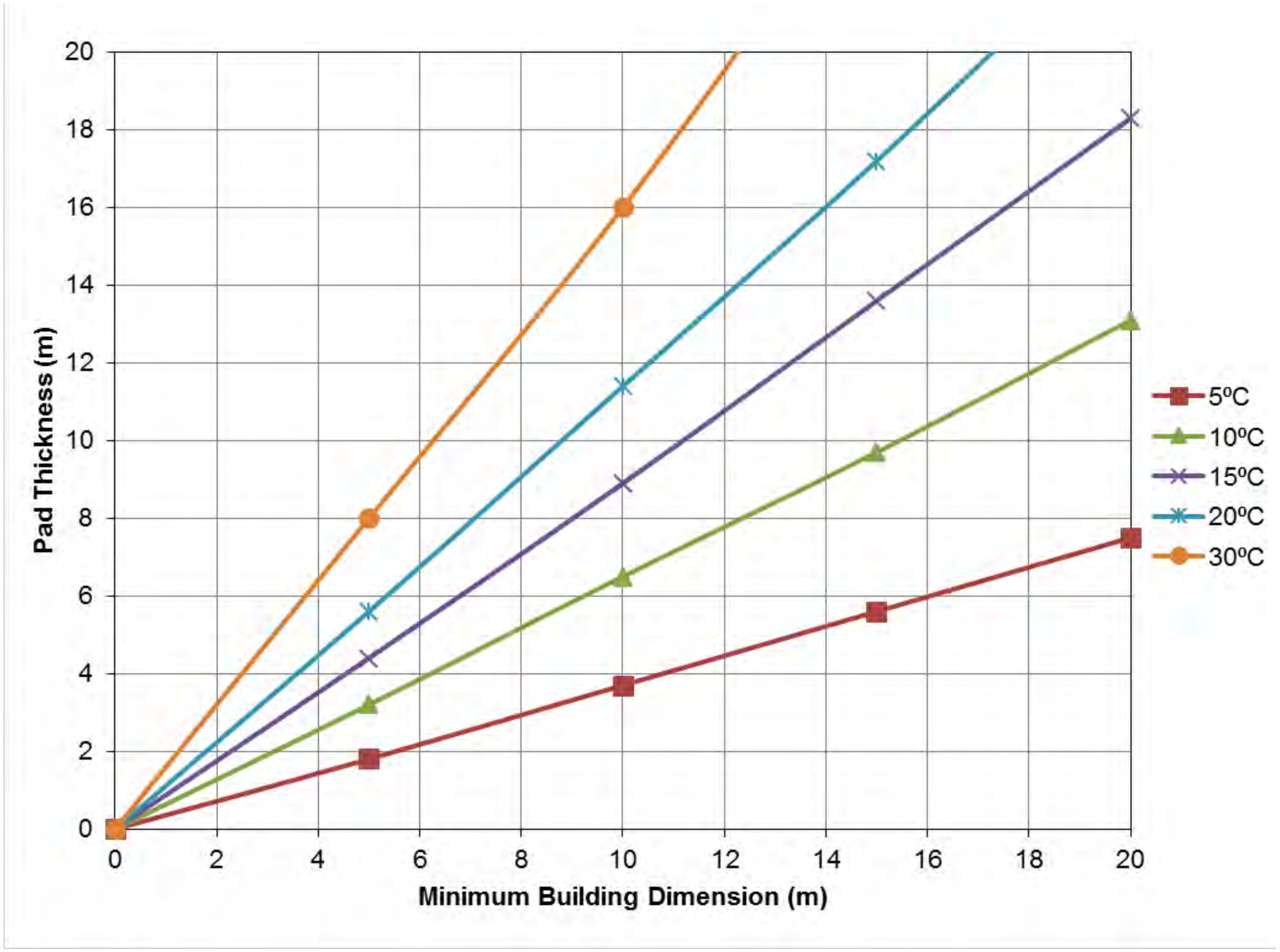
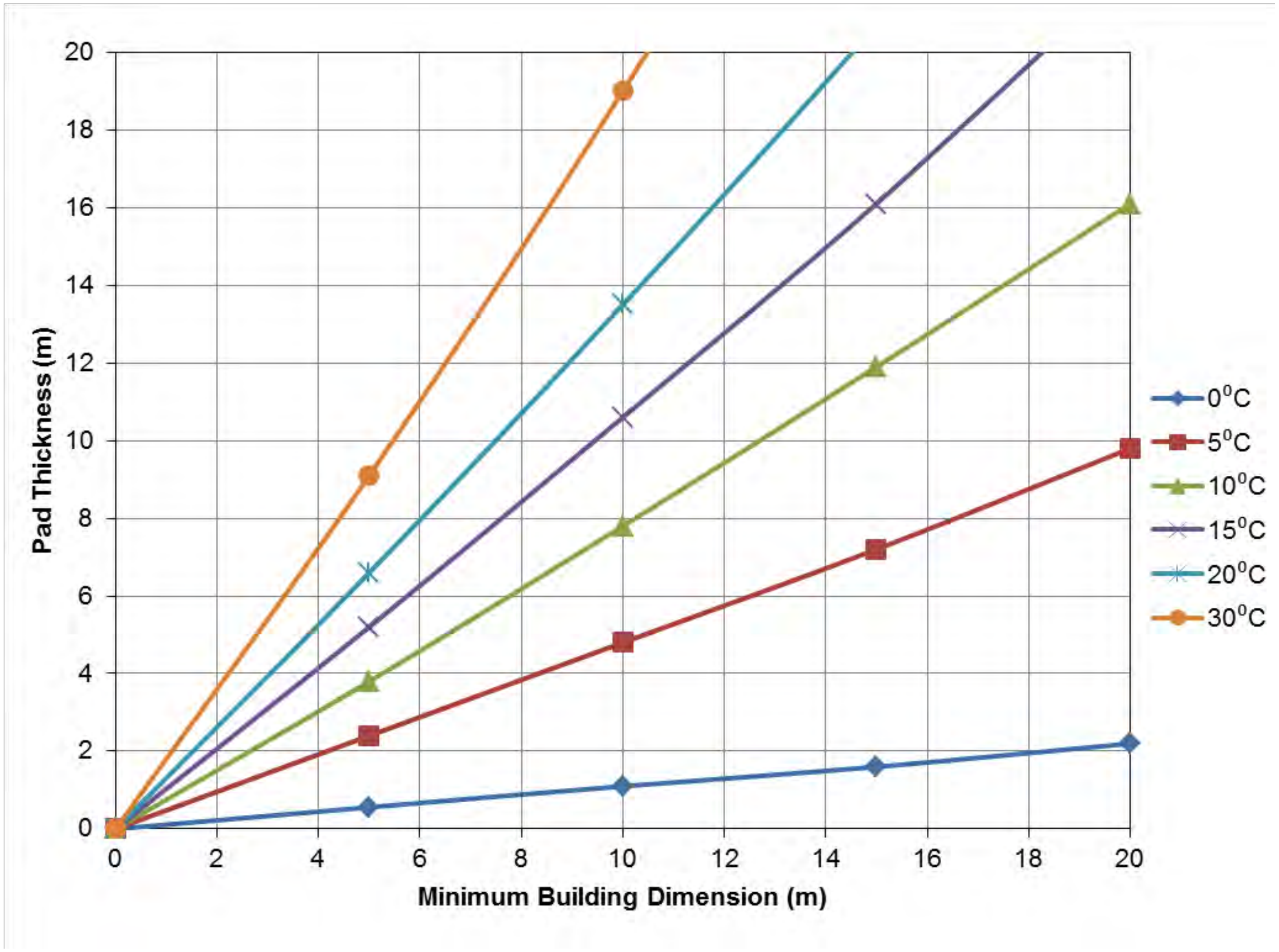


