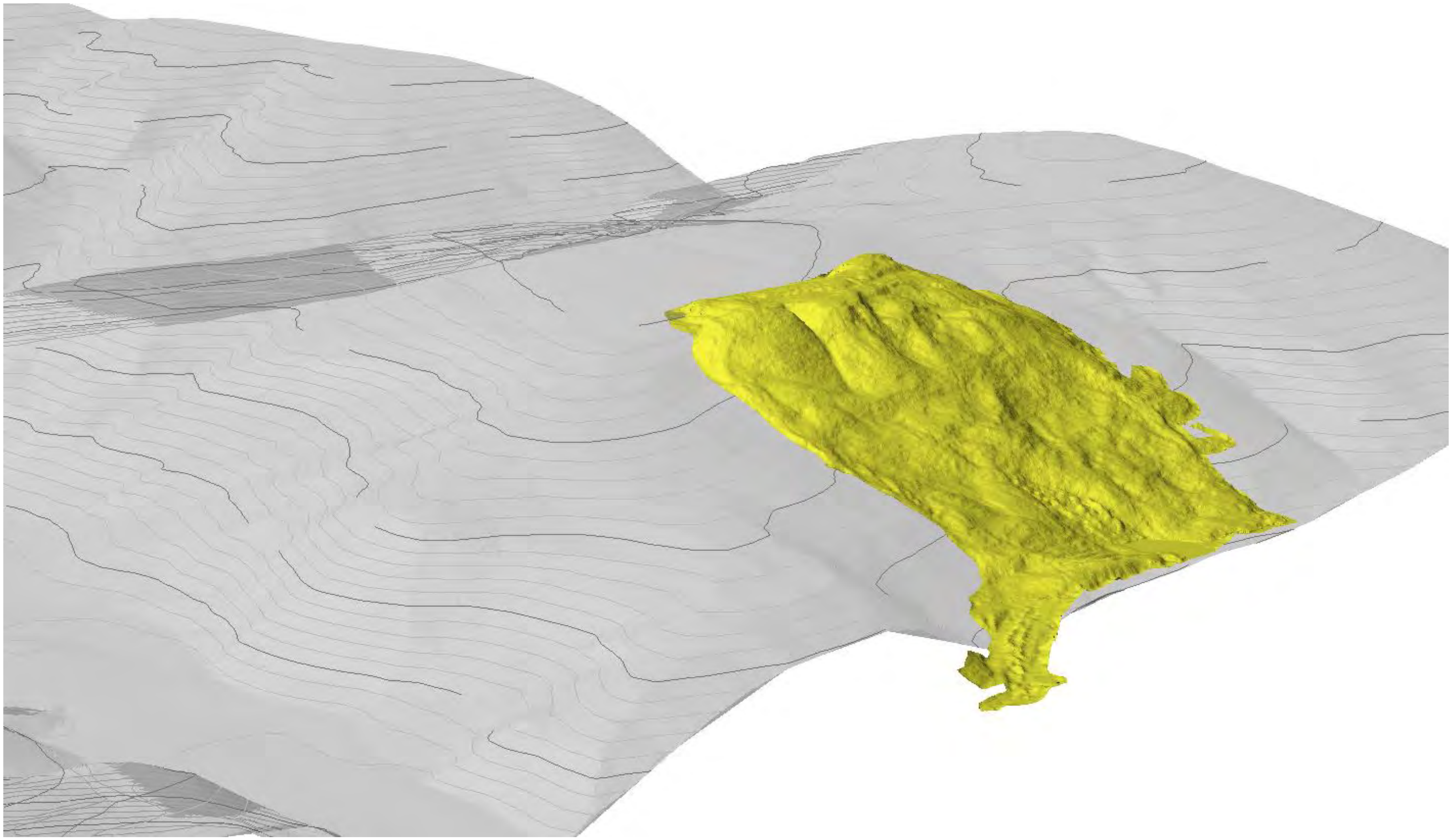
3-D Model of the combined 2012 DEM and 1949 DEM with the -20 m adjustment showing the pits (in green) and waste dumps (in red)

Wood Environment & Infrastructure Solutions	PROJECT: Clinton Creek Remediation Project				
CLIENT: Government of Yukon	TITLE: 3-D model of the pits and waste dumps				
	DATE: August 2019	JOB No.: VE52705D	FILE: Storage & Excavation Figures.Rev-01.xlsx	FIGURE No.: 6.2	REV. A

Plan view of the Wolvering Tailings Dump footprint, with a disturbed footprint of 448,000 m^2.

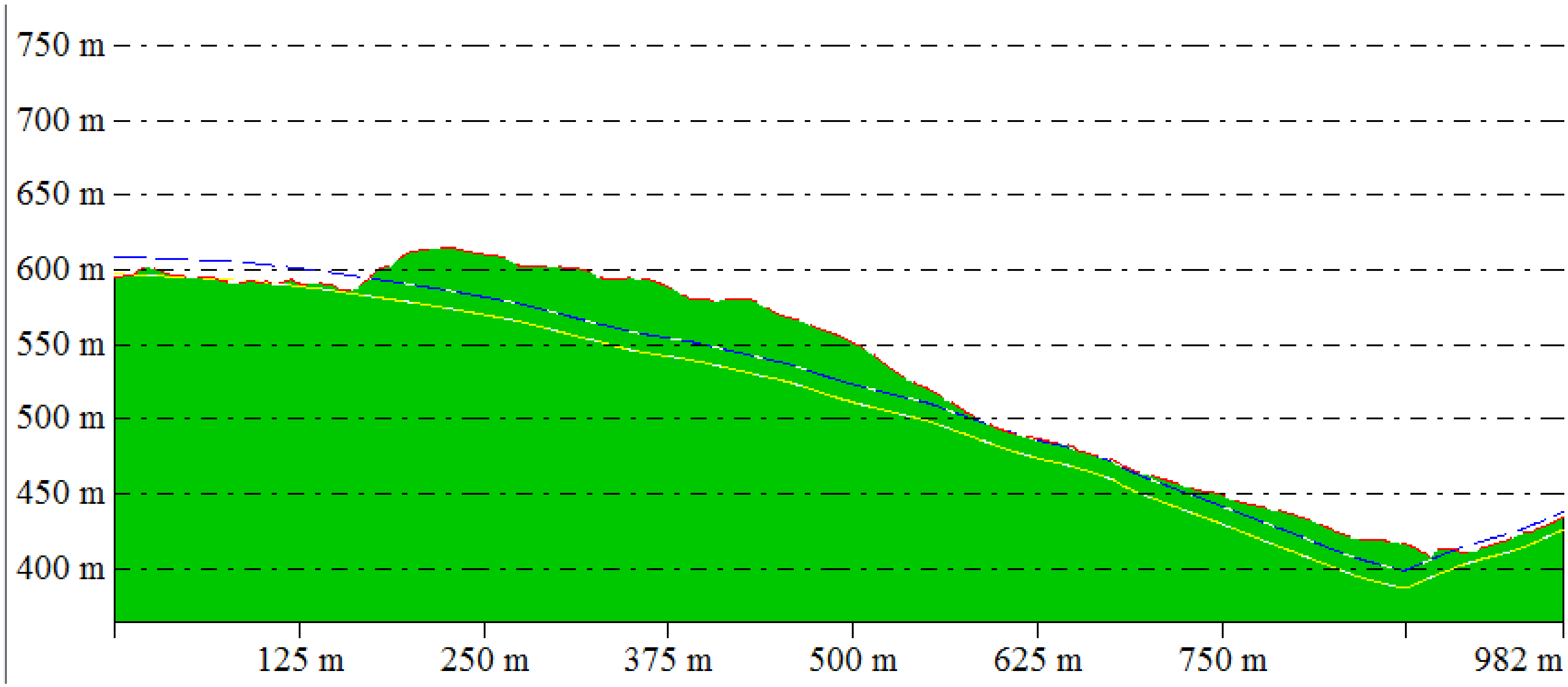
Wood Environment & Infrastructure Solutions — wood.	PROJECT:	**Clinton Creek Remediation Project**
CLIENT: **Government of Yukon**	TITLE:	**Plan View of Wolverine Tailings Dump**

DATE:	JOB No.:	FILE:	FIGURE No.:	REV.
August 2019	VE52705D	Storage & Excavation Figures.Rev-01.xlsx	6.3	A



3-D model of the tailngs draped on the 1949 DEM contours.

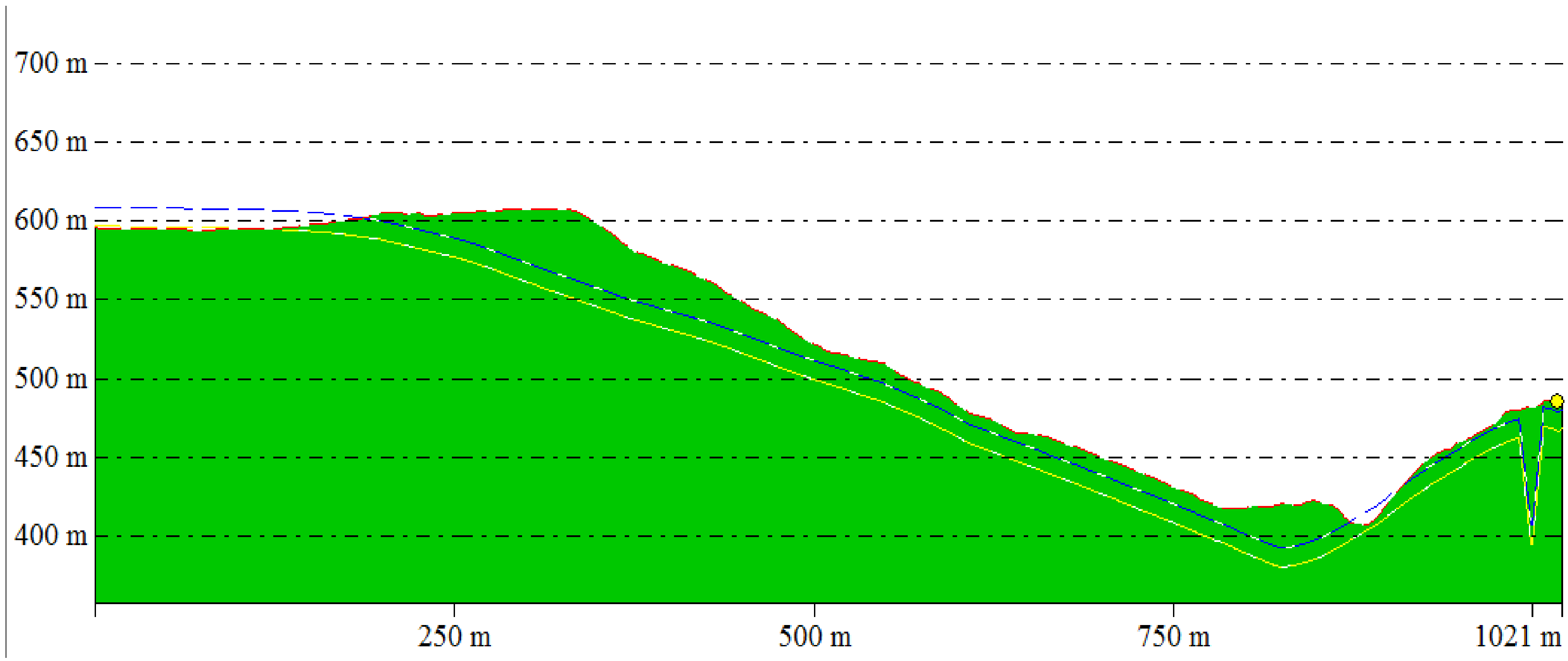
Wood Environment & Infrastructure Solutions		PROJECT:	Clinton Creek Remediation Project			
		TITLE:	3-D Model of the Tailings			
CLIENT: Government of Yukon		DATE: August 2019	JOB No.: VE52705D	FILE: Storage & Excavation Figures.Rev-01.xlsx	FIGURE No.: 6.4	REV. A



Cross-section of the north lobe of the Wolverine Tailings Dump, showing the 2012 DEM surface (the area colourd green), the 1949 DEM with the -13 m adjustment (blue dashed line) and the 1949 DEM with the -25 m adjustment (yellow line).

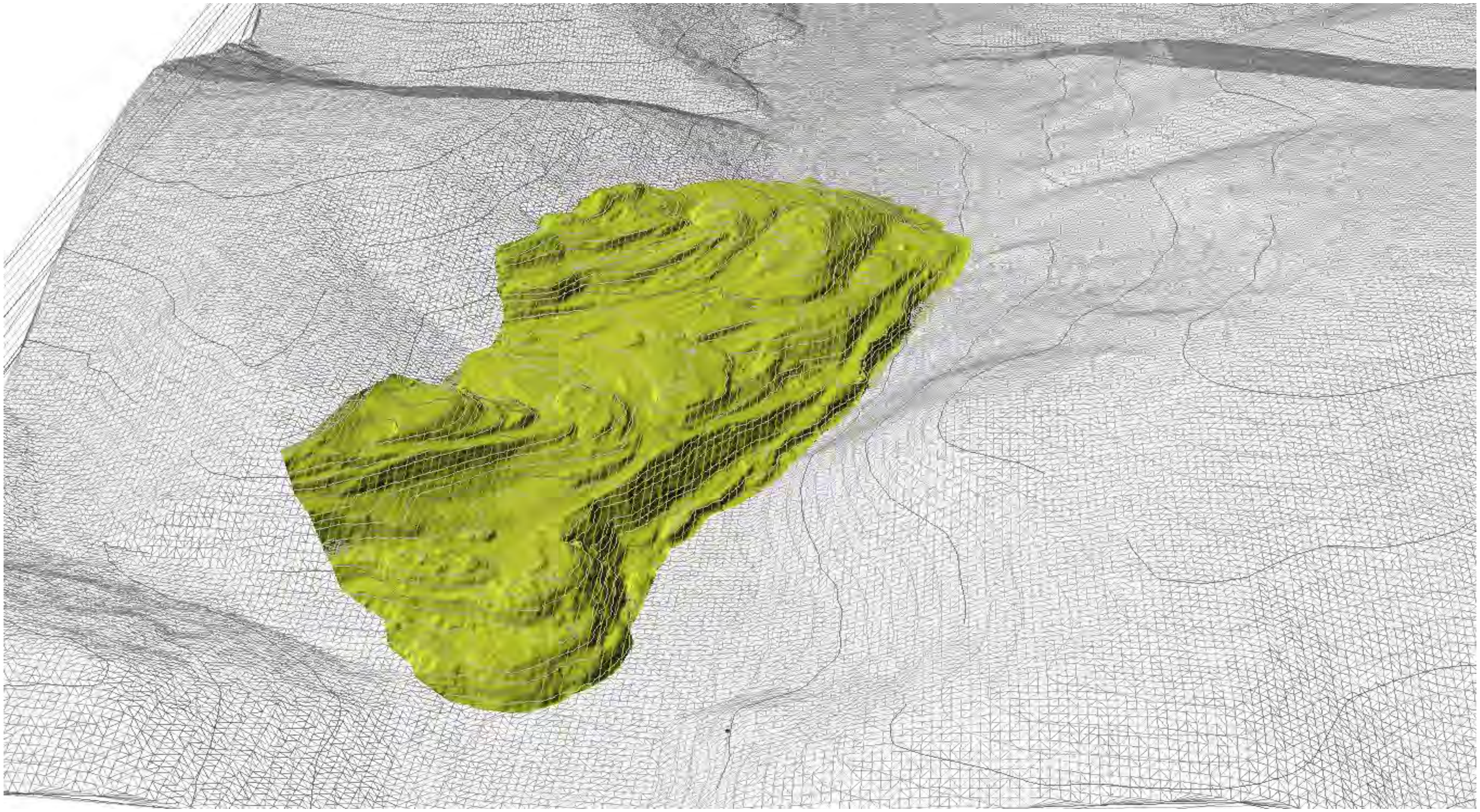
Wood Environment & Infrastructure Solutions

CLIENT: Government of Yukon	PROJECT: Clinton Creek Remediation Project				
	TITLE: **North Lobe Cross-Section**				
	DATE: August 2019	JOB No.: VE52705D	FILE: Storage & Excavation Figures.Rev-01.xlsx	FIGURE No.: 6.5	REV. A



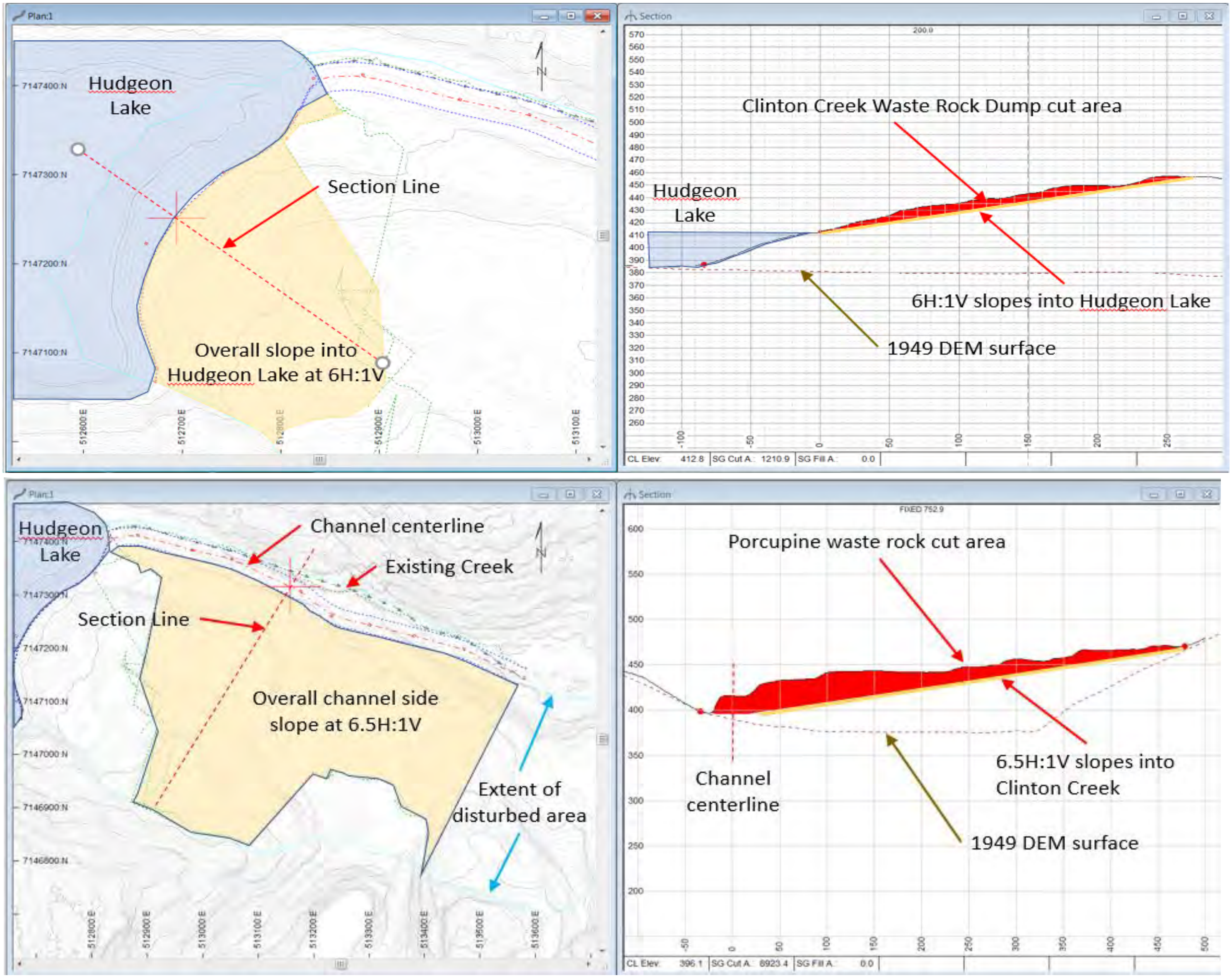
Cross-section of the south lobe of the Wolverine Tailings Dump, showing the 2012 DEM surface (the area colourd green), the 1949 DEM with the -13 m adjustment (blue dashed line) and the 1949 DEM with the -25 m adjustment (yellow line).