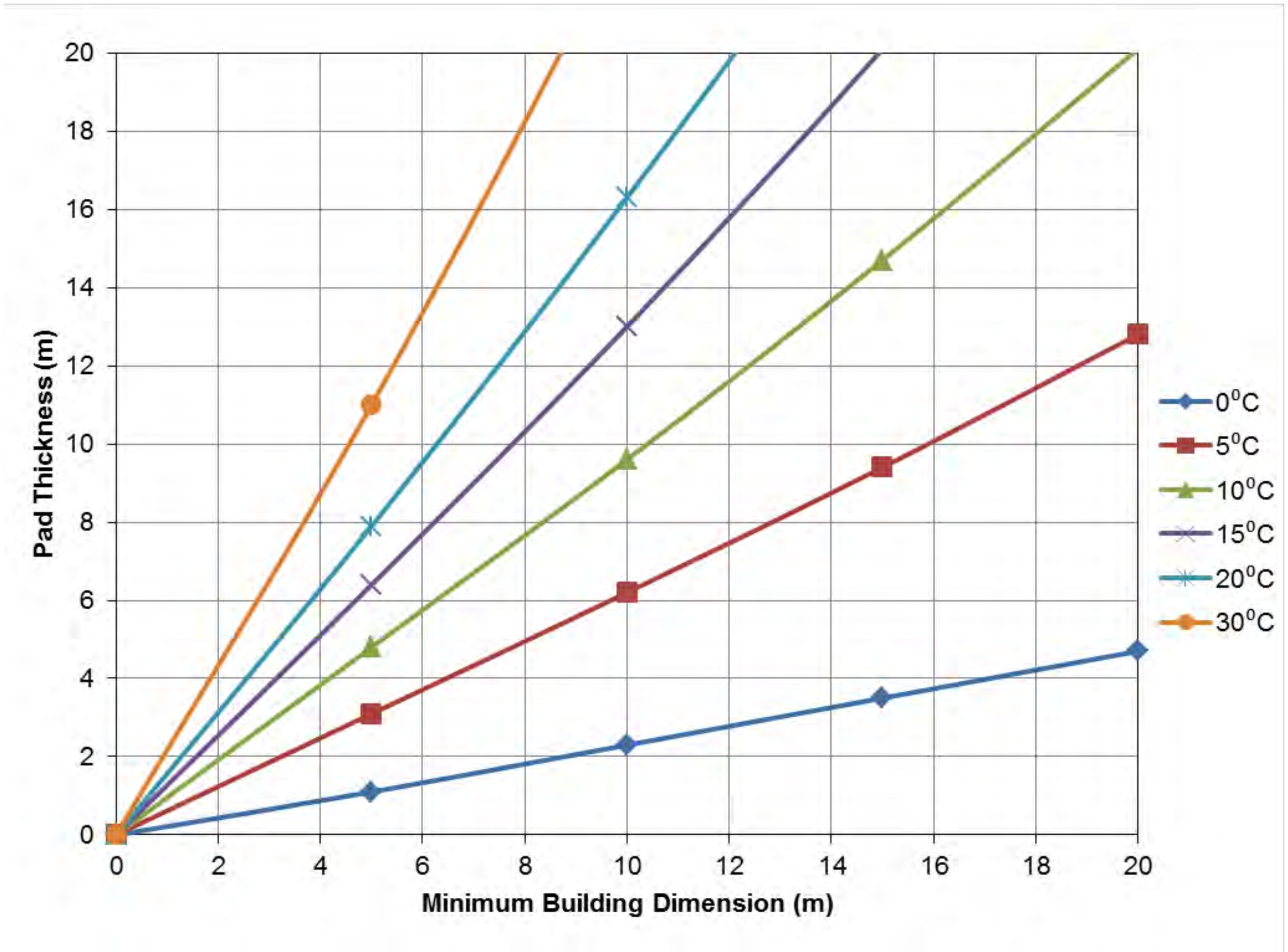
Notes:
1. Heated building temperatures include 5°C, 10°C, 15°C, 20°C, and 30°C
2. Minimum non-insulated pad thickness to maintain 0°C isotherm within pad

srk consulting	TMAC RESOURCES	Infrastructure Thermal Modeling
		Heated Building – Non-Insulated Pad (0°C Isotherm)
Job No: 1CT022.004 Filename: HeatedBuildingNonInsultatedPad_0C.pptx	HOPE BAY PROJECT	Date: 4/18/2016 · Approved: cws · Figure: **10**



Notes:
1. Heated building temperatures include 0°C, 5°C, 10°C, 15°C, 20°C, and 30°C
2. Minimum non-insulated pad thickness to maintain -1°C isotherm within pad

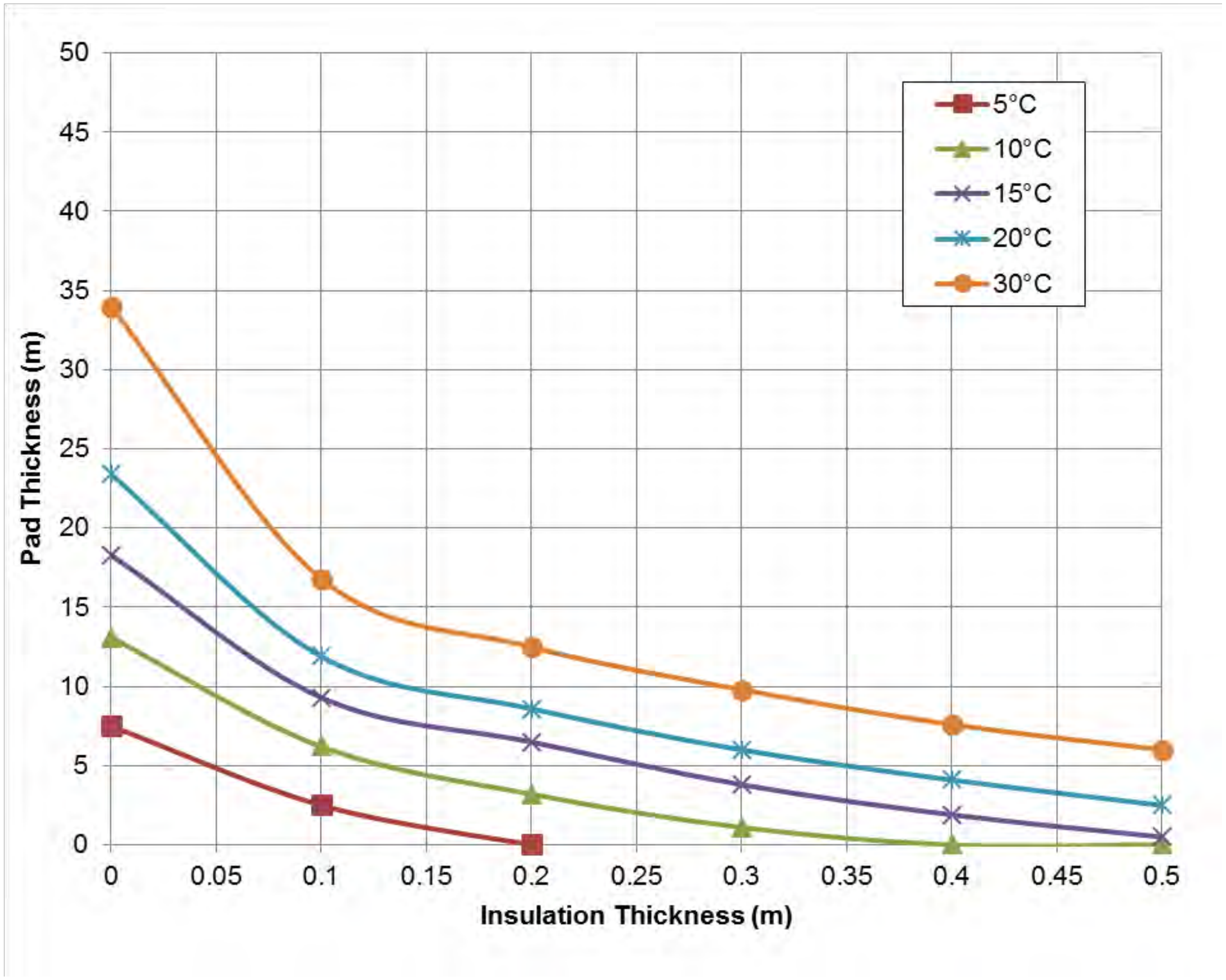
srk consulting	TMAC RESOURCES	Infrastructure Thermal Modeling
		Heated Building – Non-Insulated Pad (-1°C Isotherm)
Job No: 1CT022.004 Filename: HeatedBuildingNonInsultatedPad_-1C.pptx	HOPE BAY PROJECT	Date: 4/18/2016 · Approved: cws · Figure: **11**