Wood Environment & Infrastructure Solutions		PROJECT: Clinton Creek Remediation Project			
		TITLE: South Lobe Cross-Section			
CLIENT: Government of Yukon		DATE: August 2019	JOB No.: VE52705D	FILE: Storage & Excavation Figures.Rev-01.xlsx	FIGURE No.: 6.6 / REV. A

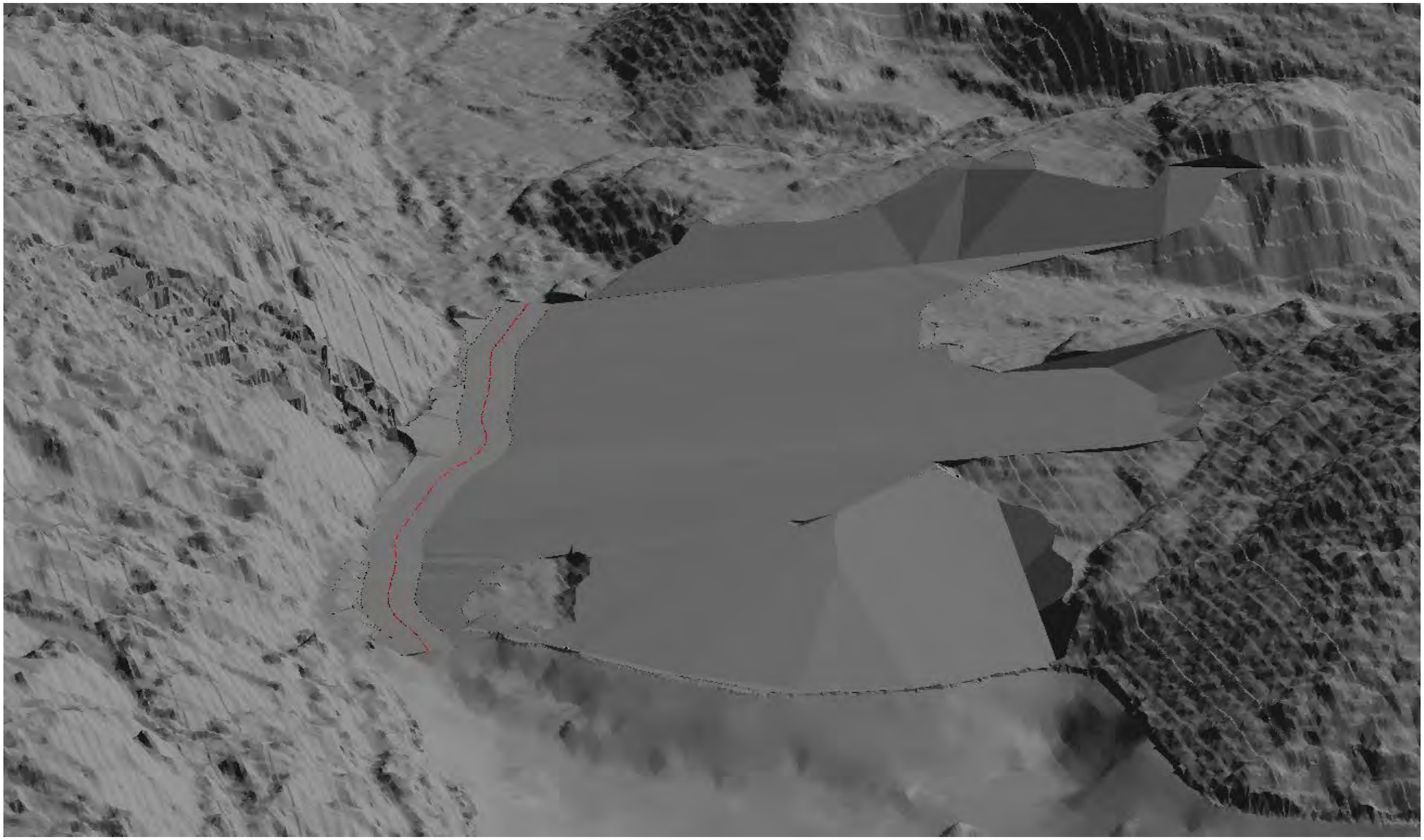


The Clinton Creek Waste Dump, shown in green with the 2012 DEM contours shown, contains approximatly 21,657,000 m3 of waste material, based on the difference between the 2012 DEM and the 1949 DEM.

Wood Environment & Infrastructure Solutions	PROJECT: Clinton Creek Remediation Project				
CLIENT: Government of Yukon	TITLE: 3-D View of the Clinton Creek Waste Dump on the 1949 DEM				
	DATE: August 2019	JOB No.: VE52705D	FILE: Storage & Excavation Figures.Rev-01.xlsx	FIGURE No.: 8.1	REV. A

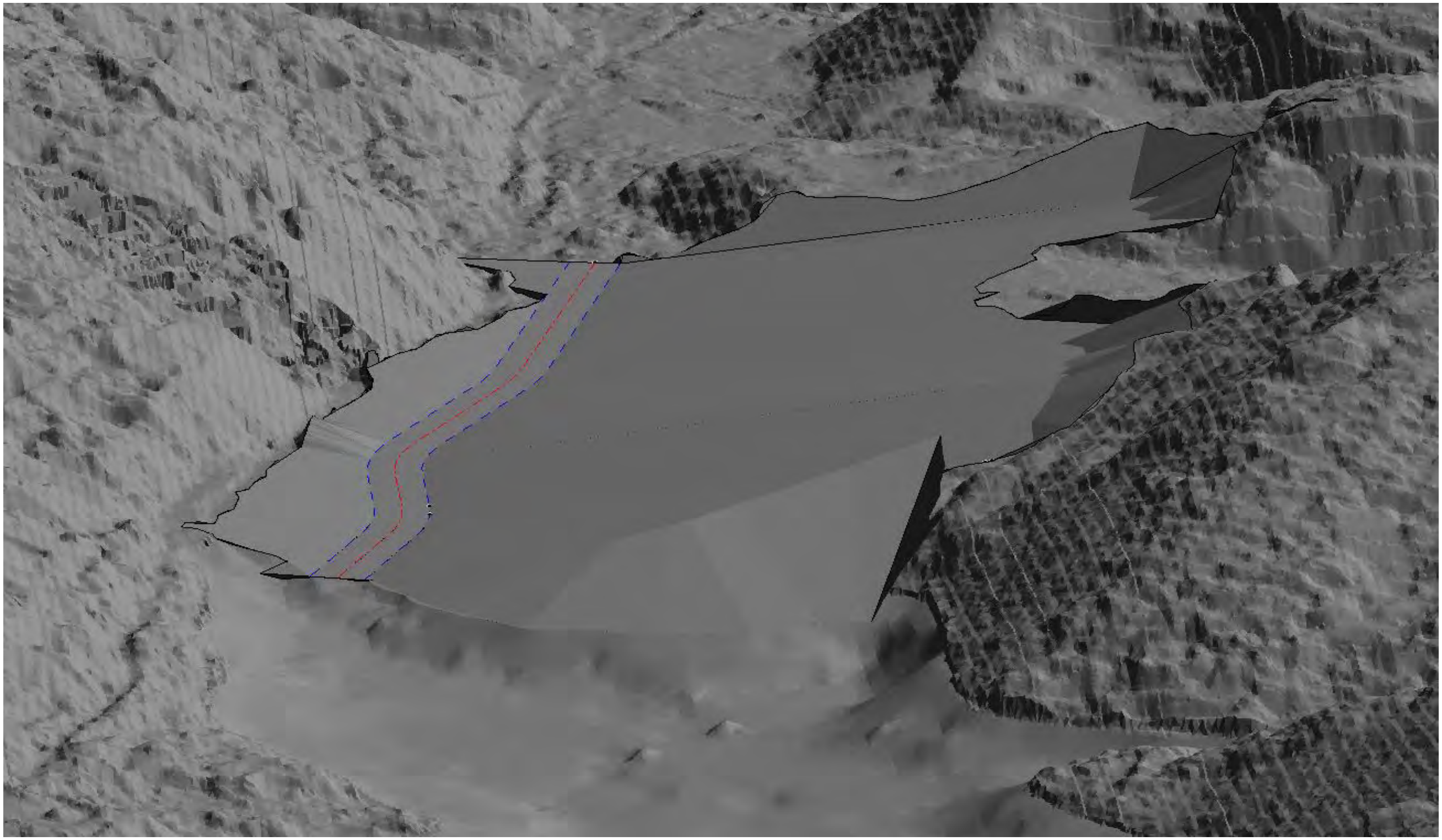


Wood Environment & Infrastructure Solutions		PROJECT:	Clinton Creek Remediation Project		
		TITLE:	CC1 - Required Cuts		
CLIENT: Government of Yukon	DATE: August 2019	JOB No.: VE52705D	FILE: Storage & Excavation Figures.Rev-01.xlsx	FIGURE No.: 8.2	REV. A



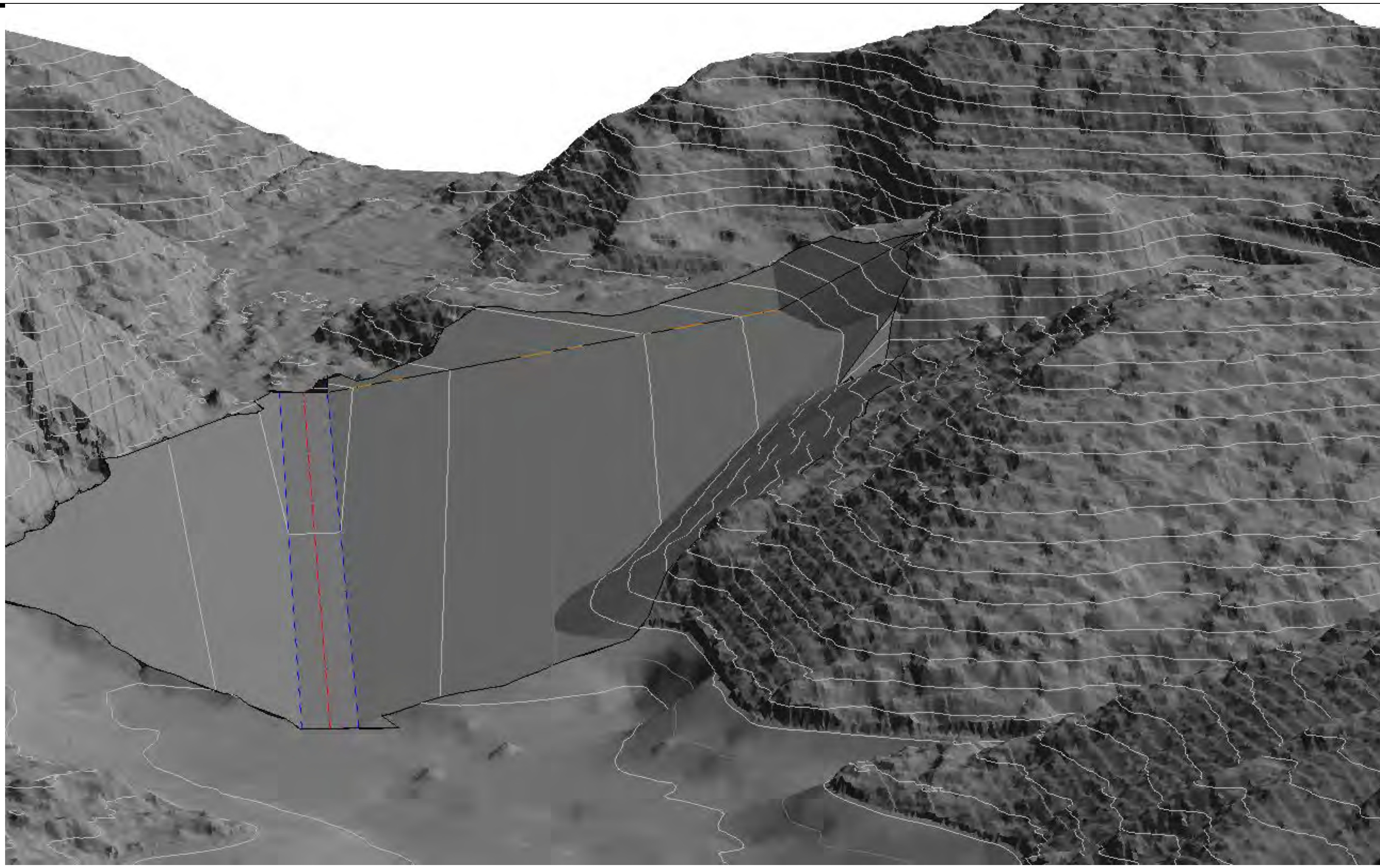
3-D model of the completed Clinton Creek Option 1 channel and excavated slopes, looking east

Wood Environment & Infrastructure Solutions wood.	PROJECT: Clinton Creek Remediation Project				
	TITLE: CC1 - 3-D Model of the Completed Channel Construction				
CLIENT: Government of Yukon	DATE: August 2019	JOB No.: VE52705D	FILE: Storage & Excavation Figures.Rev-01.xlsx	FIGURE No.: 8.3	REV. A



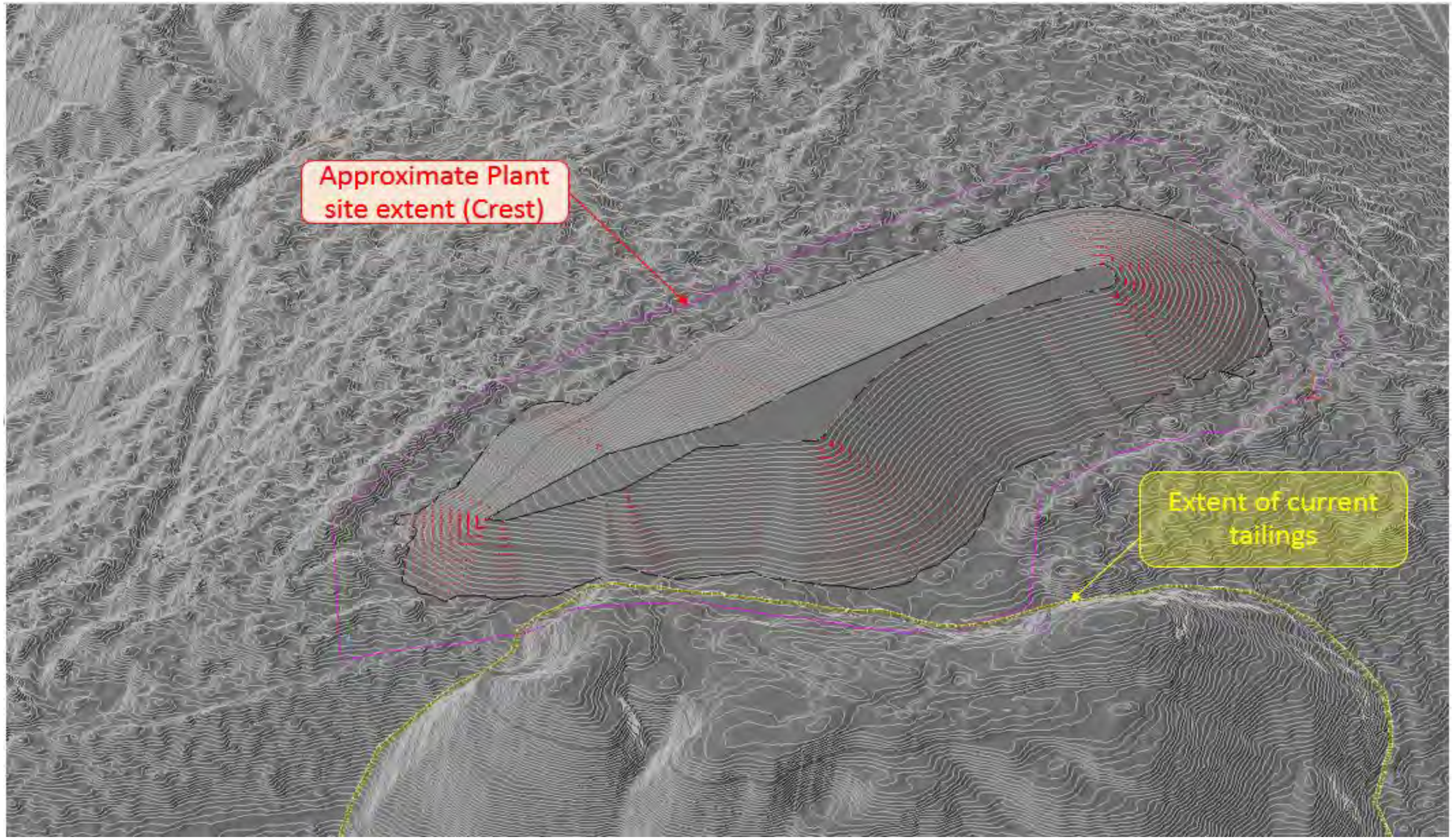
3-D model of the completed Clinton Creek Option 2 channel and excavated slopes, looking east

Wood Environment & Infrastructure Solutions	PROJECT: Clinton Creek Remediation Project				
	TITLE: **CC2 - 3-D Model of the Completed Channel Construction**				
CLIENT: **Government of Yukon**	DATE: August 2019	JOB No.: VE52705D	FILE: Storage & Excavation Figures.Rev-01.xlsx	FIGURE No.: 8.4	REV. A



3-D model of the completed Clinton Creek Option 3 channel and excavated slopes, looking east

Wood Environment & Infrastructure Solutions	PROJECT:	Clinton Creek Remediation Project				
	TITLE:	CC3 - 3-D Model of the Completed Channel Construction				
CLIENT: Government of Yukon	DATE: August 2019	JOB No.: VE52705D	FILE: Storage & Excavation Figures.Rev-01.xlsx	FIGURE No.: 8.5	REV. A	



Wood Environment & Infrastructure Solutions

CLIENT: **Government of Yukon**

PROJECT: **Clinton Creek Remediation Project**

TITLE: **3-D Model of the Proposed Tailings Stockpile at the former Mill Site**

DATE:	JOB No.:	FILE:	FIGURE No.:	REV.
August 2019	VE52705D	Storage & Excavation Figures.Rev-01.xlsx	8.6	A

Appendix H
WC2 Design Basis

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Memo

To:	File		
From:	Ed McRoberts, PhD, PEng	**Reviewer:**	Brian Ross, PEng (AB)
	Karen Hincks, MSc, PGeo	**Wood File No.:**	VE52705
Date:	26 August 2019		
Re:	**Clinton Creek Site: Wolverine Creek 2 (WC2) Option Development**		

1.0 Scope

The scope of this memorandum is to present the geotechnical components of the WC2 Option for the Wolverine Tailings Dump, which is intended to provide a stable tailings configuration in the Wolverine Creek valley, such that the tailings remain in the valley. This option has several elements:

1. The North and South tailings lobes along the west valley wall, must be made stable under static and seismic loading conditions. These slopes require a buttress fill placed against the east valley wall, as discussed in Appendix F-2 of the main report. The earlier work simply looked at the size of berm required and did not address the following components.
2. In order to contain the buttress fills to the south, down the Wolverine Creek valley, a Buttress Fill Dam is required across Wolverine Creek.
3. The stability of the remaining perimeter tailings slopes to the west, south and north must also be left in a stable condition.
4. There must be a long-term stable outlet for Wolverine Creek along the buttress fills, and a spillway down the Buttress Fill Dam; design of the spillway has been completed by hydrotechnical engineers under separate cover.

This memorandum completes the design of the WC2 Option to the 10% level. The current design and layouts are considered to be indicative of what would be required if this option is selected by the Project Partners. There is a range of tradeoffs discussed that could optimize the design.

1.1 Appendices

There are two appendices to this document:

- Appendix I provides a summary of the Slope/W analysis.
- Appendix II provides the design layouts adopted for the 10% design.

2.0 Buttress Fill Dam

2.1 Criteria

As the Buttress Fill Dam retains liquefiable tailings it must be designed as dam with a minimum FoS of 1.3, see the Design Basis Memorandum, Appendix I. Measures are also required to control internal erosion and drainage.

2.2 Constraints

The crest elevation of the dam is taken to be 440 masl, per the analysis completed in Appendix F-2. The toe of the dam was considered to be constrained down-creek by a small tributary creek entering Wolverine Creek from the east; the ground elevation at the confluence is about 385 masl, as shown in Appendix I, Figure 1. Based on the constraints, a 55 m high structure at an overall slope of about 5.2H:1V is required; thus, the design objective is to provide a stable structure within these geometrical constraints.

Historically, failures of the tailings piles created run-outs downstream past the designated dam toe. Subsequently, and based on aerial photography, tailings were excavated down a constrained channel along the original valley floor and a boulder lined channel was emplaced. These tailings have not been investigated, but it is assumed they are loose and liquefiable. Any tailings left in place under the dam must be assumed to be seismically liquefiable.

One borehole, BH18-16, was located on the west valley wall within what would be the right abutment (looking downstream). Approximately 6 m of ice rich colluvium was reported under about 8 m of tailings and 1 m of thawed colluvium in this borehole. There may be additional ice rich material towards the valley bottom, as the thermal influence of Wolverine Creek is unlikely to thaw out all permafrost. However, in consideration of the long-term potential for thermal degradation of permafrost, the destabilizing effects of warm ice rich permafrost, or thawing permafrost, must therefore be accounted for in design.

The dam design concept selected on which to base cost estimates for WC2 options is given in Appendix I, Figure 2, with the following features:

1. Tailings are removed from approximately the lower 45% of the dam foundation.
2. In addition, all ice rich permafrost is excavated down to either competent dense colluvium, either thawed or with W/C(ice) contents less than 20% in the same zone, and to extend up the abutment slopes.
3. The excavated areas are backfilled with 95% Standard Proctor maximum dry density (SPMDD) frictional material with a 33^{o} friction angle.
4. Above the backfill and running up the slope is a 4 m high drain consisting of an internal coarser grained drain 2 m thick and 2 – 1 m layers of finer granular transition zone between foundation materials and the select rockfill dam shell. Two drain pipes are to be installed transverse to centerline with separate out-takes.
5. The drain continues up slope to the chimney drain.
6. The shell is to be constructed out of select free-draining rock fill in order to facilitate the spillway design, and overall drainage issues.

2.3 Stability Analysis

Several models for stability analysis were considered.

1. Ice rich permafrost mobilizes 50 kPa cohesion, consistent with previous analyses.
2. Thawing colluvium mobilizes a friction angle of 22 degrees and a Ru = 0.7 to account for excess pore pressures during thaw.
3. Thawed colluvium mobilizes a friction angle of 22 degrees and a piezometric level in drain blanket.

4. Excavated colluvium in the lower 45% of a given 2-D section in the downslope direction replaced down to either low ice content colluvium, dense bedrock, or a maximum cut of 7 m.
5. The piezometric surface was assumed to be contained in the drain, and conservatively be at El 440 masl behind the chimney.

A summary of analysis is provided in Table 1 and Slope/W analyses are included as Appendix I.

Table 1: Summary of Buttress Fill Dam Analysis

Case	FoS
All frozen colluvium	**0.54**
Frozen colluvium under tailings; thawed downstream	**1.15**
Frozen colluvium under tailings; select granular downstream	**1.63 overall** **1.46 midslope popout**
All thawing colluvium	**0.52**
Thawing colluvium under tailings; thawed downstream	**1.23**
Thawing colluvium under tailings; select granular downstream	**1.72 overall** **1.61 midslope popout**
All thawed colluvium	**1.63 midslope popout**

Based on these analyses it can be concluded that unsatisfactory short-term performance could result if active measures are not taken to account for ice rich permafrost in the foundations. Active-measures could take the form of pre-thawing all ice rich material and installing wick drains to dissipate excess pore pressures in a timely manner; however, executing such a program on the relatively steep abutments slopes is problematic. Therefore, at this stage of design, it is assumed that for this option the ice rich permafrost should be excavated down to competent ice poor foundations, or bedrock, within the as indicated 45% zone. Clearly some optimization of the excavation zones for tailings and permafrost and/or the overall dam slope would be possible in subsequent design stages. The FoS presented are for the overall slope in the steepest part of a "V" shaped valley and there are 3-Dimensional effects that have not been assessed at this stage. It is also noted that the details of the intersection of the buttressed east facing main tailings slope with the Buttress Fill Dam has not been determined at this time.

3.0 Perimeter Slopes of Main Tailings Pile

3.1 3.1 Criteria

A FoS of 1.3 is adopted for determining the required perimeter slopes. The perimeter slopes are defined herein as the remaining west, north, and south slopes.

3.2 Stability Analysis

Stability analyses have considered either slope flattening or provision of a toe berm in order to obtain the required FoS for long term stability. Table 2 presents a summary of the slope stability analyses that we undertaken for the options for stabilizing the side slopes of the tailings. A brief discussion of the options is presented in the following paragraphs.

For slope flattening, the primary failure mechanism for the existing perimeter slopes is seismically induced liquefaction of a relatively thin zone at the tailings / original ground contact. The liquefied strength mobilized is considered to be weaker that the strength of either ice rich frozen ground or thawing permafrost at todays rates of thaw. Rapid thawing during initial tailings placement has been considered to be the main contributor to the original failures but static liquefaction cannot be ruled out.

A toe berm would necessarily be founded on currently undisturbed terrain just beyond the perimeter toe. The lower portion of the north slope provides evidence of historical attempts to construct a buttress fill directly on permafrost terrain. Photographs reported by Golder (1978) considered to be taken along the toe of the north perimeter indicate the push-up of thawed silt/clay colluvium, and BH 78-13 reports permafrost with excess ice contents to at least 7 m depth, which is similar to the stratigraphy in BH18-16. The attempts to buttress this slope apparently ceased when the mine was shut-down. Any toe berm therefore should allow for excavation of 7m of original grounds and replacement with compacted free-draining granular fill. Stability analysis were undertaken for the berm option, see Appendix I, Figure 3 for a generic 60 m high tailings slope example.

Alternatively, to achieve criteria the existing slopes can be cut back from the toe. This would require cuts in the order of 7H:1V, per the analysis undertaken for a generic 60 m high slope presented in Appendix I, Figure 4.

Table 2: Perimeter Berm for 10% Design Cost Estimate

Tailing Height	Berm Width m	Berm Height m
≤ 20 m	35	10
≤ 40 m	60	15
≤ 60 m	70	20

Notes:

1. Excavate and backfill at toe to depth of 7 m
2. Downstream slope of berm is 3H:1V.
3. Height of berm based on contact between tailings and upslope crest of excavation cut.
4. Field fit for location of upstream crest of excavation cut.
5. Excavation must be made with "3-D" panels based on local stability.
6. Base of excavation cut may slope depending on local geometry.

3.3 Selection of Cut versus Toe Berm

The Buttress Fill requires an estimated 3,954,000 m^3 of fill (see Appendix F-2 for details on the required Buttress Fill). This fill could be all imported from excavated Clinton Creek waste. In this case, the stabilization of the perimeter slopes would be attained by the perimeter toe berm.

At this time, a hybrid approach was considered with the following components, per Appendix II Slides 12 through 17:

1. Major cut slopes at 7H:1V were considered, as shown. This would supply 2,370,000 m^3 of fill for the buttress, which would be slightly less when compacted.
2. Perimeter berms are deployed along the north perimeter up to the 7H:1V cut area.
3. The fill slopes between the south side of the Buttress Fill and the crest of the Buttress Fill Dam are about 4 to 5H:1V, this is considered satisfactory for local stability.
4. The perimeter slopes between the south buttress fill and the original ground, facing south, is judged stable as is. The natural ground under the tailings rises up as the tailings in this area were disposed in a deeper gully.
5. Where practical the tailings slope facing West can be taken as 3H:1V, as any failure of this slope would not lead to blocking of any streams or water bodies. Stability analysis has not been completed on this slope.

An alternative approach that was not considered would be to excavate even more tailings (given there is about 7,800,000 m^3 total tailings) for the buttress fill.

4.0 Volume Summary & Compaction Requirements

A summary of all volume requirements is provided in Table 3.

Table 3: Summary of Volumes

	Components	Length (m)	Surface Area (m^2)	Volume (m^3)
Tailings Dump	Overall Tailings Volume	-	448,000	7,688,000
	Main Buttress File Volume (4.5H:1V)	-	190,000	3,954,000
	Excavated Tailing (7H:1V)	-	168,000	2,370,000
	Sub-Excavation Volume Perimeter Berm (2H:1V)	-	24,000	121,000
	Compacted Granular Fill (Berm)	-	50,000	550,000
	1 m Capping Over All Tailings	-	358,000	358,000
Buttress Fill Dam	Excavated Tailings and Ice Rich Colluvium Volume	-	29,000	169,000
	Select Rockfill Shell and Backfill Volume	-	53,000	738,000
	Chimney and Basal Drain Volume	-	52,000	192,000
	8 inch Perforated Pipes	400	-	-
	8 inch Solid Pipe Length	300	-	-

Notes:

1. All Buttress Fill Dam fill compacted to 95% SPMDD
2. All Perimeter Berms fill compacted to 95% SPMDD
3. Buttress fills above the expected long term water table to be compacted. It is anticipated that a lake could form upstream of the overall buttress fills depending on the final crest of the Buttress fill Dam (i.e., at 440 masl for the 10% Design scenario). Fills at 440 masl plus 5 m or 445 masl (allowing for capillary rise) would remain unsaturated and not seismically liquefiable. All fills to 10 m above the tailings / original ground contact will likely remain unsaturated. Therefore, all buttress fills at less than 445 masl, or a zone 10 m deep above the tailings / original ground contact must be compacted to 95% SPMDD.

5.0 Complications With 3-D Effects

The interactions with 3-D effects have not been considered for the 10% Design and need consideration if the Project Partners select this option. Some considerations are as follows:

1. **Perimeter Cut** Adopting the perimeter cuts as presented herein will also reduce the driving stress in the west downslope and therefore could reduce the current buttress fill requirements. This might also have a knock-on effect on reducing the height of Buttress Fill Dam.
2. **Stability Interactions** The stability interaction between a west sloping buttress fill and a right-angle transition to the north to south Buttress Fill Dam has not been analyzed. This would require a 3-D assessment utilizing a code such as FLAC.
3. **Buttress Fill Dam** Optimization of this dam with a 3-D model as complemented by appropriate site data may reduce the currently recommended measures taken for overall stability.

Sincerely,

Wood Environment & Infrastructure Solutions
a Division of Wood Canada Limited

Prepared by:

REGISTERED PROFESSIONAL GEOSCIENTIST
K.D. HINCKS
LICENSEE
NT/NU

28 August 2019.

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Principal Geotechnical Engineer

27 Aug 19

Karen Hincks, MSc, PGeo (NT)
Senior Engineering Geologist

Reviewed by:

Brian Ross, PEng (AB)
Geotechnical Engineer

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Appendix I

Stability Analysis

OVERALL SITE PLAN WC2

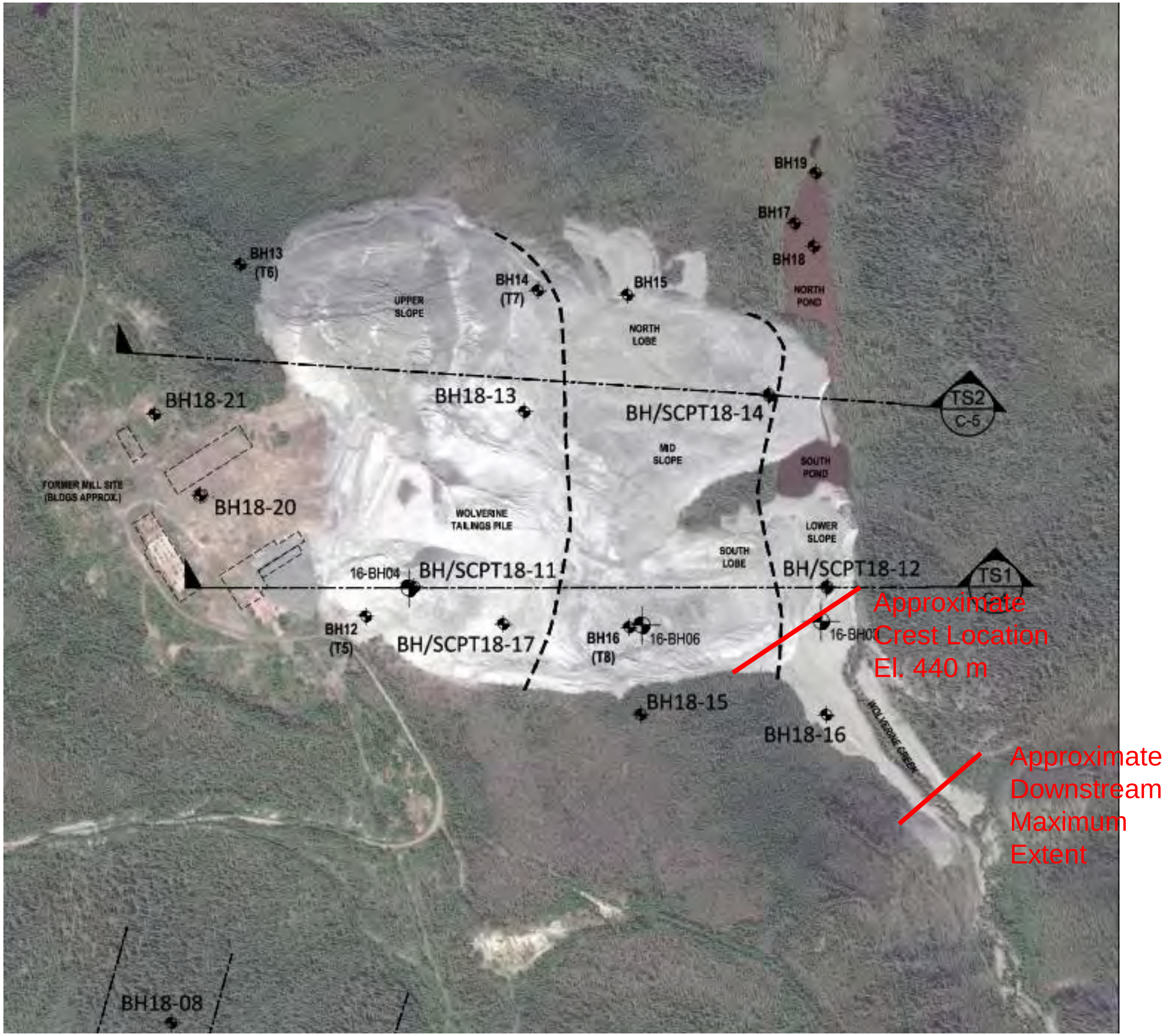


Figure 1

BUTTRESS FILL DAM

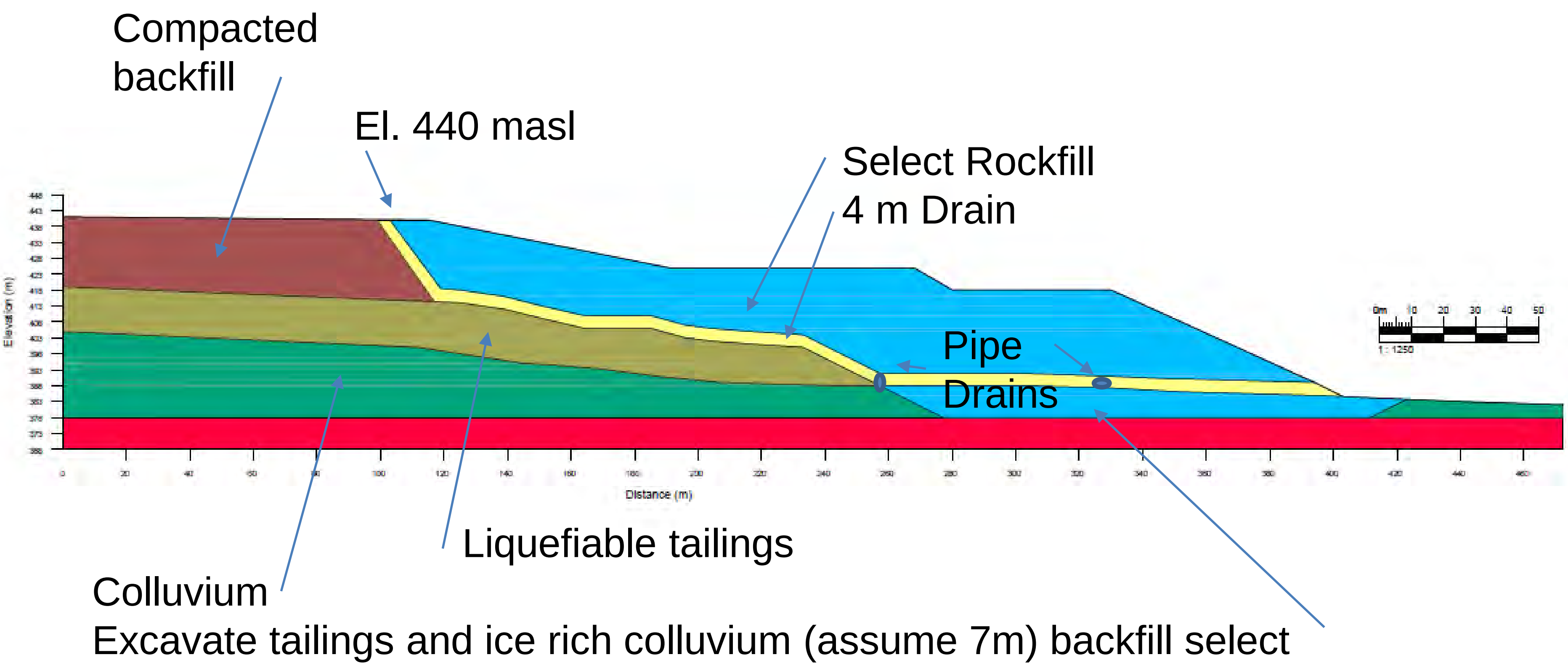
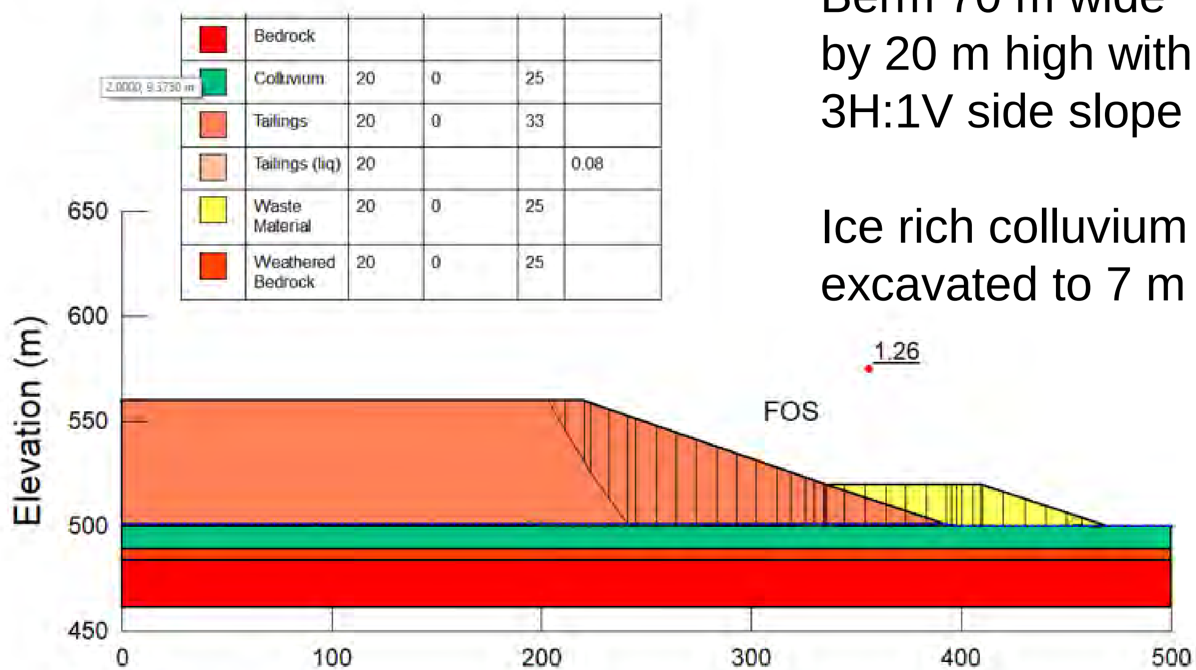