Notes:

1. Heated building temperatures include 0°C, 5°C, 10°C, 15°C, 20°C, and 30°C
2. Minimum non-insulated pad thickness to maintain -2°C isotherm within pad

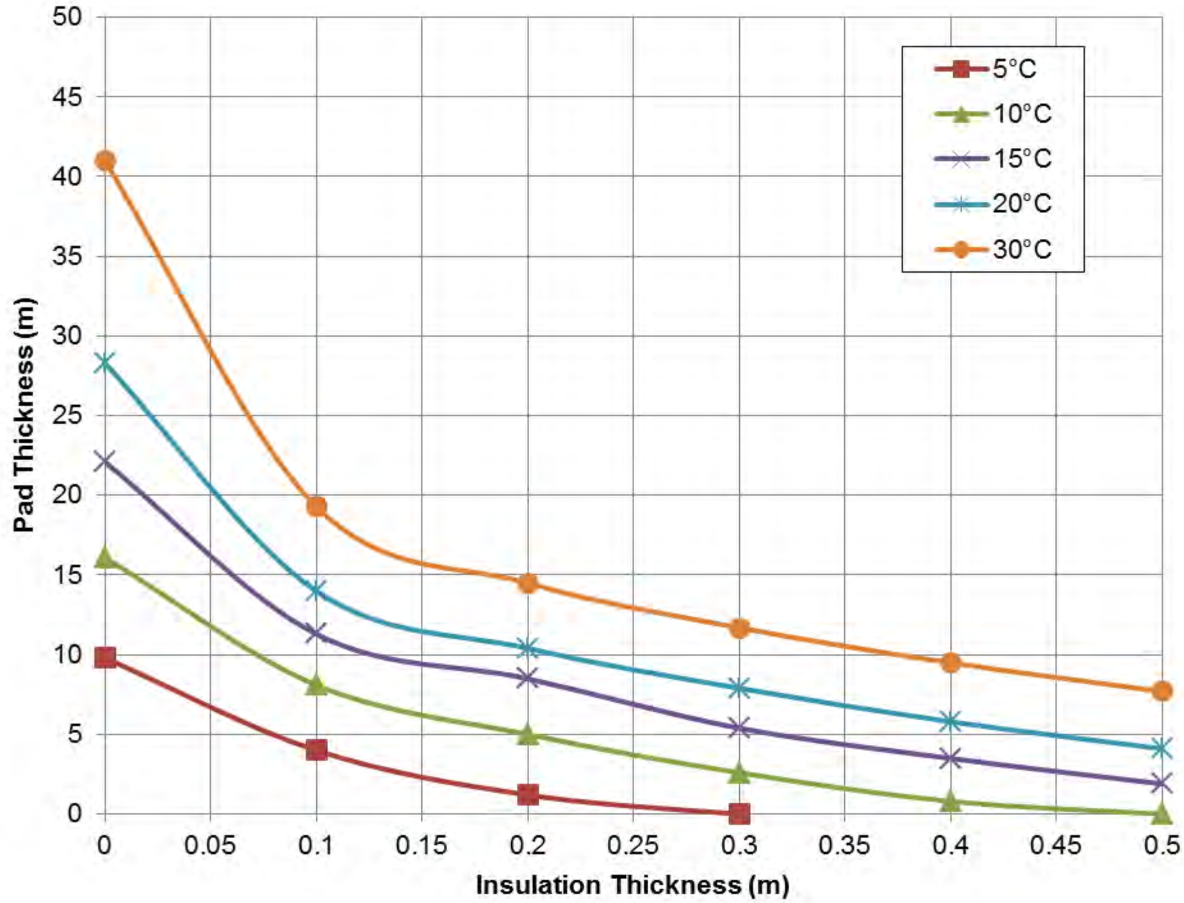
srk consulting	TMAC RESOURCES	Infrastructure Thermal Modeling
		Heated Building – Non-Insulated Pad (-2°C Isotherm)
Job No: 1CT022.004 Filename: HeatedBuildingNonInsultatedPad_-2C.pptx	HOPE BAY PROJECT	Date: 4/18/2016 · Approved: cws · Figure: **12**



Notes:

1. Heated building temperatures include 5°C, 10°C, 15°C, 20°C, and 30°C
2. Minimum pad thickness and insulation to maintain 0°C isotherm within pad

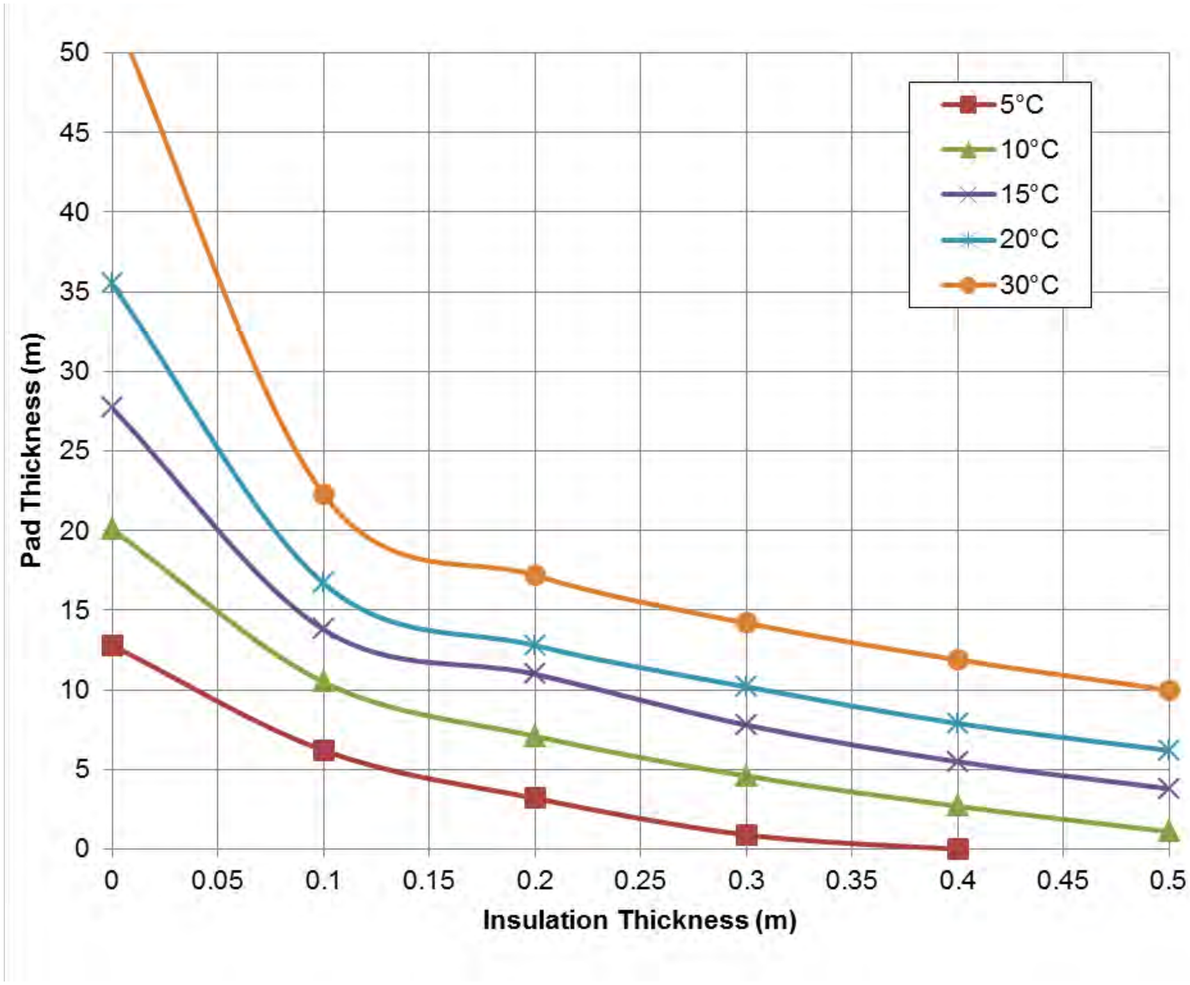
srk consulting	TMAC RESOURCES	Infrastructure Thermal Modeling
		Heated Building Pad Thickness – 0°C Isotherm
Job No: 1CT022.004 Filename: InsulatedPadThickness_0C.pptx	HOPE BAY PROJECT	Date: 4/18/2016 · Approved: cws · Figure: **13**



Notes:

1. Heated building temperatures include 5°C, 10°C, 15°C, 20°C, and 30°C
2. Minimum pad thickness and insulation to maintain -1°C isotherm within pad

srk consulting	MAC RESOURCES	Infrastructure Thermal Modeling
		Heated Building Pad Thickness – -1°C Isotherm
Job No: 1CT022.004 Filename: InsulatedPadThickness_-1C.pptx	HOPE BAY PROJECT	Date: 4/18/2016 · Approved: cws · Figure: **14**



Notes:
1. Heated building temperatures include 5°C, 10°C, 15°C, 20°C, and 30°C
2. Minimum pad thickness and insulation to maintain -2°C isotherm within pad

srk consulting	TMAC RESOURCES	Infrastructure Thermal Modeling
		Heated Building Pad Thickness – -2°C Isotherm
Job No: 1CT022.004 Filename: InsulatedPadThickness_-2C.pptx	HOPE BAY PROJECT	Date: 4/18/2016 · Approved: cws · Figure: **15**

Appendix D – Adfreeze Pile Bond Strength

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Memo

To:	John Roberts, PEng, Vice President Environment	**Client:**	TMAC Resources Inc.
From:	Megan Miller, PEng	**Project No:**	1CT022.004
Reviewed By:	Maritz Rykaart, PhD, PEng	**Date:**	November 22, 2016
Subject:	Hope Bay Project: Adfreeze Pile Bond Strength		

1 Introduction

1.1 General

The Hope Bay Project (the Project) is a gold mining and milling undertaking of TMAC Resources Inc. The Project is located 705 km northeast of Yellowknife and 153 km southwest of Cambridge Bay in Nunavut Territory, and is situated east of Bathurst Inlet. The Project comprises of three distinct areas of known mineralization plus extensive exploration potential and targets. The three areas that host mineral resources are Doris, Madrid, and Boston.

The Project consists of two phases; Phase 1 (Doris project), which is currently being carried out under an existing Water Licence, and Phase 2 which is in the environmental assessment stage. Phase 1 includes mining and infrastructure at Doris, while Phase 2 includes mining and infrastructure at Madrid and Boston located approximately 10 and 60 km due south from Doris respectively.

1.2 Objective

The objective of this memo is to provide a procedure to determine bond strength for adfreeze piles in permafrost foundations and adfreeze piles driven through engineered fill into permafrost foundations for the Project. The adfreeze pile's strengths developed with this method are only applicable to the Project.

While this memo is intended to provide adfreeze pile bond strengths to be used in design, these values should only be used when site specific data is not available.

2 Design Concept

2.1 Approach

Critical Project infrastructure should be founded on bedrock foundations or thermal pads which do not allow settlement. However, in some cases the use of piles may be required to meet the design objectives. In most cases these piles will be founded directly in permafrost, but in some case the piles will be driven through rockfill pads into permafrost.

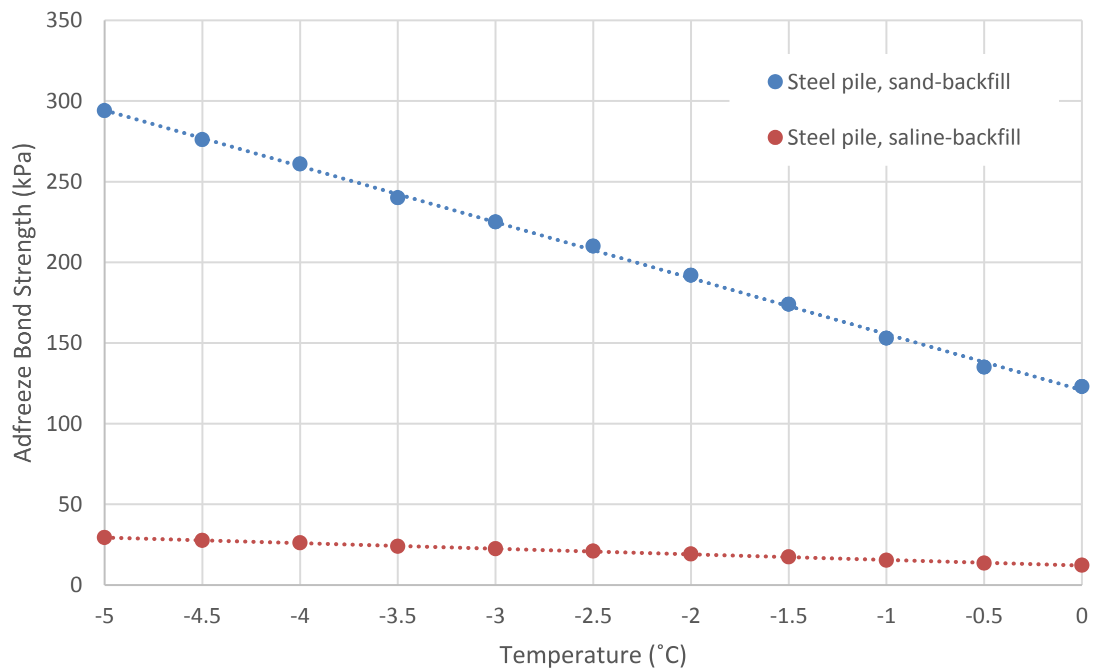
2.2 Foundation Conditions

Project-wide overburden consists of permafrost soils which are mainly marine clays, silty clay and clayey silt, with pockets of moraine till underlying these deposits. The marine silts and clays contain ground ice ranging from 10 to 30% by volume on average, but occasionally as high as 50% (SRK 2016a). The till typically contains low to moderate ice contents ranging from 5 to 25%. Overburden soil pore water is typically saline due to past inundation of the land by seawater following deglaciation of the Project area. The salinity of the marine silts and clays typically range from 37 to 47 parts per thousand which depresses the freezing point and contributes to higher unfrozen water content at below freezing temperatures.