Figure 2

Generic N-S Side slope – 60 m high

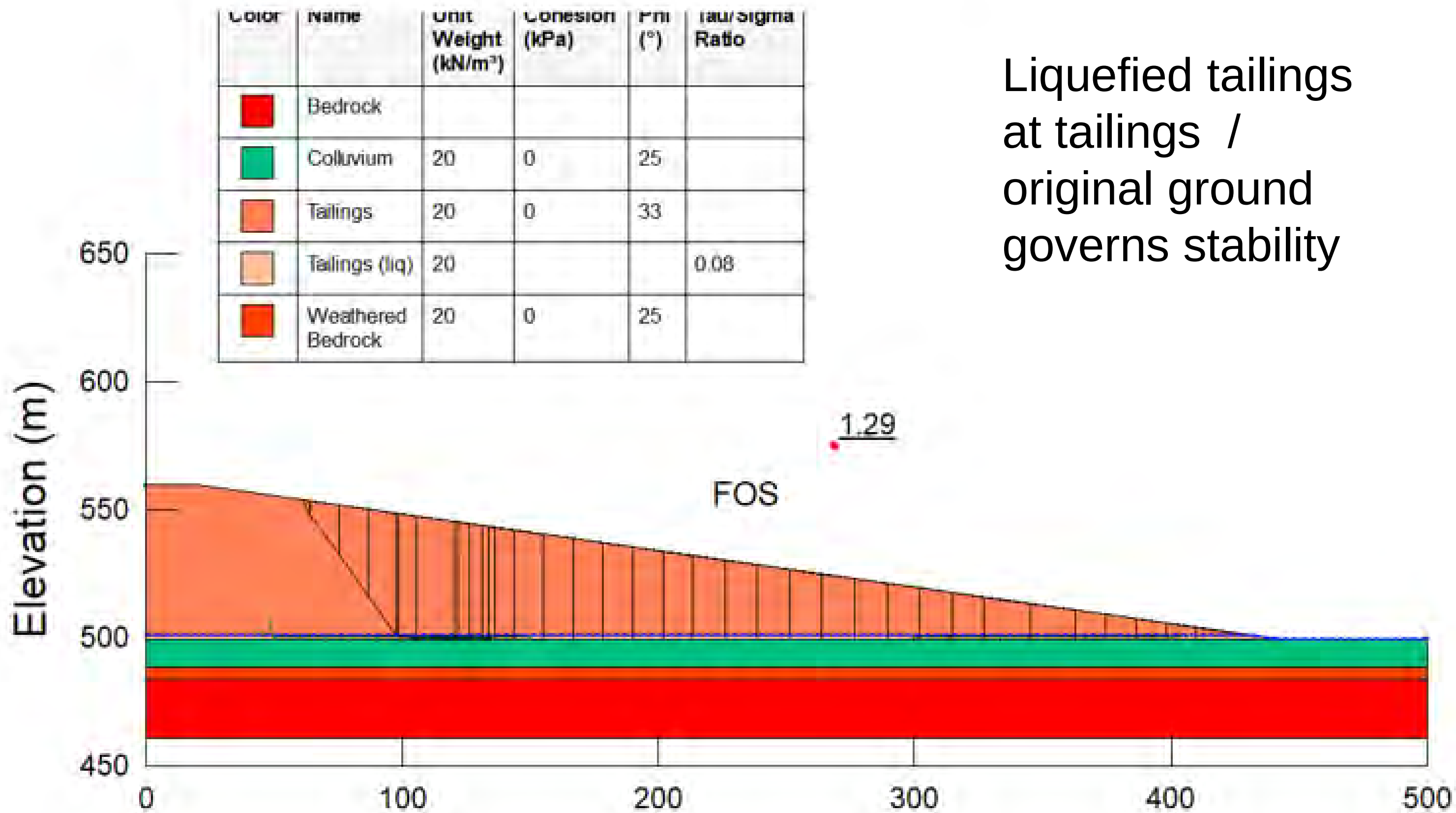


Berm 70 m wide by 20 m high with 3H:1V side slope

Ice rich colluvium excavated to 7 m

Figure 3

Generic N-S Side slope – 60 m high
Slope cut to 7H:1V



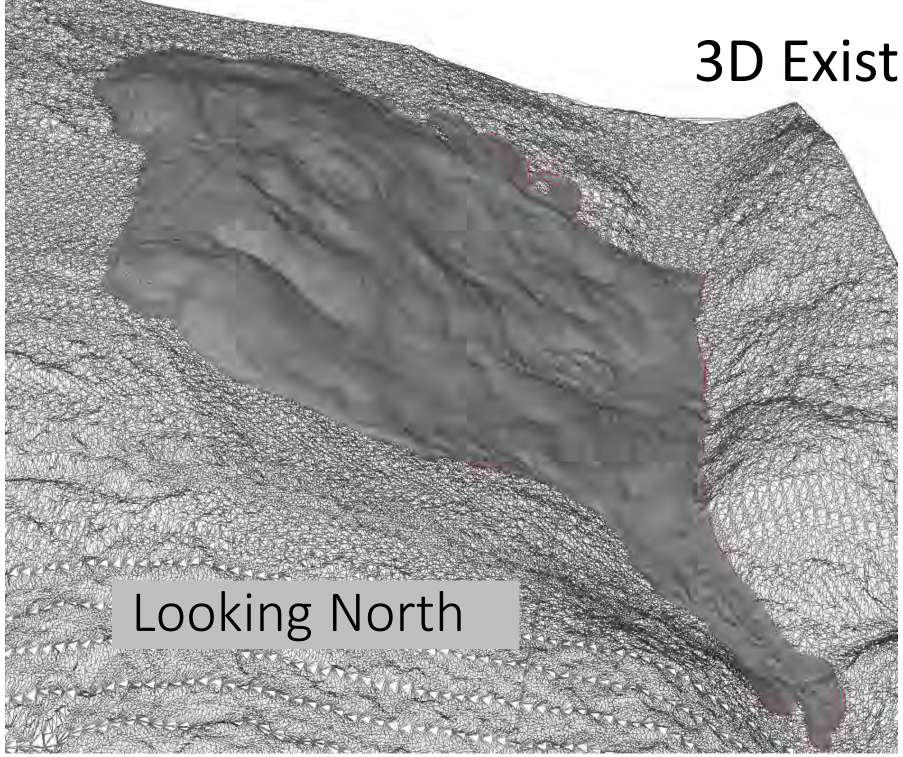
Liquefied tailings at tailings / original ground governs stability

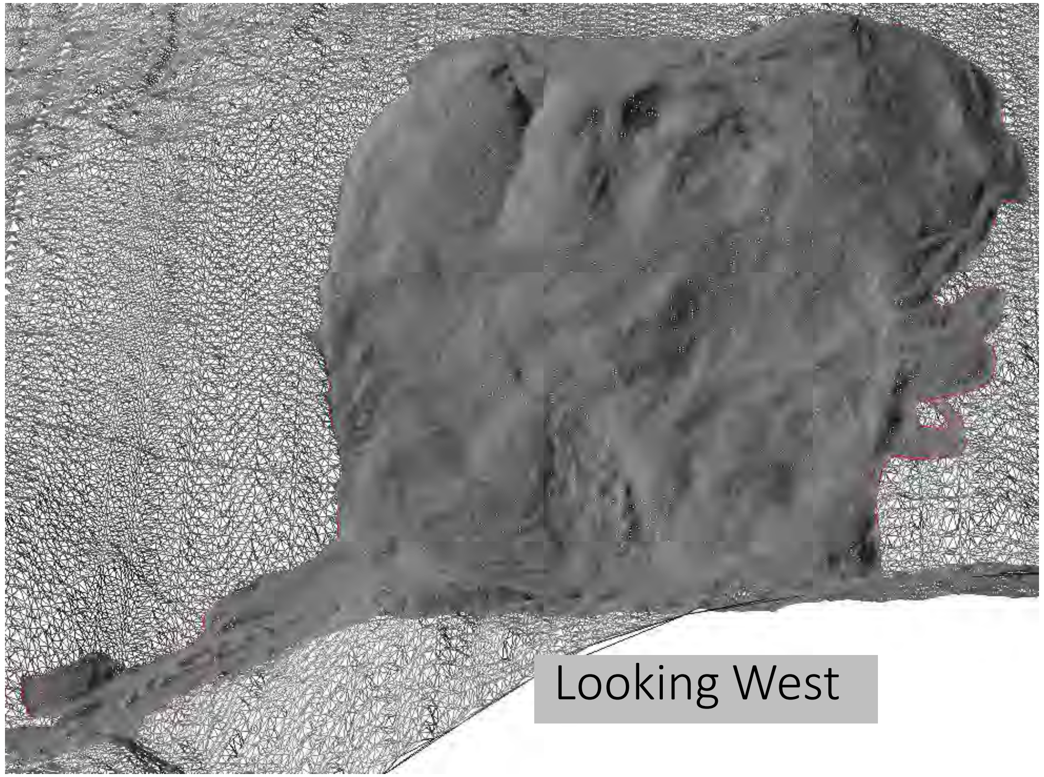
Figure 4

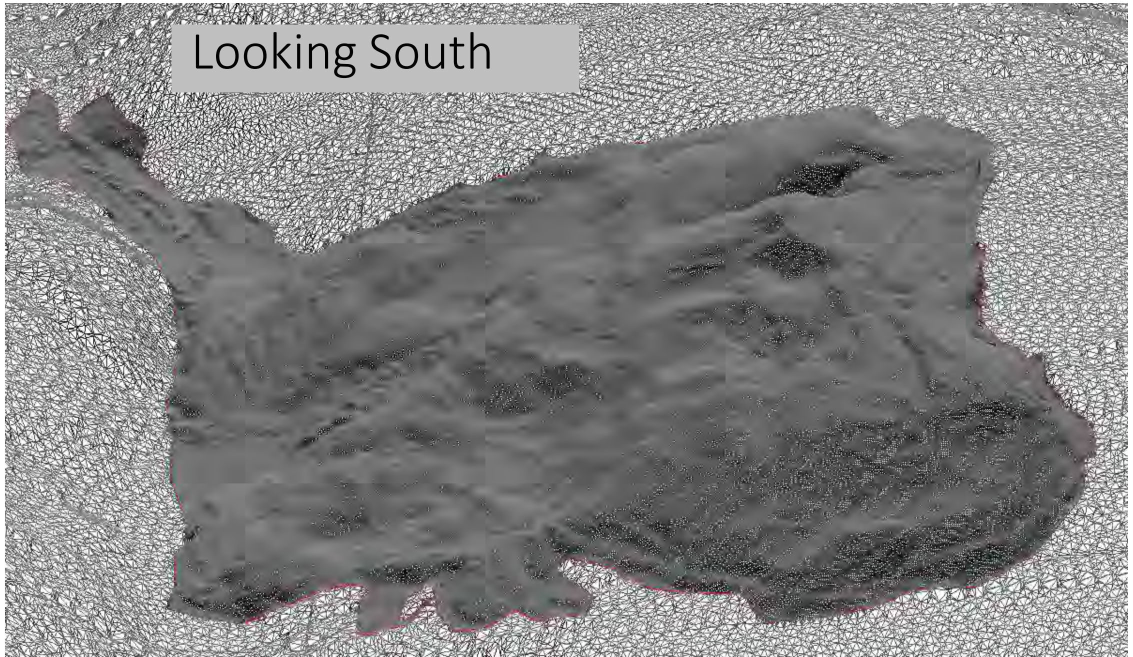
Appendix II

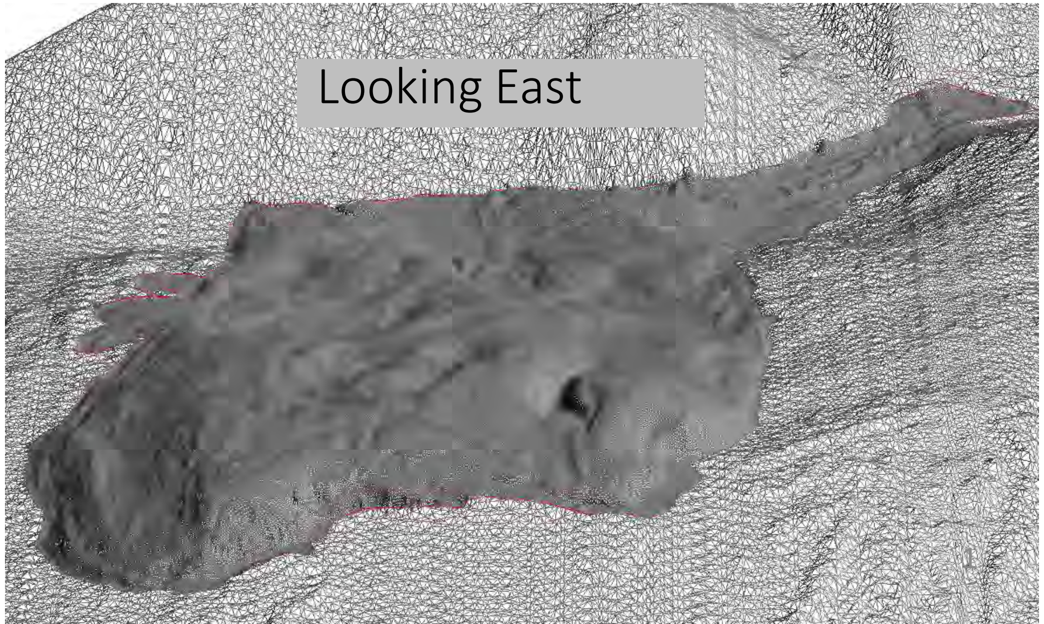
Design Layouts Adopted for the 10% Design

3D Existing Surface

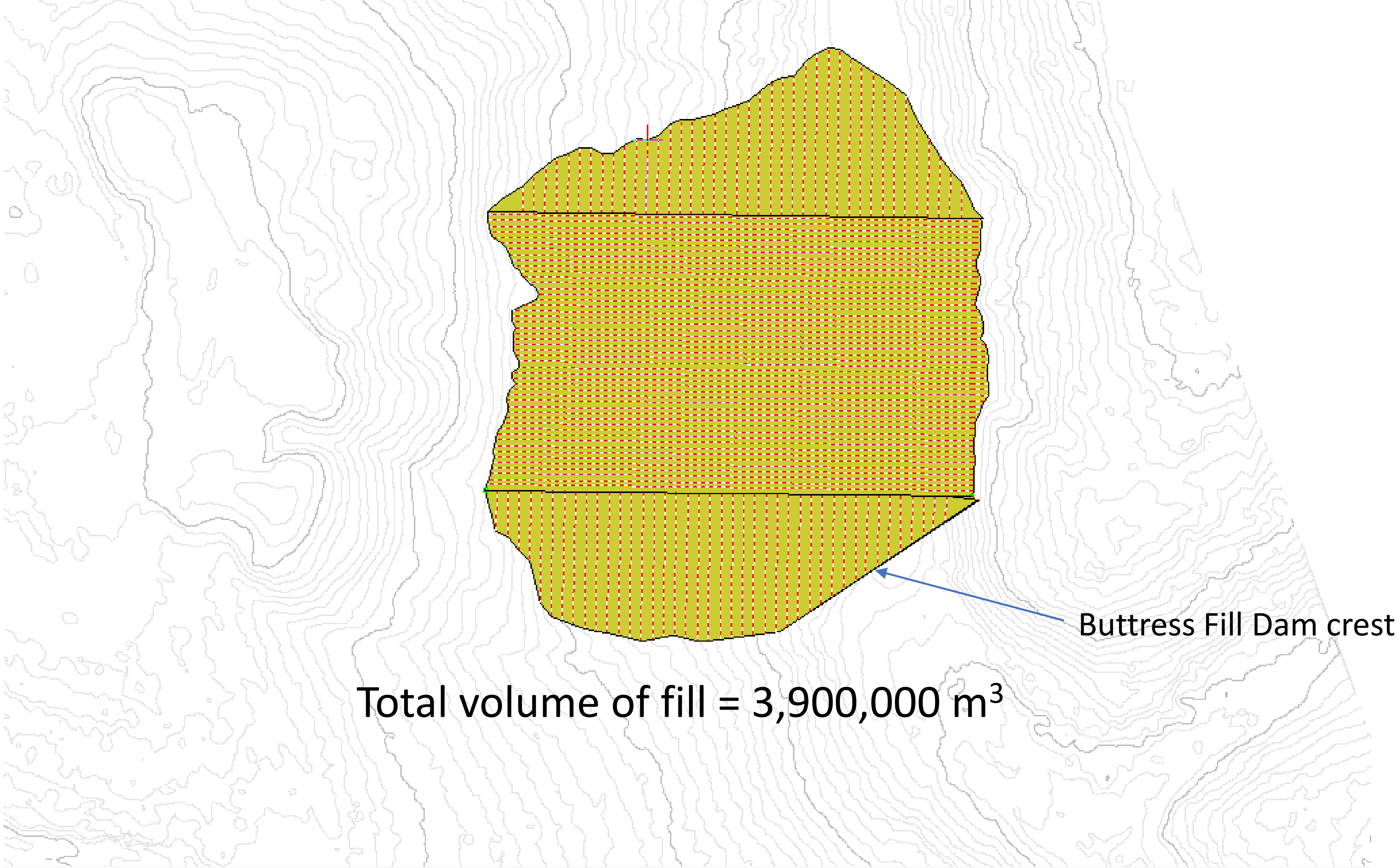








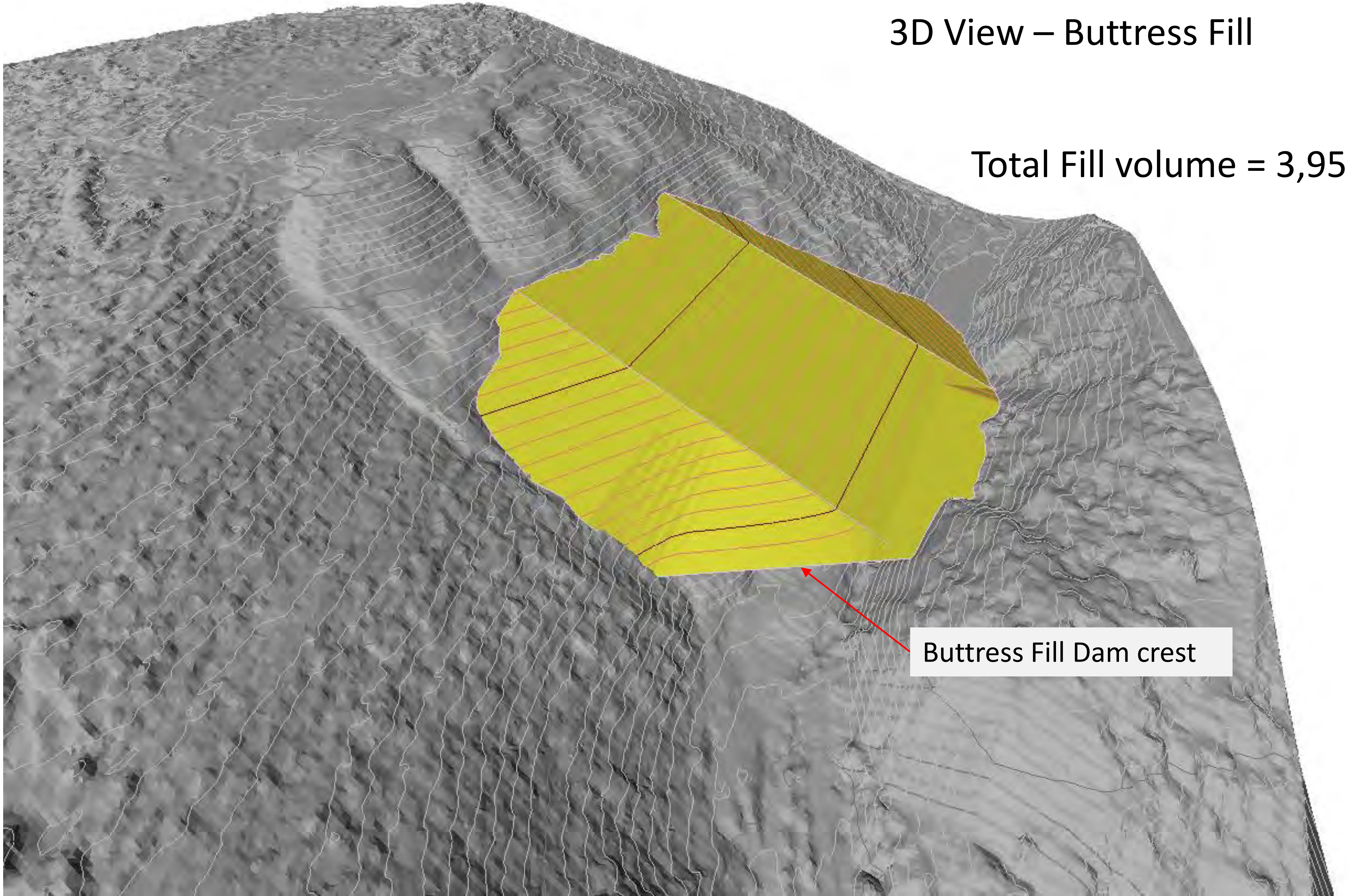
Plan View - Buttress fill at 4.5H:1V slope (E-W direction), side slopes of 3H:1 at north side, and various side slopes from 3H:1 (East) to 7H:1V (West) at the south side.