2.3 Design Criteria

Based on measured and modelled ground temperatures (SRK 2016b) and literature adfreeze bond strengths (Weaver and Morgenstern 1981), SRK developed a series of graphs to estimate the strength of the adfreeze bond for steel piles backfilled with a non-saline sand slurry. The estimated adfreeze bond values presented are not valid if an overburden slurry is used for backfill as backfilling with saline permafrost cuttings greatly reduces adfreeze bond strength; a salinity of 15 ppt or greater reduces the bond strength by approximately 90% (Bigger and Sego 1993).

Figure 1 provides adfreeze bond strengths versus temperature for steel piles backfilled with a non-saline sand slurry, and a slurry of saline soil cuttings. The adfreeze bond strength is based on literature values provided in Weaver and Morgenstern (1981), and a saline soil reduction factor of 90% as described in Bigger and Sego (1993). These results show that under non-saline conditions, the maximum adfreeze bond strength at a temperature of −5°C is 294 kPa.



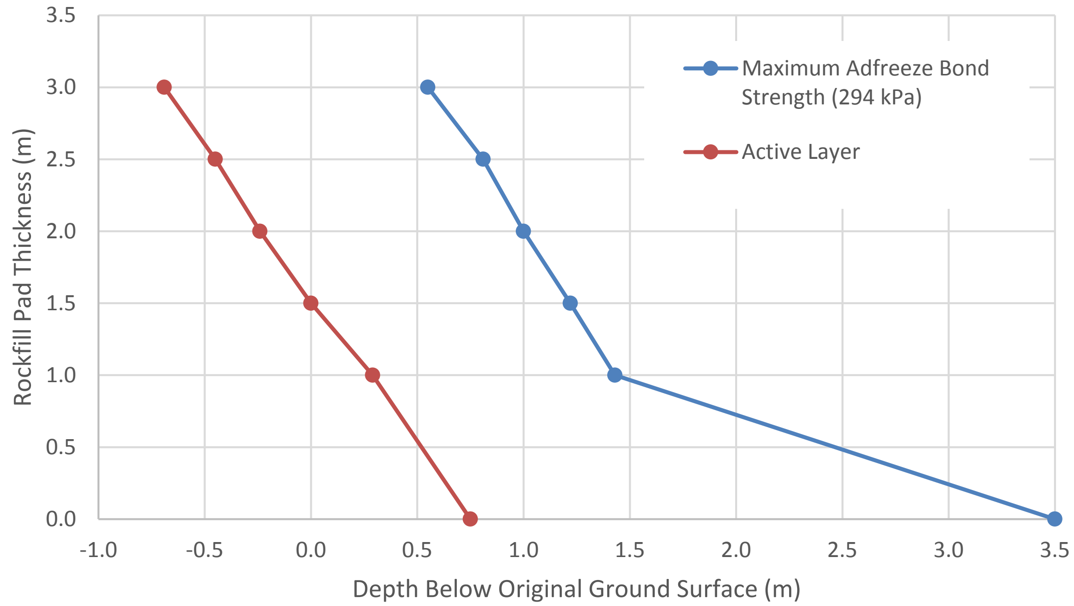
Source: \\srk.ad\dfs\na\van\Projects\01_SITES\Hope.Bay\1CT022.004_Phase 2 DEIS - Engineering Support\Task 210_Geotechnical_Overburden\[AdfreezePile_Calculations_1CT022.004_rev01_mmm.xlsx]

Figure 1: Adfreeze Bond vs Temperature for Steel Pile Backfilled with Non- saline Sand Backfill, and Saline Backfill

The blue line in Figure 2 provides the depth below overburden surface where the maximum adfreeze bond strength of 294 kPa (at −5°C ground temperature) is first encountered, for various rockfill pad thicknesses. The adfreeze bond strength along the pile below this point remains at 294 kPa. Above the depth of maximum adfreeze pile bond strength (Figure 2, blue line) the adfreeze bond varies linearly from:

- The adfreeze bond strength at a temperature of 0°C (123 kPa, Figure 1), at the maximum depth of the active layer, to the maximum adfreeze bond strength at the depth shown in Figure 2 (for thin rockfill pads, or piles directly in overburden where there is an active layer within the overburden soils), or
- The bond strength associated with the maximum temperature at original ground surface (Figure 3) to the maximum bond strength (294 kPa) over the depth shown in Figure 2 (for thick pads where the overburden remains frozen all year round).

The adfreeze bond strength of 123 kPa at a temperature of 0°C is obtained from Figure 1, the maximum predicted depth of the active layer for the various fill thicknesses is provided in Figure 2 (red line). For thick pads where the overburden is expected to remain frozen year round (e.g., where the red line in Figure 2 has a negative depth below original ground surface) the adfreeze bond strength at the top of overburden can be obtained from Figure 3.



Source: \\srk.ad\dfs\na\van\Projects\01_SITES\Hope.Bay\1CT022.004_Phase 2 DEIS - Engineering Support\Task 210_Geotechnical_Overburden\[AdfreezePile_Calculations_1CT022.004_rev01_mmm.xlsx]

Figure 2: Active Layer and Maximum Adfreeze Bond Strength Depth for Various Thicknesses of Rockfill Pad

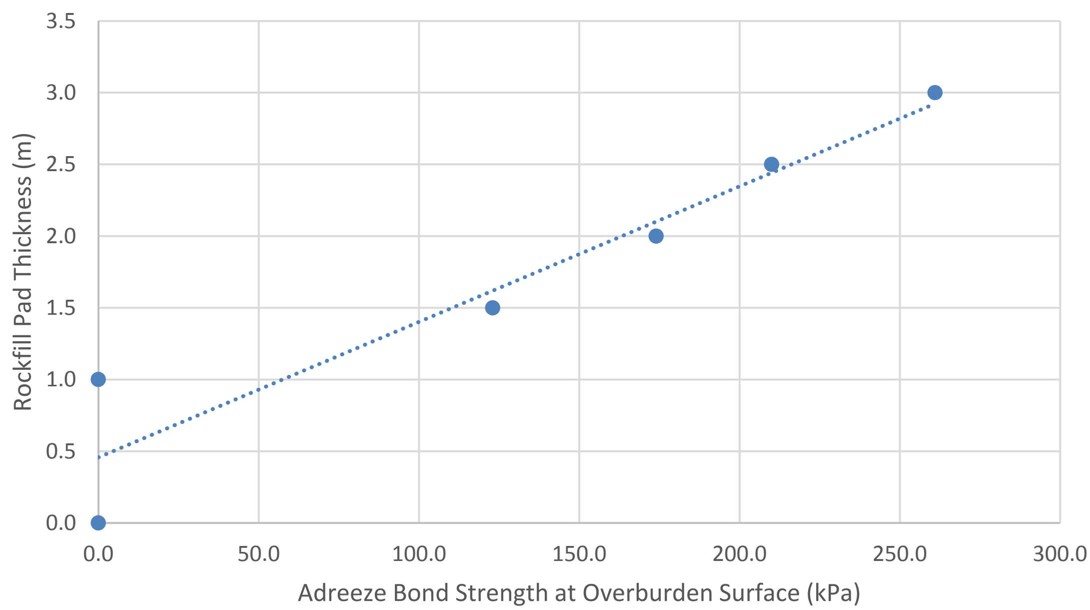


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Figure 3: Adfreeze Bond Strength at Overburden Surface

3 References

Bigger, K.W. and Sego, D.C. 1993. The Strength and Deformation Behavior of Model Adfreeze and Grouted Piles in Saline Frozen Soil. Canadian Geotechnical Journal. Volume 30. pp. 319-337.

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SRK Consulting (Canada) Inc. 2016b. Hope Bay Project: Thermal Modelling to Support Run-of-Quarry Pad Design. Memo prepared for TMAC Resources Inc. Project No.: 1CT022.004. 2016.

Weaver, J.S., and Morgenstern, N.R. 1981. Pile Design in Permafrost. Canadian Geotechnical Journal. Volume 18. pp. 357-370.

Appendix E – Waste Rock Stability Analysis

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Memo

To:	John Roberts, PEng, Vice President Environment	**Client:**	TMAC Resources Inc.
From:	Sam Amiralaei, PEng Megan Miller, PEng	**Project No:**	1CT022.004
Reviewed by:	Maritz Rykaart, PhD, PEng	**Date:**	November 8, 2016
Subject:	Hope Bay Project: Waste Rock Pile Stability Analysis		

1 Introduction

1.1 General

The Hope Bay Project (the Project) is a gold mining and milling undertaking of TMAC Resources Inc. The Project is located 705 km northeast of Yellowknife and 153 km southwest of Cambridge Bay in Nunavut Territory, and is situated east of Bathurst Inlet. The Project comprises of three distinct areas of known mineralization plus extensive exploration potential and targets. The three areas that host mineral resources are Doris, Madrid, and Boston.

The Project consists of two phases; Phase 1 (Doris project), which is currently being carried out under an existing Water Licence, and Phase 2 which is in the environmental assessment stage. Phase 1 includes mining and infrastructure at Doris only, while Phase 2 includes mining and infrastructure at Madrid and Boston located approximately 10 and 60 km due south from Doris respectively.

Waste rock piles are planned at Madrid North (646,000 tonnes, 359,000 m^3), Madrid South (65,000 tonnes, 361,000 m^3), and Boston (628,000 tonnes, 349,000 m^3). These waste rock piles are however temporary, since all mine waste rock will be backhauled underground as structural mine backfill during the life of the Project.

1.2 Objectives

This memo presents the results of stability analysis completed for the most critical cross section (i.e. the highest) of the Madrid South waste rock pile to confirm its surficial and overall stability. The Madrid South waste rock pile was selected to be representative of the waste rock pile stability at all locations because it had the steepest foundation slope, and the pile height is only expected to be 3 m less than that of the Madrid North waste rock pile. To account for higher piles, or a future increase in pile height, a conceptual 100 m high pile located on the Madrid South pile location was also analyzed. The analysis results are used to define general design guidelines for all of the Project waste rock piles.

2 Stability Criteria

Federal or territorial (Nunavut) guidelines for waste rock pile designs do not exist. Therefore, the draft British Columbia Guidelines for Mined Rock and Overburden Piles (Piteau, 1991), were used to suggest the design requirements of the waste rock piles. The suggested minimum design factors of safety (FOS) are presented in Table 1. The ranges in FOS for Cases A and B, reflect the different levels of confidence in understanding site conditions, material parameters, and consequences of instability.

The stability conditions mentioned in the Table 1 are described in detail in Section 3.4. Based on the current level of design and available information on foundation conditions, the Case A minimum FOS were used to assess the waste rock pile stability.

Table 1. British Columbia Mine Dump Factor of Safety Guidelines

Stability Condition	Suggested Minimum Design Values for Factor of Safety	
	Case A	**Case B**
Stability of Waste Rock Pile Surface		
Short-term (during construction) - (Stability Condition 1)	1.0	1.0
Long-term (reclamation – abandonment) – (Stability Condition 2)	1.2	1.1
Overall Waste Rock Pile Stability (Deep Seated Stability)		
Short-term (static) – (Stability Condition 3)	1.3 - 1.5	1.1 – 1.3
Long-term (static) – (Stability Condition 4)	1.5	1.3
Pseudo-static (Earthquake)[2]	1.1 – 1.3	1.0
CASE A: -Low level of confidence in critical analysis parameters -Possibly unconservative interpretation of conditions, assumptions -Severe consequences of failure -Simplified stability analysis method (charts, simplified method of slices) -Stability analysis method poorly simulates physical conditions -Poor understanding of potential failure mechanism(s)		
CASE B: -High level of confidence in critical analysis parameters -Conservative interpretation of conditions, assumptions -Minimal consequences of failure -Rigorous stability analysis method -Stability analysis method simulates physical conditions well -High level of confidence in critical failure mechanism(s)		

Source: Piteau 1991
Notes:
1. A range of suggested minimum design values are given to reflect different levels of confidence in understanding site conditions, material parameters, consequences of instability, and other factors.
2. Where pseudo-static analyses, based on peak ground accelerations which have a 10% probability of exceedance in 50 years, yield FOS < 1.0, dynamic analysis of stress-strain response, and comparison of results with stress-strain characteristics of dump materials is recommended.

3 Slope Stability Assessment

3.1 Material Properties

3.1.1 Overburden Material Properties

Geotechnical investigations have not been performed within the proposed footprints of the waste rock piles. However, numerous geotechnical investigations have been performed on site which provide a general understanding of the foundation conditions to be expected under the waste rock piles.

The general overburden profile consists of a thin veneer of hummocky organic soil covered by tundra heath vegetation. Under this organic layer is a layer of marine silts and clays (i.e. silty clay and clayey silt) typically between 5 and 20 m thick. The bedrock contact zone consists of a relatively thin rubble zone of weathered blocky host rock (SRK, 2016a).

The waste rock piles will be constructed on a pad of run-of-quarry (ROQ) material overlaying the permafrost soils. The slope stability models were set up using the geotechnical properties (SRK, 2016a) for marine silts and clays as the foundation soils. These material properties are summarized in Table 2.

The depth of the marine silts and clay layer under then Madrid South waste rock pile was estimated based on nearby geotechnical drill holes.

3.1.2 Waste Rock Pile Properties

The physical properties of the waste rock material for the Project have not been measured, but the physical properties used in the stability analyses are based on a comparison with Project's ROQ borrow material as reported in the literature and SRK's internal database. These properties can be seen in Table 2.

Table 2: Material Properties

Parameter		Marine Silt and Clay	Waste Rock Pile
Moist Unit Weight (kN/m^3)		17	20
Unfrozen	Apparent Cohesion c' (kPa)	0	0
	Friction Angle, φ ^{0}	30	40
	Undrain Shear Strength, Su	13	-
Frozen	Apparent Cohesion c' (kPa)	112	5
	Friction Angle, φ ^{0}	26	40

Source: SRK 2016a

A critical cross-section through the Madrid South waste rock pile, based on ultimate waste rock pile height, was selected to create the model used to run the analysis (Figure 1). A second model was created simply by increasing the height of the Madrid South critical cross-section to 100 m while keeping the slope and foundation conditions identical (Figure 2).

3.2 Seismic Coefficient

The British Columbia Guidelines for Mined Rock and Overburden Piles (Piteau, 1991) recommends that a seismic event with a 10% probability of exceedance in 50 years (1:500 year) should be used to assess the waste rock piles (Table 1).

Horizontal seismic coefficient for the waste rock pile analysis were developed using the Limit Equilibrium Pseudo Static Stability Analysis method (SRK, 2016b). These seismic coefficients were developed specific to the waste rock pile geometry and recommended probability of exceedance, and are not applicable to other structures. The vertical seismic coefficients are assumed to be negligible. Table 3 provides the seismic coefficients used in the slope stability analysis.