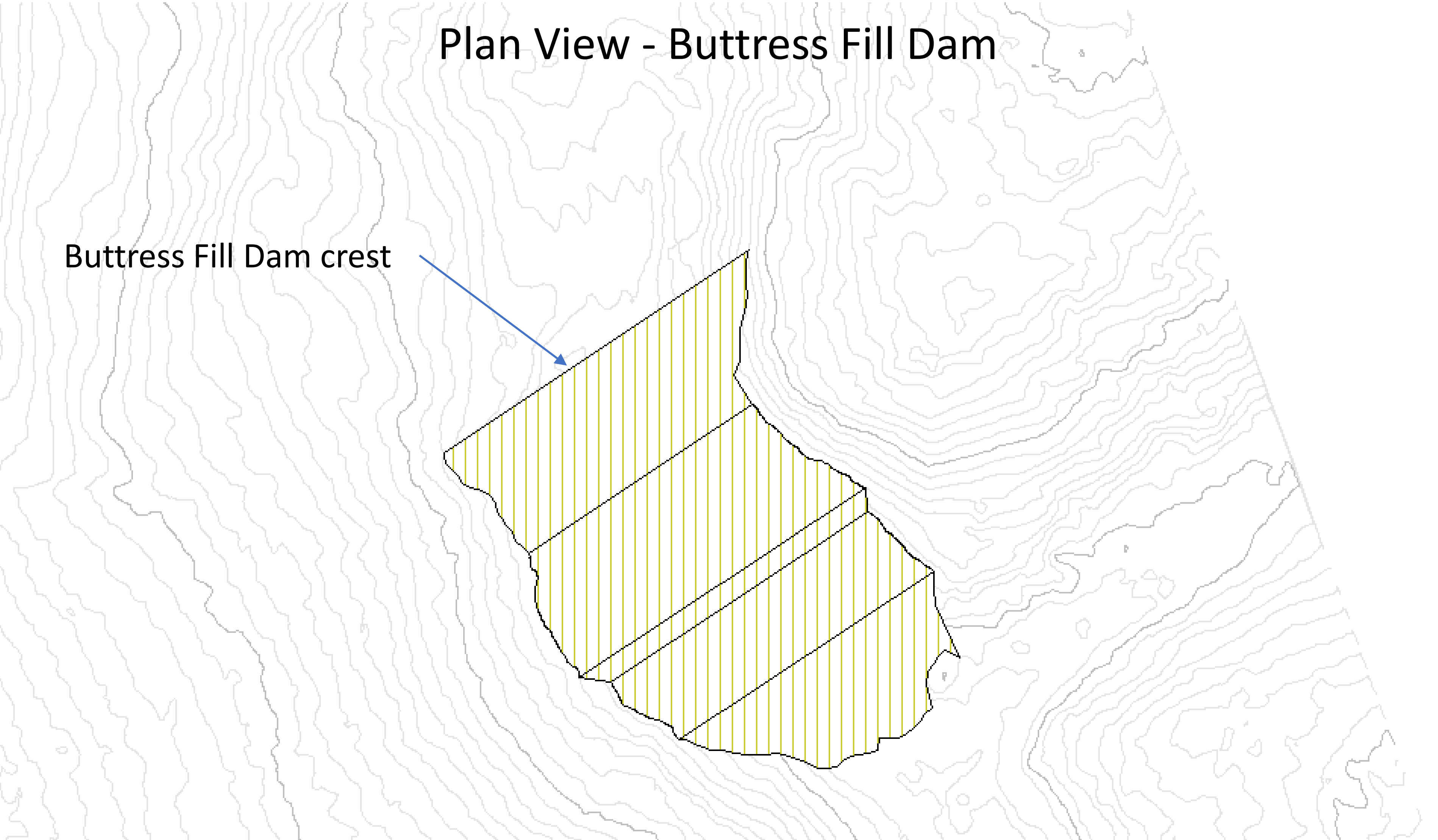
Total volume of fill = 3,900,000 m^3

3D View – Buttress Fill

Total Fill volume = 3,954,000 m^3



Plan View - Buttress Fill Dam



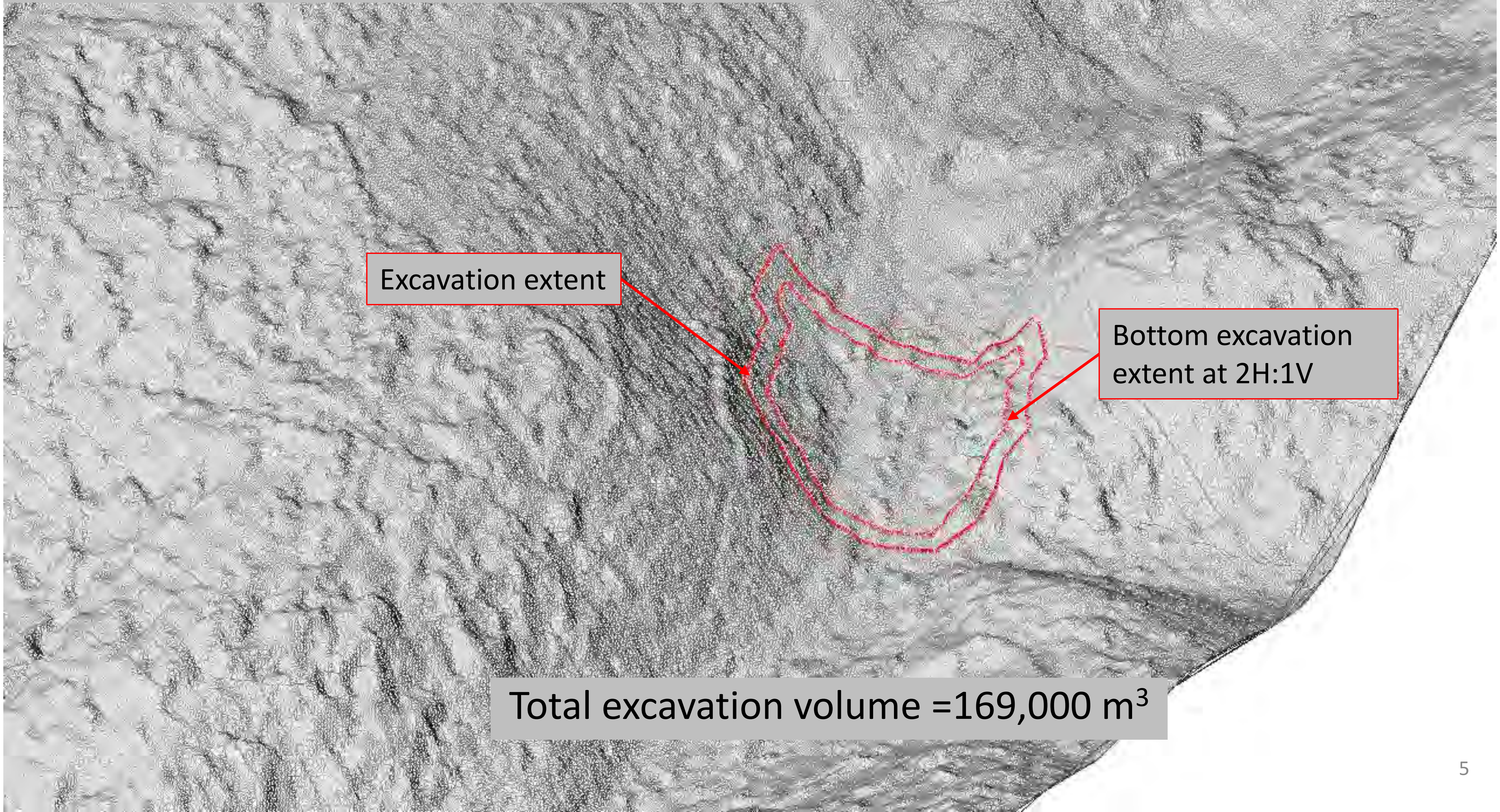
Volume of sub excavation (-7 m with side slope of 2H:1V) = 169,000 m^3

Volume of Chimney and basal drain = 192,000 ^{3}

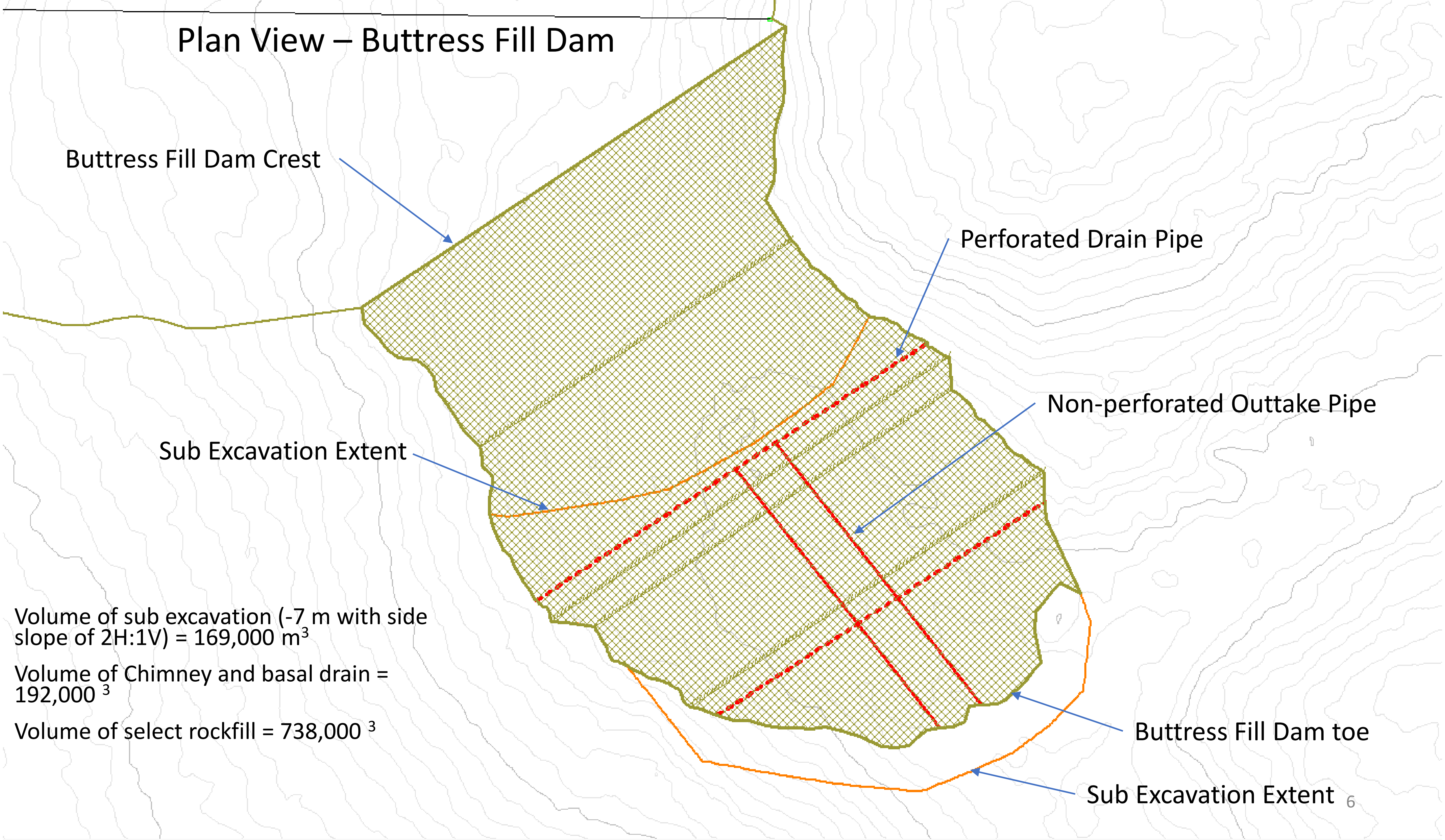
Volume of select rockfill = 738,000 ^{3}

3D View – Wolverine Creek valley bottom

Excavation extent under the Buttress Fill Dam



Plan View – Buttress Fill Dam



Volume of sub excavation (-7 m with side slope of 2H:1V) = 169,000 m^3

Volume of Chimney and basal drain = 192,000 ^{3}

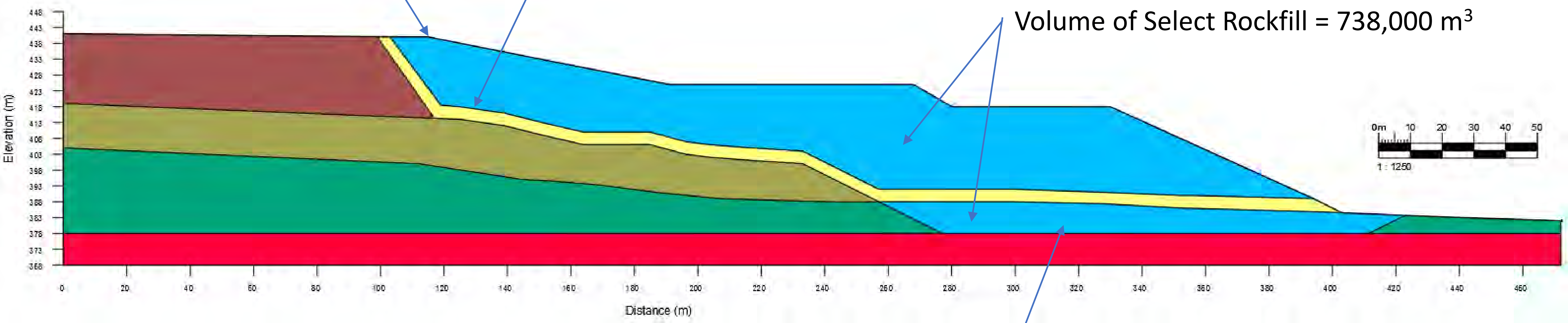
Volume of select rockfill = 738,000 ^{3}

Profile View - Buttress Fill Dam

Buttress Fill Dam crest

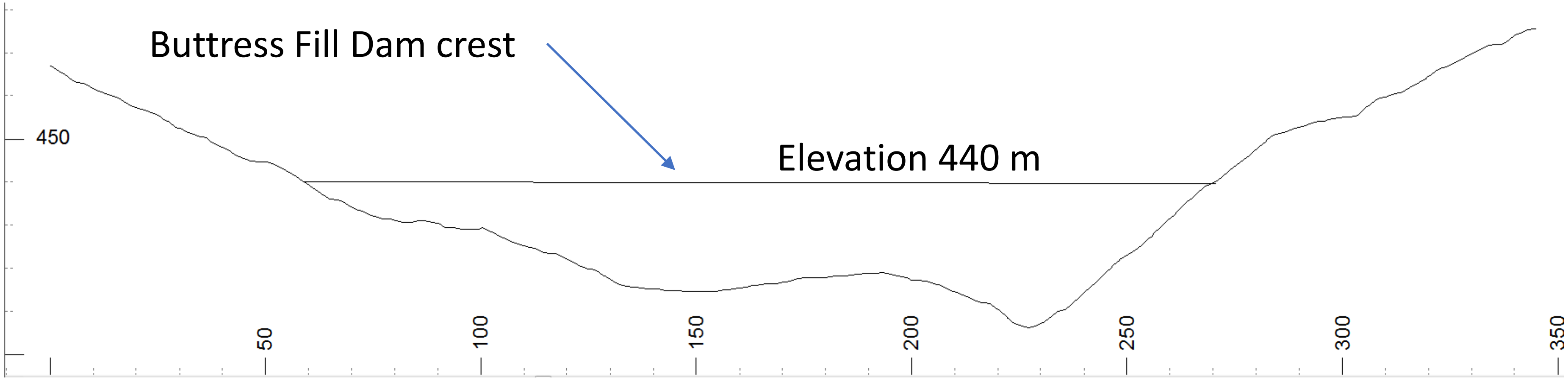
Volume of Chimney and Basal Drain = 192,000 m^3

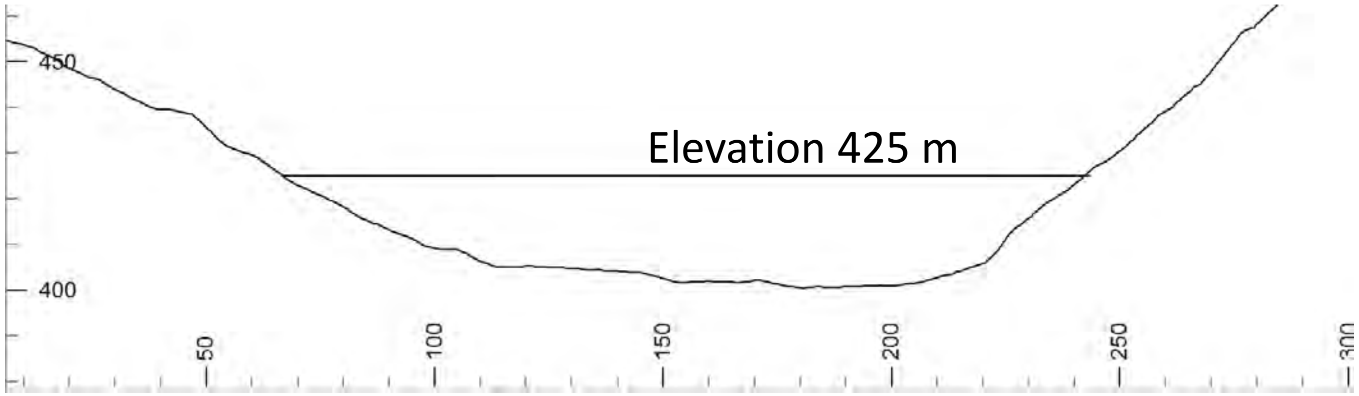
Volume of Select Rockfill = 738,000 m^3



Volume of sub excavation = 169,000 m^3

Cross Section View - Buttress Fill Dam



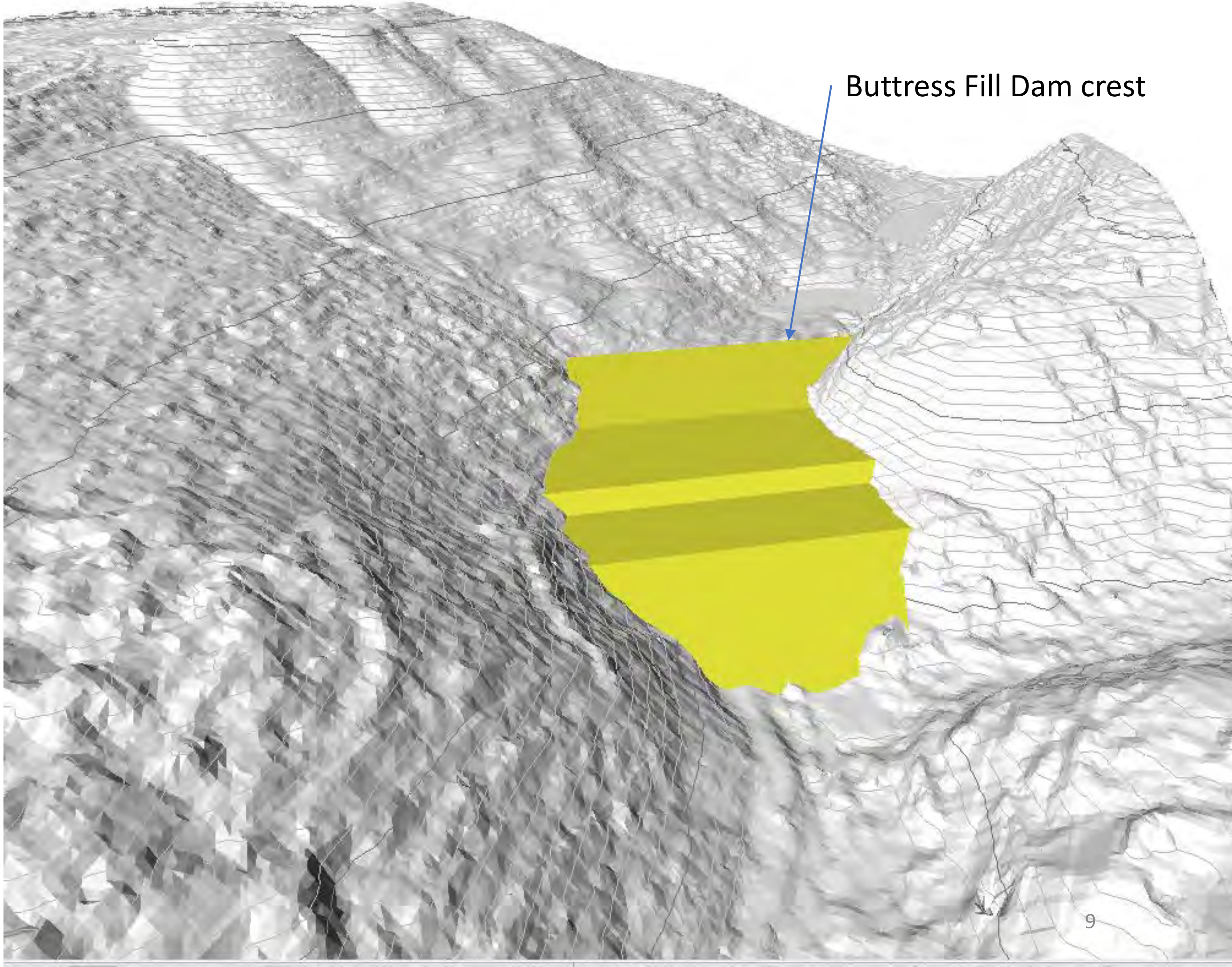


3D View - Buttress Fill Dam

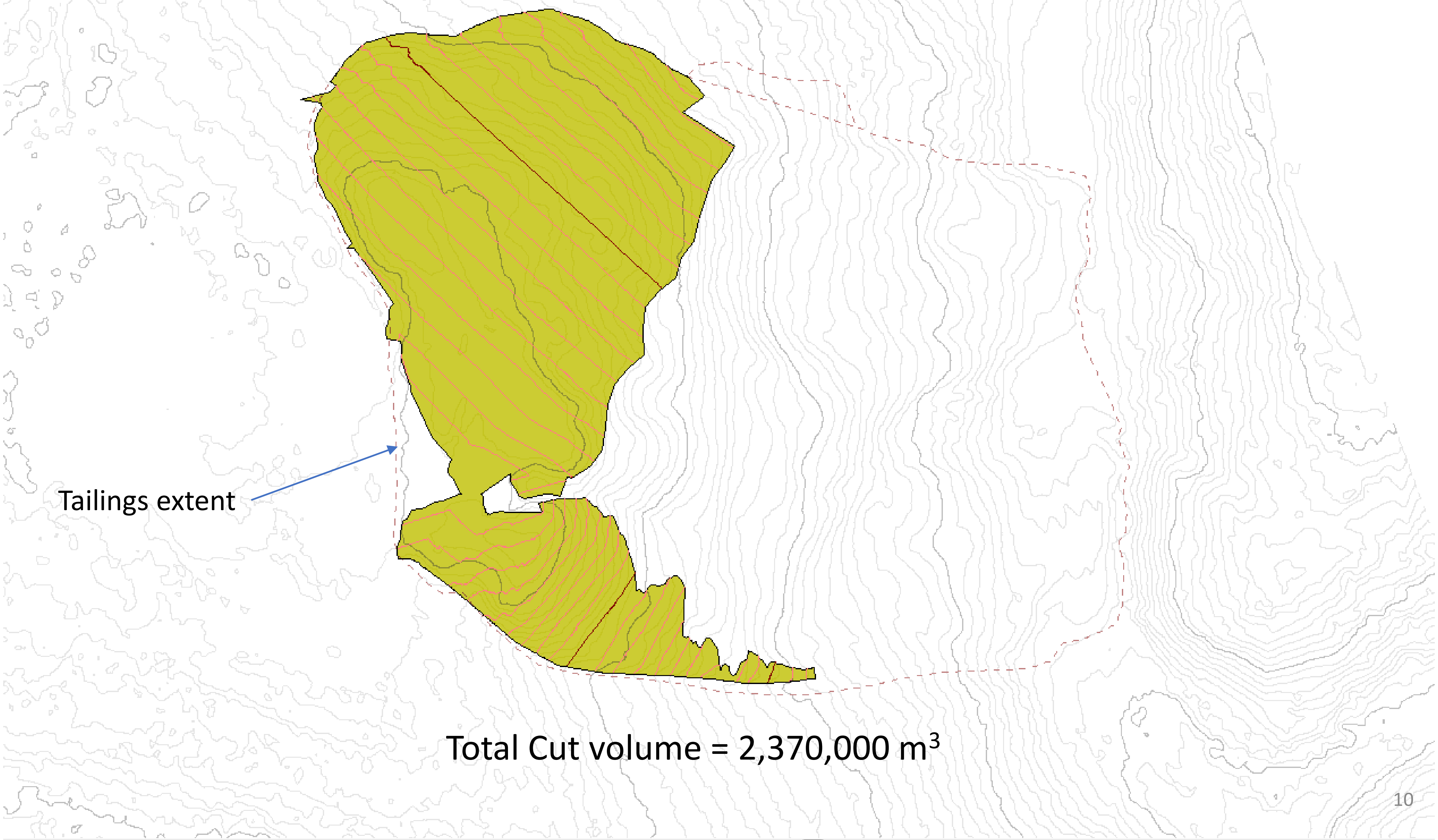
Volume of sub excavation (-7 m with side slope of 2H:1V) = 169,000 m^3