Table 3: Seismic Coefficients used in the Waste Rock Stability Analysis

Waste Rock Pile Height (m)	Design Stage	Seismic Coefficient (g)
≤ 5	First bench	0.0086
20	Madrid South ultimate planned height	0.0075
100	Largest pile (theoretical case)	0.0067

(1) Source: SRK 2016b

3.3 Model Setup

The slope stability models were set up in SLOPE/W, a limit equilibrium slope stability analysis software tool developed by GEO-SLOPE International Ltd (Geoslope, 2012). The software is commonly used to compute the FOS of earth and rock slopes.

For the stability analyses, the waste rock piles are conservatively assumed to be unfrozen. This is conservative since freeze back may occur in both the foundation pad and the waste rock pile over the life of the structure. The thickness of the thawed foundation layer at the toe of the waste rock pile is assumed to be 1 m which is in line with the results of thermal analysis (SRK, 2016a).

3.4 Methodology

The stability, of the waste rock pile also took into consideration haul truck wheel loads applied near the crest of the waste rock pile. A loaded Sandvik TH540 was assumed to be the heaviest vehicle driving on the waste rock pile. The wheel loading calculation for the TH540 haul truck is included as Attachment 1. The minimum safe distance of the truck from the crest of the waste rock pile was determined to be 5.5 m satisfying the minimum recommended FOS.

The following two scenarios were considered and analyzed:

- Madrid South waste rock pile at the maximum planned design height (19 m); and
- Madrid South waste rock pile, assuming a theoretical maximum height of 100 m.

The slope stability of the waste rock piles were evaluated under five stability conditions (Table 1):

- Short-term (surficial/static) (Stability Condition 1): This stability case considers the stability of the waste rock pile surface with the truck loading applied at 5.5 m away from the crest of the waste rock pile.
- Long-term (surficial/static) (Stability Condition 2): This stability case considers the stability of the waste rock pile surface without the haul truck loads near the crest.
- Short-term (overall/static) (Stability Condition 3): This stability case considers the stability of the overall waste rock pile with the truck loading applied near the crest of the waste rock pile, and is only analyzed for deep seated stability by forcing the slip-surface to a particular path or certain depth.
- Long-term (overall/static) (Stability Condition 4): This stability case considers the stability of the overall waste rock pile without the truck loading applied, only is only analyzed for deep seated stability by forcing the slip-surface to a particular path or certain depth.
- Earthquake (overall/pseudo-static) (Stability Condition 5): This stability case considers the stability of the overall waste rock pile with the truck loading applied near the crest of the waste rock pile under seismic load.

The slope stability analyses were carried out using the Morgenstern-Price Method and were assessed for both static and pseudo-static conditions. To provide confidence in the results, the models were analyzed using three modes of searching for the failure surface:

- Grid and radius;
- Specified entry and exit locations; and
- Fully specified failure surface.

The waste rock piles were assumed to be unsaturated, so pore water pressure conditions were not applied in the analyses.

4 Results

The lowest calculated FOS for the analyzed critical section of the Madrid South waste rock pile is presented in Table 4 while the results for all of the analyses are provided in Attachment 2. Although there is a good understanding of site conditions, material parameters and consequences of instability, which suggests the waste rock piles should meet the FOS required for Case B, as the calculated FOS was found to exceed those listed for Case A as well.

Table 4. Madrid South (Maximum Height) Waste Rock Pile Stability Analysis Results

Stability Analysis	Loading Condition(1)	Recommended FOS (Case A)	Calculated FOS
Short-term (Surficial Stability) (Stability Condition 1)	Undrained	1.0	1.1
Long-term (Surficial Stability) (Stability Condition 2)	Drained	1.2	1.3
Short-term (Overall Stability) (Stability Condition 3)	Undrained	1.3-1.5	1.8
Long-term (Overall Stability) (Stability Condition 4)	Drained	1.5	2.0
Earthquake (Overall Stability) (Stability Condition 5)	Undrained	1.1-1.3	1.8

Note:

(1) Loading conditions refers to the overburden foundations, in all cases the waste rock is assumed to be drained.

In order to confirm the stability of the Madrid South waste rock pile during construction, the stability of the pile at the end of the construction of the first bench was also analyzed. The main objective for the completion of this analysis was to check the stability of the pile through foundation failure assuming thawed foundation conditions. The results of this analysis are summarized in Table 5.

Table 5. Madrid South (1st Bench) Waste Rock Pile Stability Analysis Results

Stability Analysis	Loading Condition	Recommended FOS (Case A)	Calculated FOS
Short-term (Surficial Stability) (Stability Condition 1)	Undrained	1.0	1.1
Long-term (Surficial Stability) (Stability Condition 2)	Drained	1.2	1.3
Earthquake (Overall Stability) (Stability Condition 5)	Undrained	1.1-1.3	1.1

Note:

(1) Loading conditions refers to the overburden foundations, in all cases the waste rock is assumed to be drained.

The stability analysis results for a waste rock pile with a height of 100 m is summarized in Table 6. The surficial stability was not analyzed for this model, since the slope of this analyzed section is identical to the model which was analyzed for Madrid south waste rock pile.

Table 6. Stability Analysis Results for a Waste Rock Pile With a Height of 100 m

Stability Analysis	Loading Condition	Recommended FOS (Case A)	Calculated FOS
Short-term (Overall Stability) (Stability Condition 3)	Undrained	1.3-1.5	2.1
Long-term (Overall Stability) (Stability Condition 4)	Drained	1.5	2.1
Earthquake (Overall Stability) (Stability Condition 5)	Undrained	1.1-1.3	2.1

Note:

(1) Loading conditions refers to the overburden foundations, in all cases the waste rock is assumed to be drained.

5 Discussion

As shown in Table 4 to Table 6, the FOS computed by the models exceed the minimum FOS recommended by British Columbia Guidelines for Mined Rock and Overburden Piles (Piteau 1991) in all cases analysed. Therefore, the waste rock piles as design are expected to be stable under static, and pseudo-static conditions, provided the design haul truck remains 5.5 m away from the crest of the pile. It is assumed the haul tucks unload the waste rock at least 5.5 m from the crest and a bulldozer will push the material to the crest.

The waste rock pile geometry, and foundation conditions modeled in the analysis are consistent with the conditions expected under all waste rock piles planned site; therefore, all waste rock piles are expected to be stable.

6 References

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[Piteau 1991] Piteau Associates Engineering Ltd. for British Columbia Mine Waste Rock Pile Research Committee. 1991. Mined Rock and Overburden Piles Investigation and Design Manual. Interim Guidelines. Prepared for the British Columbia Mine Dump Committee. May 1991.

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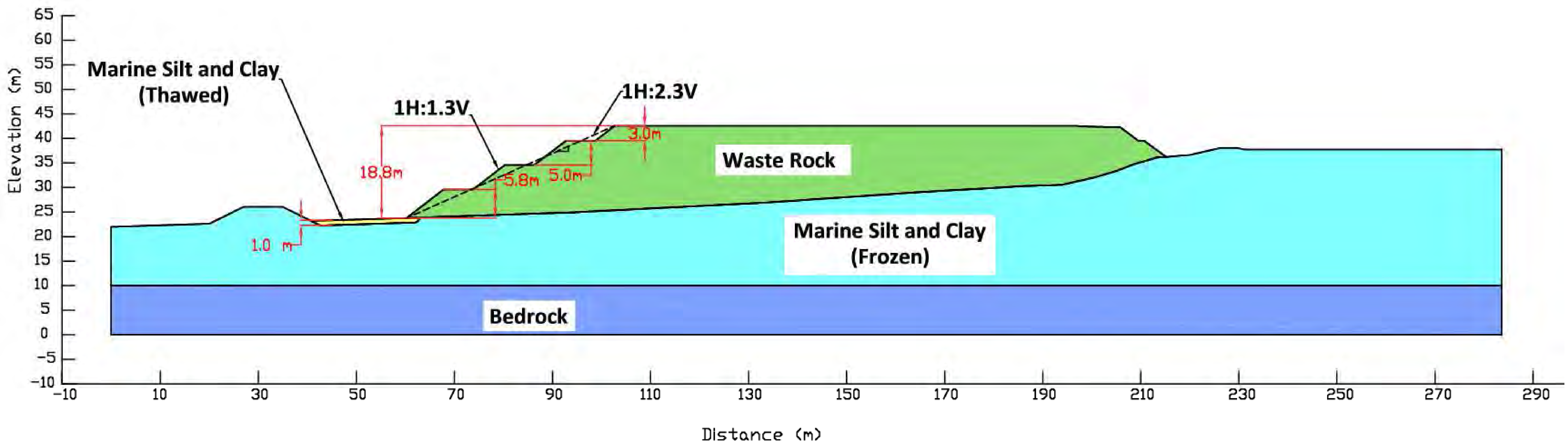
SRK Consulting (Canada) Inc. 2016b. Horizontal Seismic Parameters for Pseudo-Static Modeling Memo prepared for TMAC Resources Inc. Project No.:1CT022.004. 2016.