Volume of Chimney and basal drain = 192,000 m^3

Volume of select rockfill = 738,000 m^3

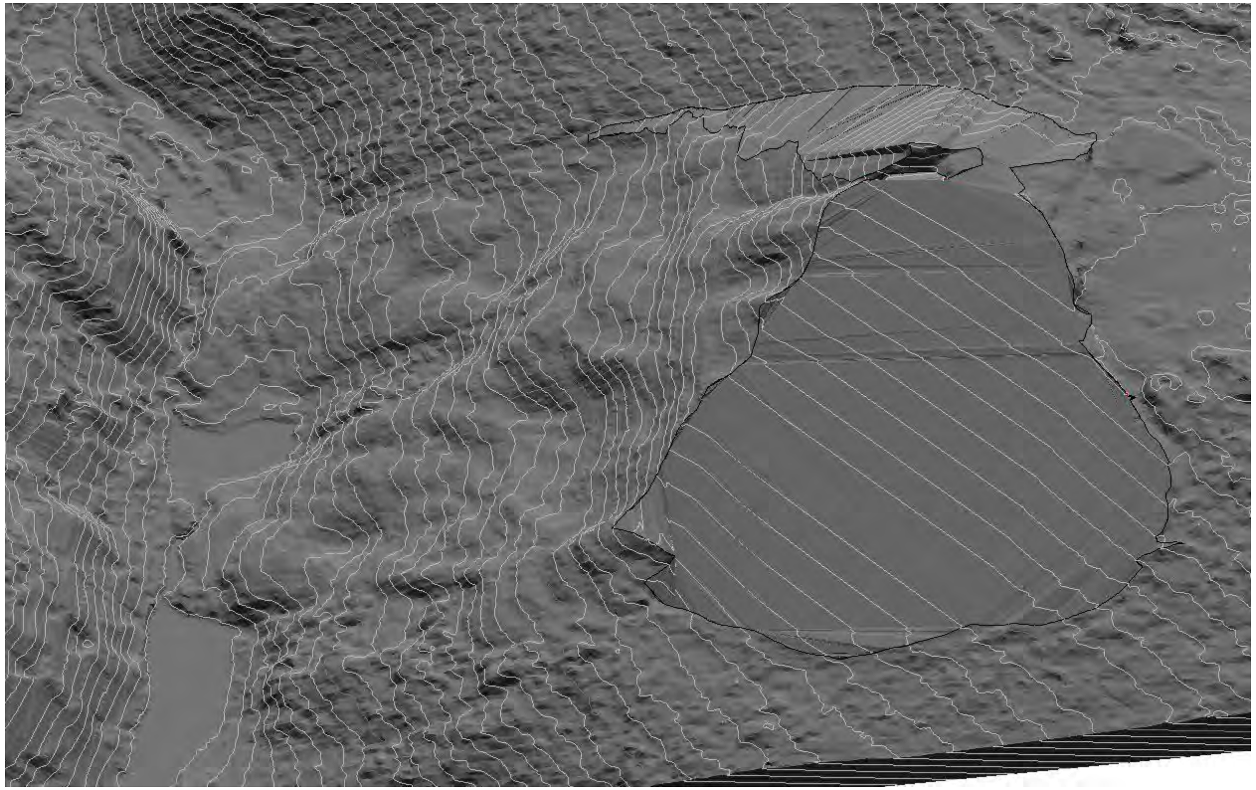


Plan View – Excavated Tailings (7H:1V) extent

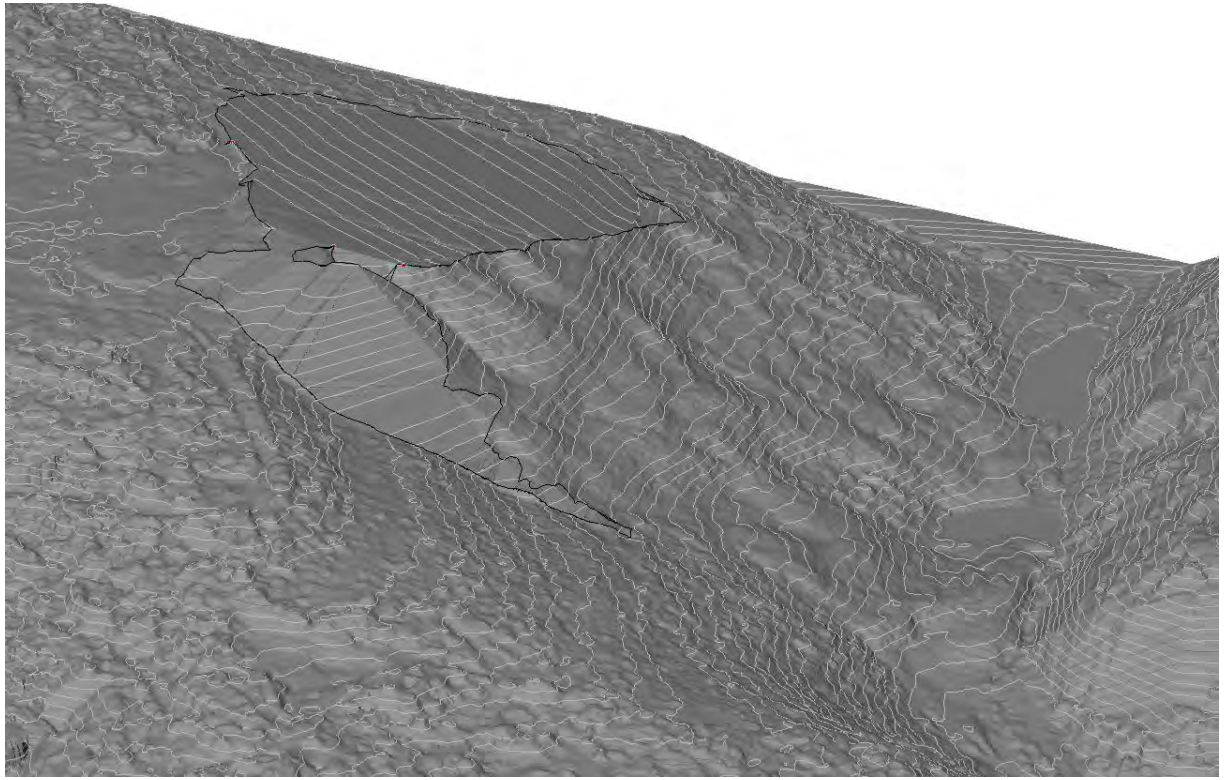


Total Cut volume = 2,370,000 m^3

3D View – Tailings cut extent



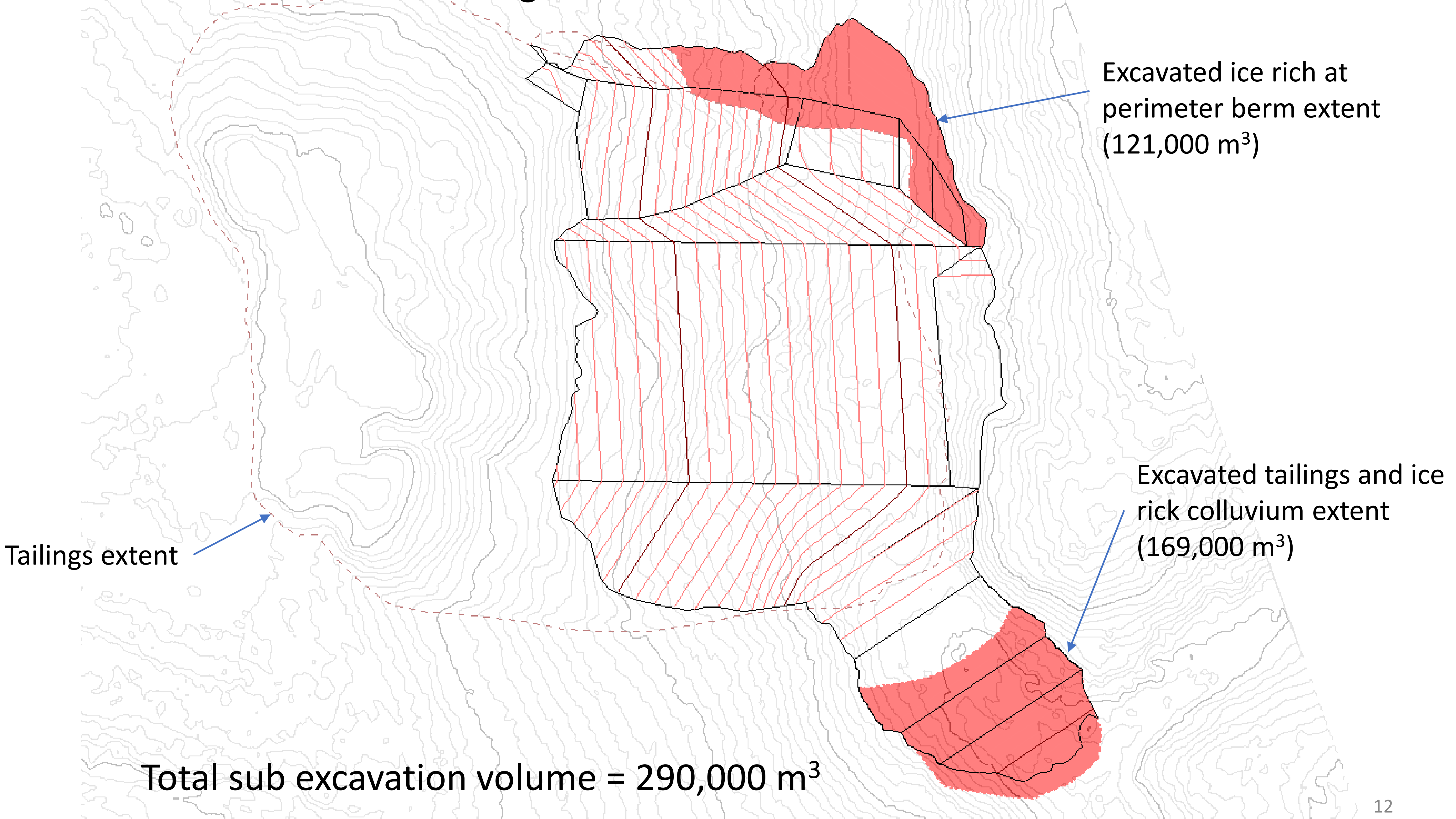
Looking South



Looking North

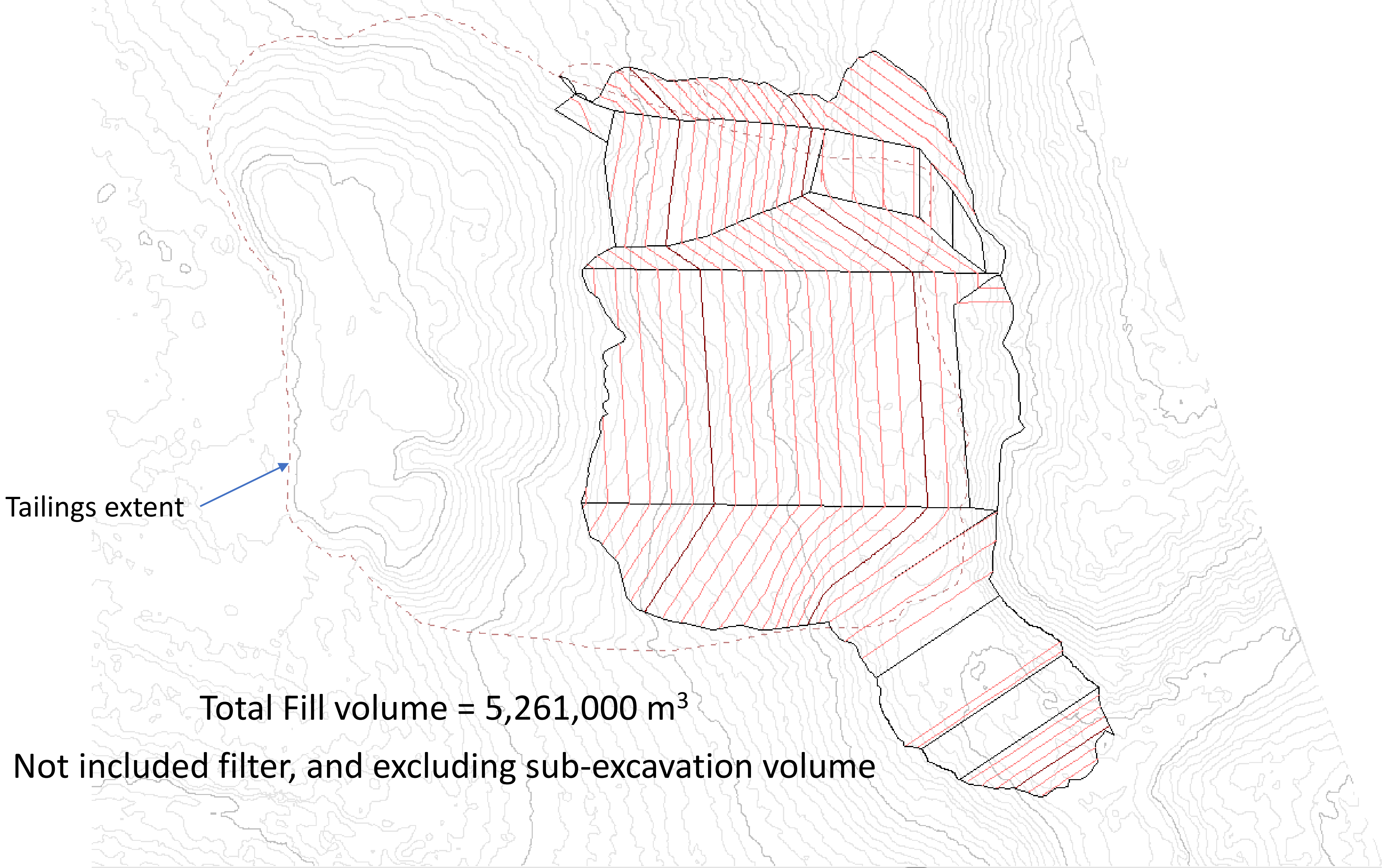
Total Cut volume = 2,370,000 m^3

Plan View – Tailings Buttress Fill Dam and Toe Berm extent



Total sub excavation volume = 290,000 m^3

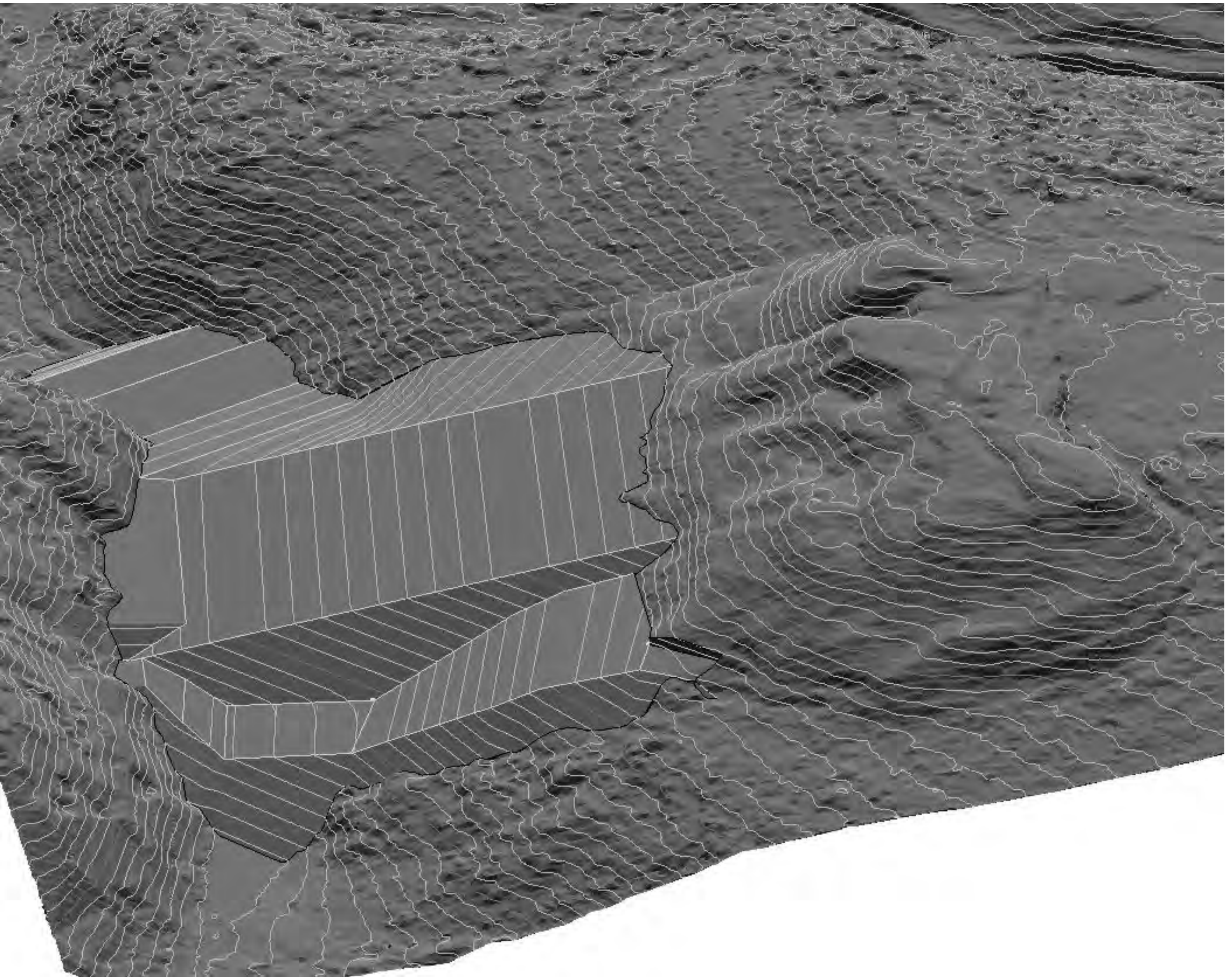
Plan View – Tailings Buttress Fill Dam and Toe Berm extent



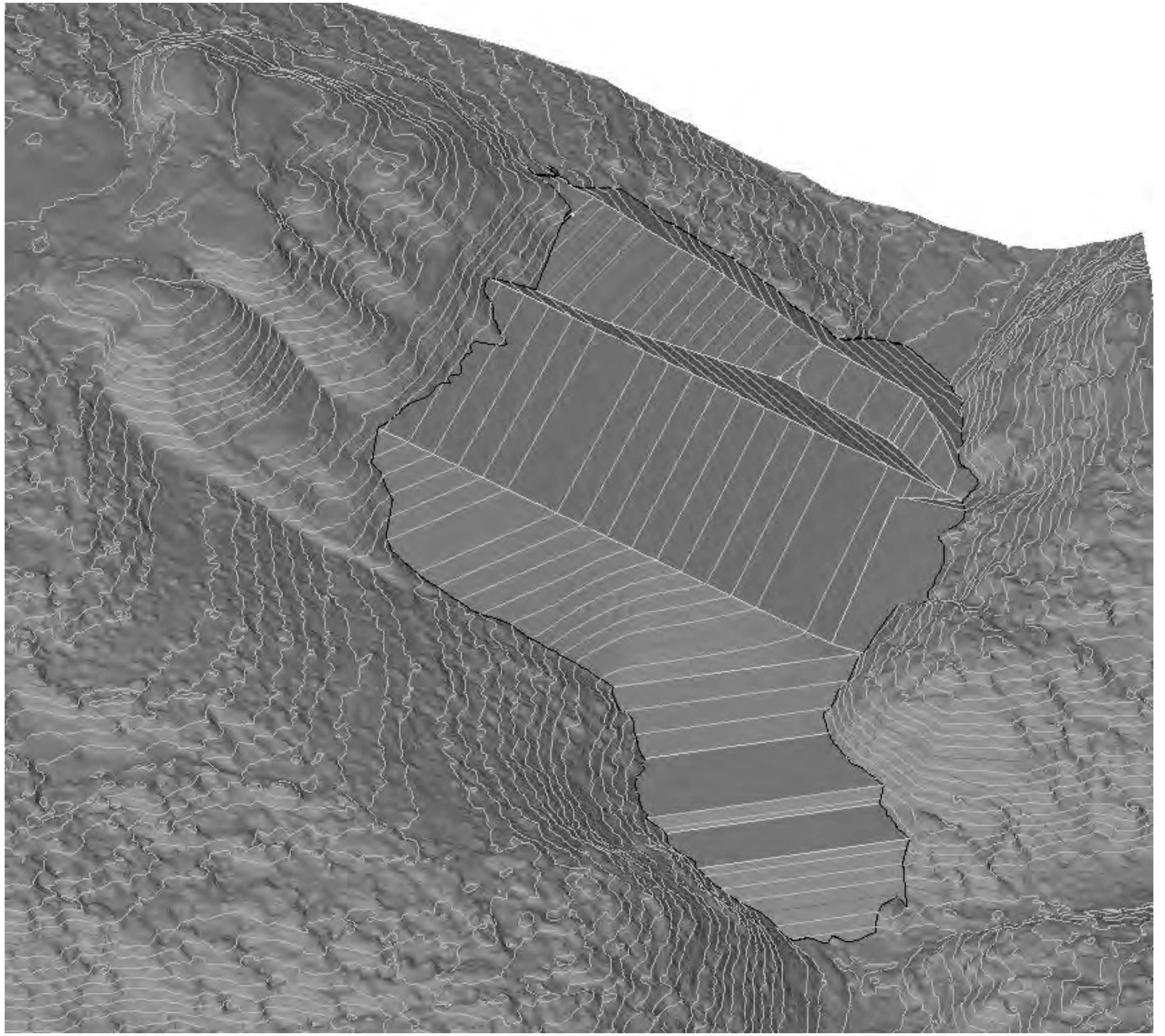
Total Fill volume = 5,261,000 m^3

Not included filter, and excluding sub-excavation volume

3D View – Tailings Buttress Fill Dam and Toe Berm extent



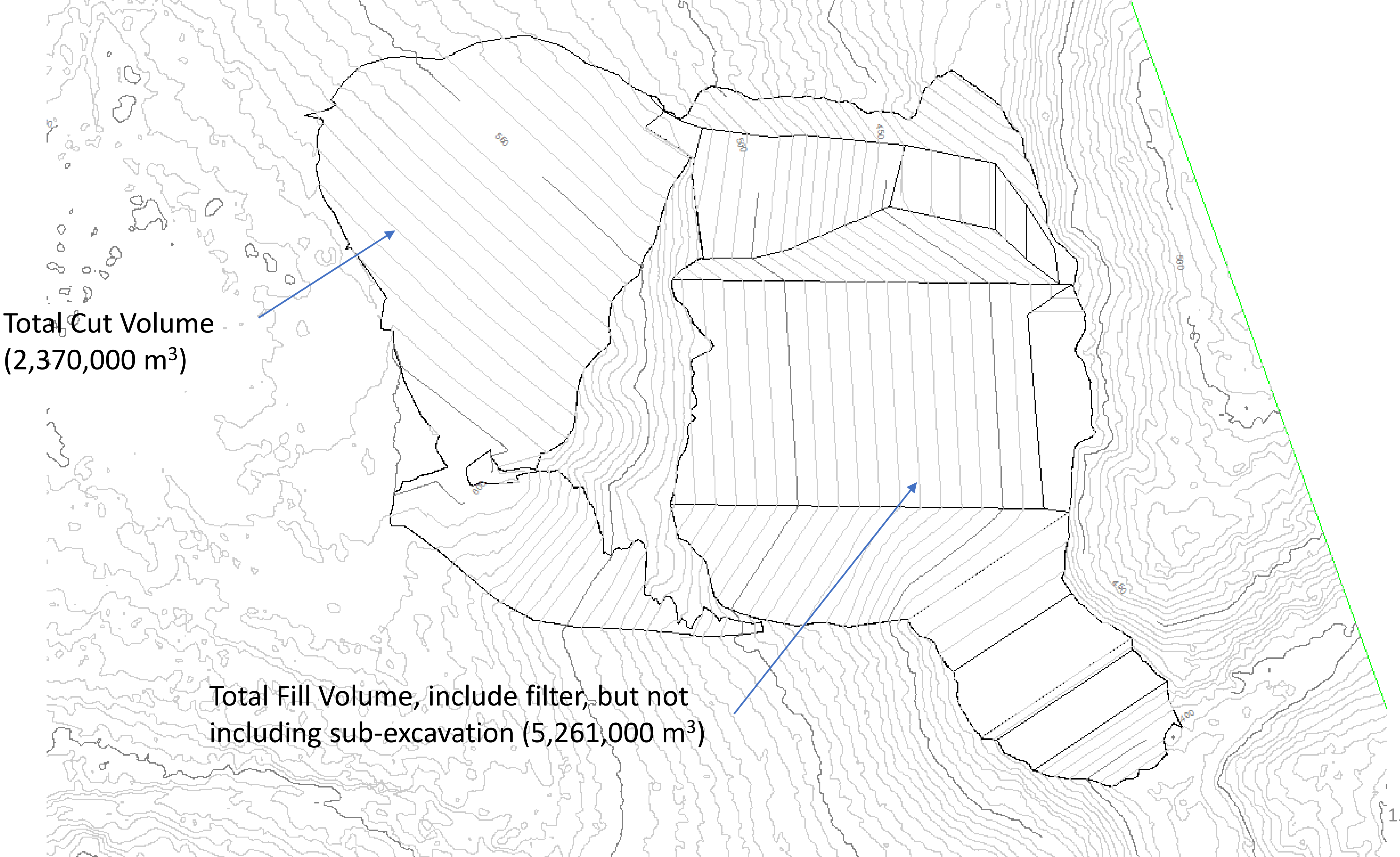
Looking South



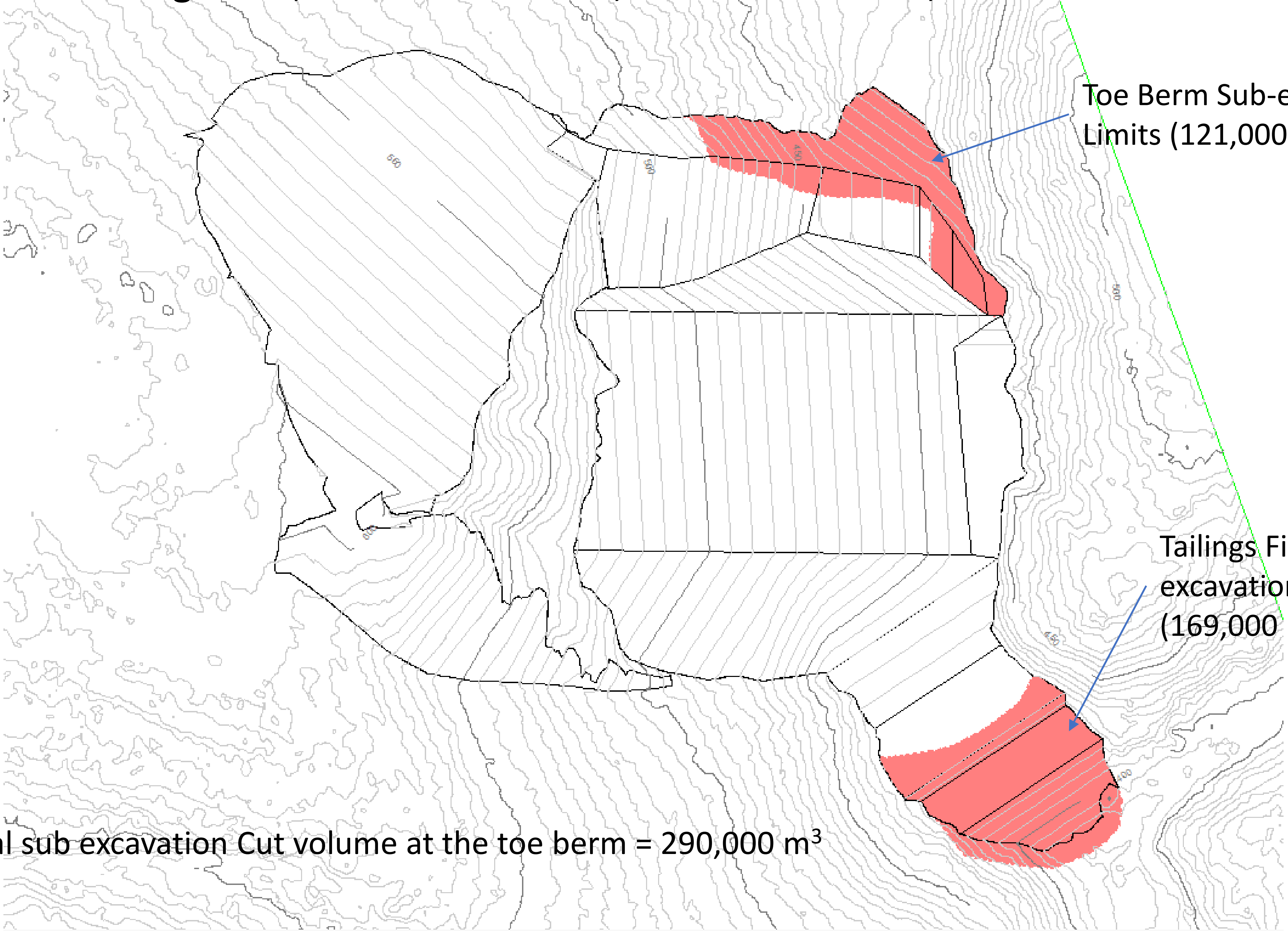
Looking North

Total Fill volume = 5,150,000 m^3 including filters, and excluding sub excavation

Plan View – Tailings cuts, Buttress Fill Dam, and Toe Berm Extent

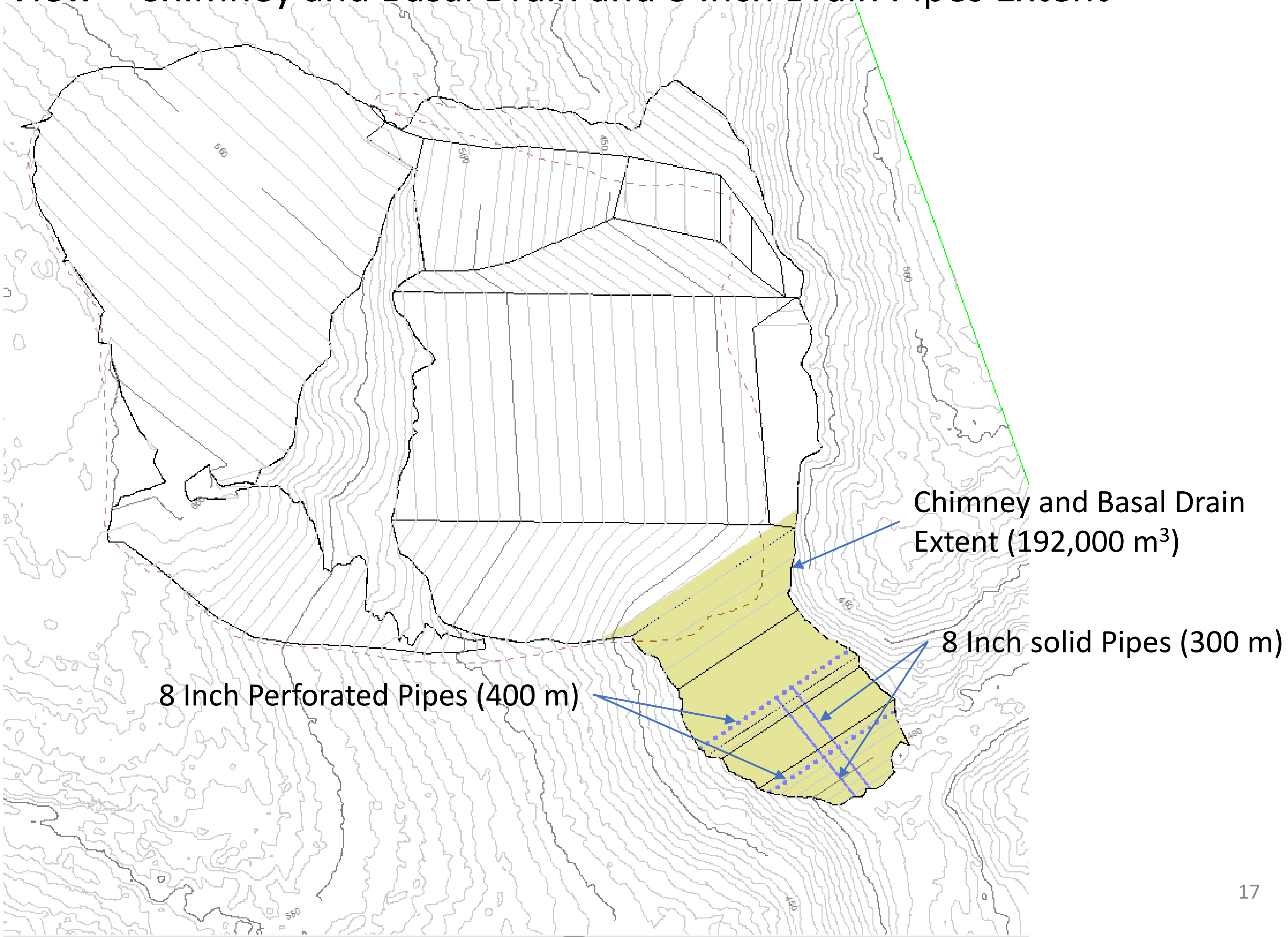


Plan View – Tailings cuts, Buttress Fill Dam, Toe Berm extent, and sub-excavation Extent



Total sub excavation Cut volume at the toe berm = 290,000 m^3

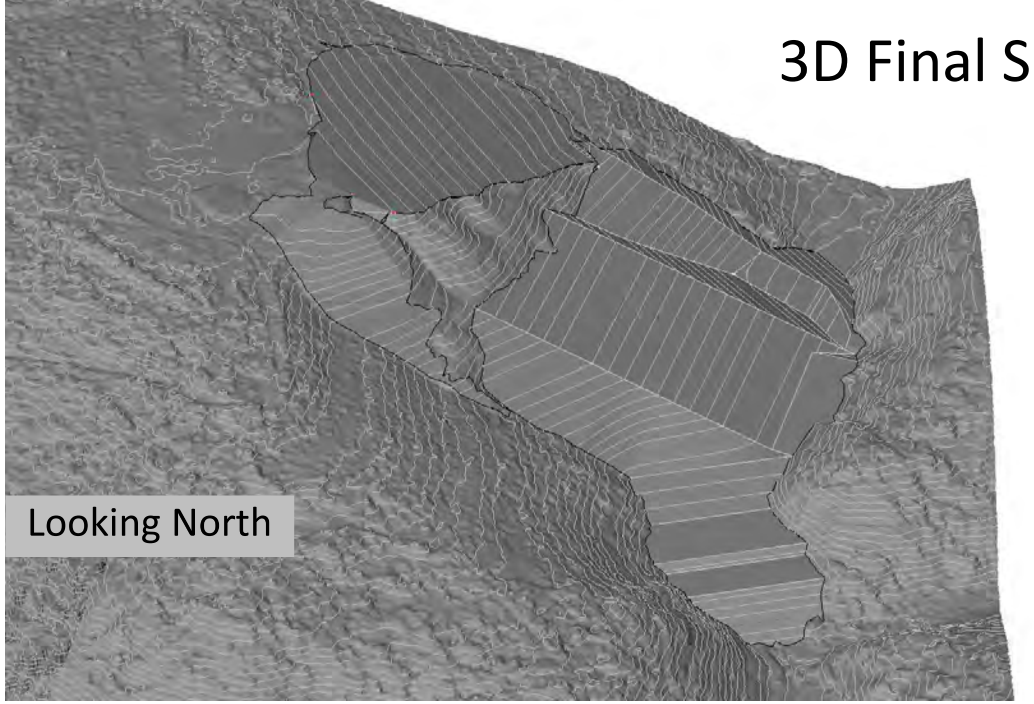
Plan View – Chimney and Basal Drain and 8 Inch Drain Pipes Extent

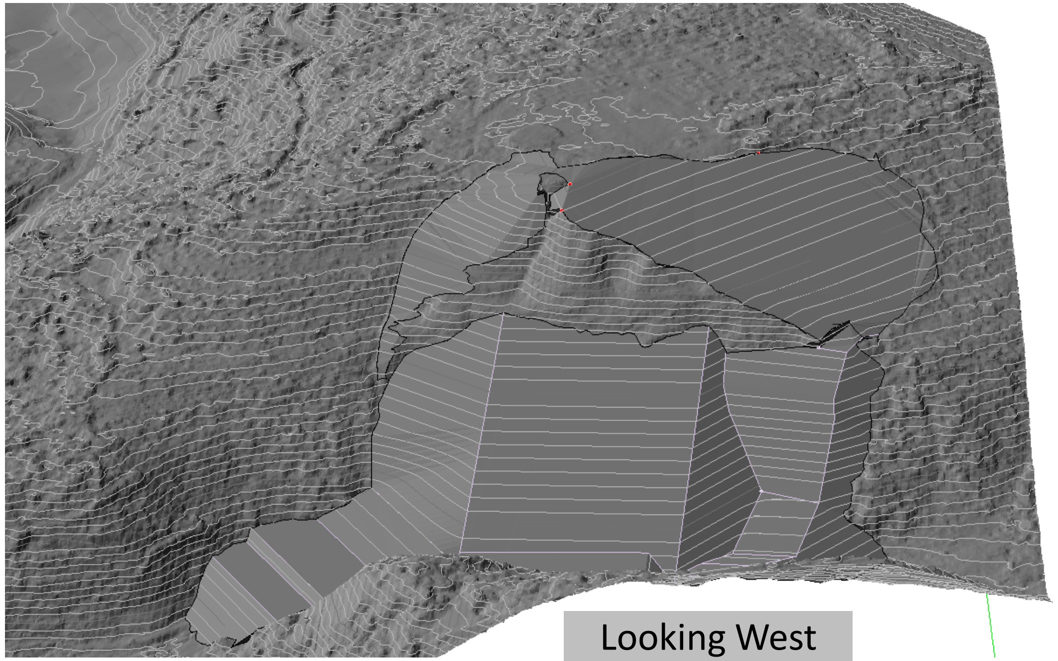


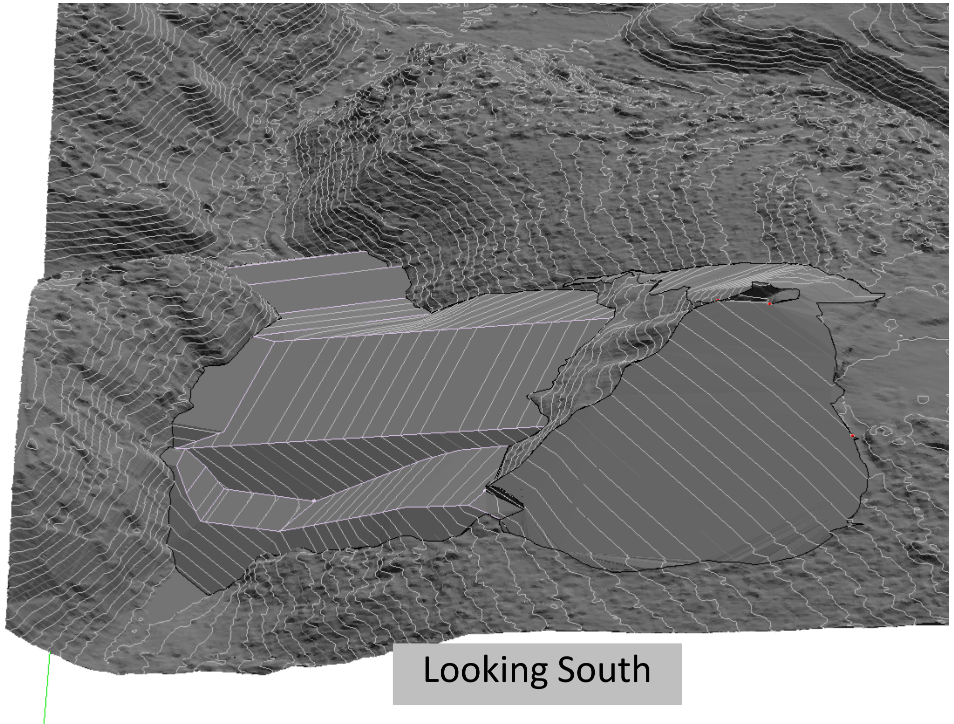
Quantity Summary Table

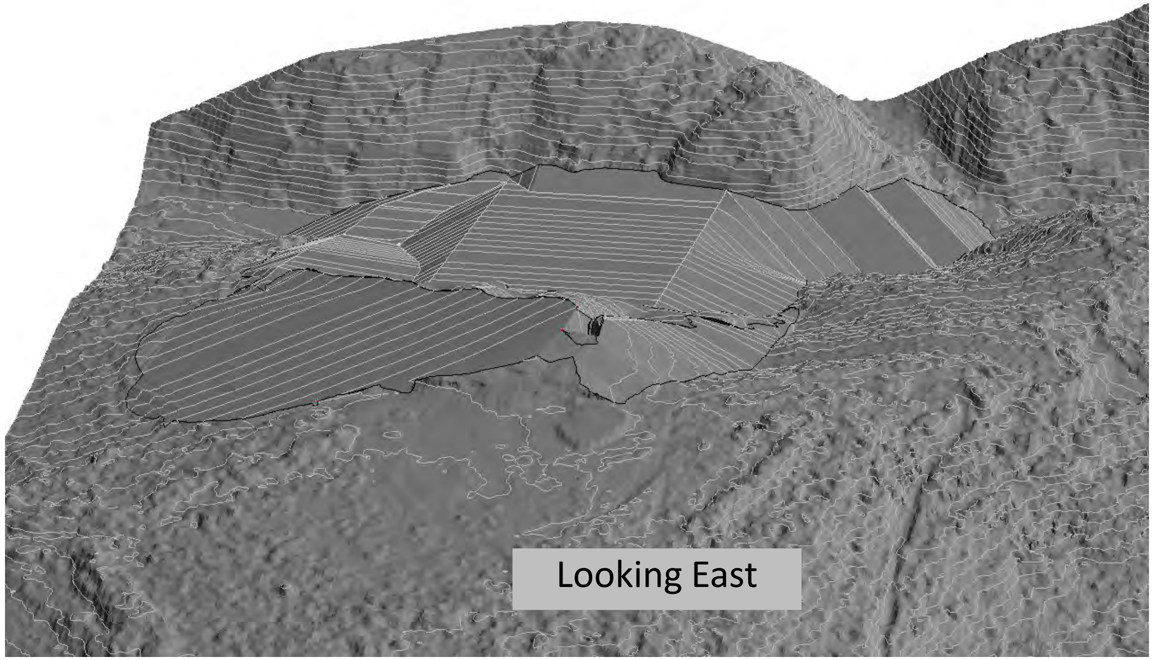
	Components	Length (m)	Surface Area (m^2)	Volume (m^3)
TAILINGS DUMP	Overall Tailings Volume	-	448,000	7,688,000
	Main Buttress Fill Volume (4.5H:1V)	-	190,000	3,954,000
	Excavated tailing (7H:1V)	-	168,000	2,370,000
	Sub-Excavation Volume Perimeter Berm (2H:1V)	-	24,000	121,000
	Compacted Granular Fill (Berm)	-	50,000	550,000
	1 m Capping over all tailings	-	358,000	358,000
BUTTRESS FILL DAM	Excavated Tailings and Ice Rich Culluvium Volume	-	29,000	169,000
	Select Rockfill Shell and Backfill Volume	-	53,000	738,000
	Chimney and Basal Drain Volume	-	52,000	192,000
	8 inch Perforated pipes	400	-	-
	8 inch Solid pipe length	300	-	-

3D Final Surface









Appendix I
Limitations

Limitations

The work performed in the preparation of this report and the conclusions presented herein are subject to the following:

a) The contract between Wood and the Client, including any subsequent written amendment or Change Order dully signed by the parties (hereinafter together referred as the "Contract");

b) Any and all time, budgetary, access and/or site disturbance, risk management preferences, constraints or restrictions as described in the contract, in this report, or in any subsequent communication sent by Wood to the Client in connection to the Contract; and

c) The limitations stated herein.

2. **Standard of care:** Wood has prepared this report in a manner consistent with the level of skill and are ordinarily exercised by reputable members of Wood's profession, practicing in the same or similar locality at the time of performance, and subject to the time limits and physical constraints applicable to the scope of work, and terms and conditions for this assignment. No other warranty, guaranty, or representation, expressed or implied, is made or intended in this report, or in any other communication (oral or written) related to this project. The same are specifically disclaimed, including the implied warranties of merchantability and fitness for a particular purpose.

3. **Limited locations:** The information contained in this report is restricted to the site and structures evaluated by Wood and to the topics specifically discussed in it, and is not applicable to any other aspects, areas, or locations.

4. **Information utilized:** The information, conclusions and estimates contained in this report are based exclusively on: i) information available at the time of preparation, ii) the accuracy and completeness of data supplied by the Client or by third parties as instructed by the Client, and iii) the assumptions, conditions, and qualifications/limitations set forth in this report.

5. **Accuracy of information:** No attempt has been made to verify the accuracy of any information provided by the Client or third parties, except as specifically stated in this report (hereinafter "Supplied Data"). Wood cannot be held responsible for any loss or damage, of either contractual or extra-contractual nature, resulting from conclusions that are based upon reliance on the Supplied Data.

6. **Report interpretation:** This report must be read and interpreted in its entirety, as some sections could be inaccurately interpreted when taken individually or out-of-context. The contents of this report are based upon the conditions known and information provided as of the date of preparation. The text of the final version of this report supersedes any other previous versions produced by Wood.

7. **No legal representations:** Wood makes no representations whatsoever concerning the legal significance of its findings, or as to other legal matters touched on in this report, including but not limited to, ownership of any property, or the application of any law to the facts set forth herein. With respect to regulatory compliance issues, regulatory statutes are subject to interpretation and change. Such interpretations and regulatory changes should be reviewed with legal counsel.

8. **Decrease in property value:** Wood shall not be responsible for any decrease, real or perceived, of the property or site's value or failure to complete a transaction, as a consequence of the information contained in this report.

9. **No third-party reliance:** This report is for the sole use of the party to whom it is addressed unless expressly stated otherwise in the report or Contract. Any use or reproduction which any third party makes of the report, in whole or in part, or any reliance thereon or decisions made based on any information or conclusions in the report is the sole responsibility of such third party. Wood does not represent or warrant the accuracy, completeness, merchantability, fitness for purpose or usefulness of this document, or any information contained in this document, for use or consideration by any third party. Wood accepts no responsibility whatsoever for damages or loss of any nature or kind suffered by any such third party as a result of actions taken or not taken or decisions made in reliance on this report or anything set out therein. including without limitation, any indirect, special, incidental, punitive, or consequential loss, liability or damage of any kind.

10. **Assumptions**: Where design recommendations are given in this report, they apply only if the project contemplated by the Client is constructed substantially in accordance with the details stated in this report. It is the sole responsibility of the Client to provide to Wood changes made in the project, including but not limited to, details in the design, conditions, engineering, or construction that could in any manner whatsoever impact the validity of the recommendations made in the report. Wood shall be entitled to additional compensation from Client to review and assess the effect of such changes to the project.

11. **Time dependence**: If the project contemplated by the Client is not undertaken within a period of 18 months following the submission of this report, or within the time frame understood by Wood to be contemplated by the Client at the commencement of Wood's assignment, and/or, if any changes are made, for example, to the elevation, design or nature of any development on the site, its size and configuration, the location of any development on the site and its orientation, the use of the site, performance criteria and the location of any physical infrastructure, the conclusions and recommendations presented herein should not be considered valid unless the impact of the said changes is evaluated by Wood, and the conclusions of the report are amended or are validated in writing accordingly.

    Advancements in the practice of geotechnical engineering, engineering geology and hydrogeology and changes in applicable regulations, standards, codes or criteria could impact the contents of the report, in which case, a supplementary report may be required. The requirements for such a review remain the sole responsibility of the Client or their agents.

    Wood will not be liable to update or revise the report to take into account any events or emergent circumstances or facts occurring or becoming apparent after the date of the report.

12. **Limitations of visual inspections:** Where conclusions and recommendations are given based on a visual inspection conducted by Wood, they relate only to the natural or man-made structures, slopes, etc. inspected at the time the site visit was performed. These conclusions cannot and are not extended to include those portions of the site or structures, which were not reasonably available, in Wood's opinion, for direct observation.

13. **Limitations of site investigations**: Site exploration identifies specific subsurface conditions only at those points from which samples have been taken and only at the time of the site investigation. Site investigation programs are a professional estimate of the scope of investigation required to provide a general profile of subsurface conditions.

    The data derived from the site investigation program and subsequent laboratory testing are interpreted by trained personnel and extrapolated across the site to form an inferred geological representation and an engineering opinion is rendered about overall subsurface conditions and their likely behaviour with regard to the proposed development. Despite this investigation, conditions between and beyond the borehole/test hole locations may differ from those encountered at the borehole/test hole locations and the actual conditions at the site might differ from those inferred to exist, since no subsurface exploration program, no matter how comprehensive, can reveal all subsurface details and anomalies.

Final sub-surface/bore/profile logs are developed by geotechnical engineers based upon their interpretation of field logs and laboratory evaluation of field samples. Customarily, only the final bore/profile logs are included in geotechnical engineering reports.

Bedrock, soil properties and groundwater conditions can be significantly altered by environmental remediation and/or construction activities such as the use of heavy equipment or machinery, excavation, blasting, pile-driving or draining or other activities conducted either directly on site or on adjacent terrain. These properties can also be indirectly affected by exposure to unfavorable natural events or weather conditions, including freezing, drought, precipitation and snowmelt.

During construction, excavation is frequently undertaken which exposes the actual subsurface and groundwater conditions between and beyond the test locations, which may differ from those encountered at the test locations. It is recommended that Wood be retained during construction to confirm that the subsurface conditions throughout the site do not deviate materially from those encountered at the test locations, that construction work has no negative impact on the geotechnical aspects of the design, to adjust recommendations in accordance with conditions as additional site information is gained, and to deal quickly with geotechnical considerations if they arise.

Interpretations and recommendations presented herein may not be valid if an adequate level of review or inspection by Wood is not provided during construction.

14. **Factors that may affect construction methods, costs and scheduling**: The performance of rock and soil materials during construction is greatly influenced by the means and methods of construction. Where comments are made relating to possible methods of construction, construction costs, construction techniques, sequencing, equipment or scheduling, they are intended only for the guidance of the project design professionals, and those responsible for construction monitoring. The number of test holes may not be sufficient to determine the local underground conditions between test locations that may affect construction costs, construction techniques, sequencing, equipment, scheduling, operational planning, etc.

    Any contractors bidding on or undertaking the works should draw their own conclusions as to how the subsurface and groundwater conditions may affect their work, based on their own investigations and interpretations of the factual soil data, groundwater observations, and other factual information.

15. **Groundwater and Dewatering:** Wood will accept no responsibility for the effects of drainage and/or dewatering measures if Wood has not been specifically consulted and involved in the design and monitoring of the drainage and/or dewatering system.

16. **Environmental and Hazardous Materials Aspects:** Unless otherwise stated, the information contained in this report in no way reflects on the environmental aspects of this project, since this aspect is beyond the Scope of Work and the Contract. Unless expressly included in the Scope of Work, this report specifically excludes the identification or interpretation of environmental conditions such as contamination, hazardous materials, wild life conditions, rare plants or archeology conditions that may affect use or design at the site. This report specifically excludes the investigation, detection, prevention or assessment of conditions that can contribute to moisture, mould or other microbial contaminant growth and/or other moisture related deterioration, such as corrosion, decay, rot in buildings or their surroundings. Any statements in this report or on the boring logs regarding odours, colours, and unusual or suspicious items or conditions are strictly for informational purposes.

17. **Sample Disposal:** Wood will dispose of all uncontaminated soil and rock samples after 30 days following the release of the final geotechnical report. Should the Client request that the samples be retained for a longer time, the Client will be billed for such storage at an agreed upon rate. Contaminated samples of soil, rock or groundwater are the property of the Client, and the Client will be responsible for the proper disposal of these samples, unless previously arranged for with Wood or a third party.