Figures

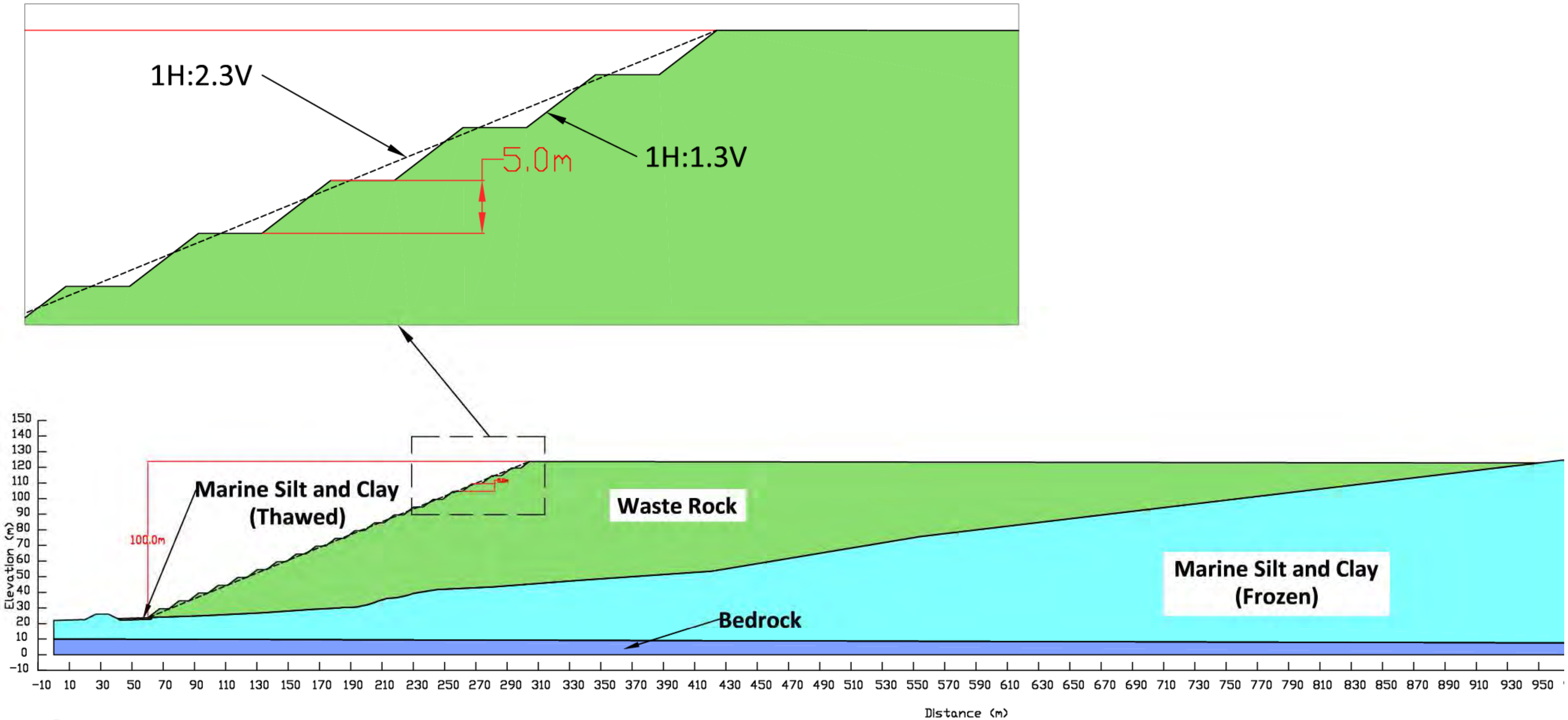


LEGEND

- Waste Rock
- Marine Silt and Clay (Frozen)
- Marine Silt and Clay (Thawed, Undrained)
- Marine Silt and Clay (Thawed, Drained)
- Bedrock

Note: Only undrained loading condition is shown in this figure. The drained analysis were completed by simply changing the material properties for the same boundary.

srk consulting	TMAC RESOURCES	WASTE ROCK PILE STABILITY ANALYSIS
Job No: 1CT22.004 Filename: HopeBay_MadridSouth_WRD_SlopeStability_160419_sa	HOPE BAY PROJECT	**Madrid South Waste Rock Pile Analyzed Cross Section**
		Date: April 2016 / Approved: SA / Figure: 1



LEGEND

- Waste Rock
- Marine Silt and Clay (Frozen)
- Marine Silt and Clay (Thawed, Undrained)
- Marine Silt and Clay (Thawed, Drained)
- Bedrock

Note: Only undrained condition is shown in this figure. The drained analysis were completed by simply changing the material properties for the same region.

srk consulting	TMAC RESOURCES	WASTE ROCK PILE STABILITY ANALYSIS
		Waste Rock Pile With a Height of 100 Analyzed Section
Job No: 1CT22.004 Filename: HopeBay_MadridSouth_WRD_SlopeStability_160419_sa	HOPE BAY PROJECT	Date: April 2016 · Approved: SA · Figure: 2

Attachment 1: Truck Loading Calculations

Wheel load approximation for the Sandvik TH540			Reference
Operating Weight	34700	kg	(1)
Payload Capacity	40000	kg	(1)
Gross operating weight	74700	kg	(1)
Max operating weight	82700	kg	(2)
Loaded front axel weight	37200	kg	
% of gross operating weight	49.8%		
Loaded front rear weight	37500	kg	
% of gross operating weight	50.2%		
Front axel maximum weight	41184	kg	
Rear axel weight	41516	kg	
Load on each front tire	202.0072	kN	
Load on each rear tire	203.6363	kN	
Tire static loaded width	743	mm	(3)
Static loaded radius	784	mm	(3)
Assumed Contact length	743	mm	
Contact Area of one tire	0.552049	m2	
Ground pressure applied by each rear tire	368.87	kPa	

(1) Details on the Sandvik TH540 can be found in the Technical specs online

http://www.miningandconstruction.sandvik.com/sandvik/5100/SAM/Internet/cl01023.nsf/Alldocs/Products*5CLoad*and*haul*machines*5CUnderground*trucks*2ASandvik*40/$FILE/Sandvik%20TH540%20techspec.pdf

(2) 10-10-20 Payload Policy documents

Weight Calculation extracted from the Caterpillar 10-10-20 Payload Policy documents applied to the Sandvik Specs

Empty Chassis Weight (ECW) + Body and Liner = Empty Machine Weight (EMW) + Debris Fuel Attachments = Empty Operating Weight (EOW)

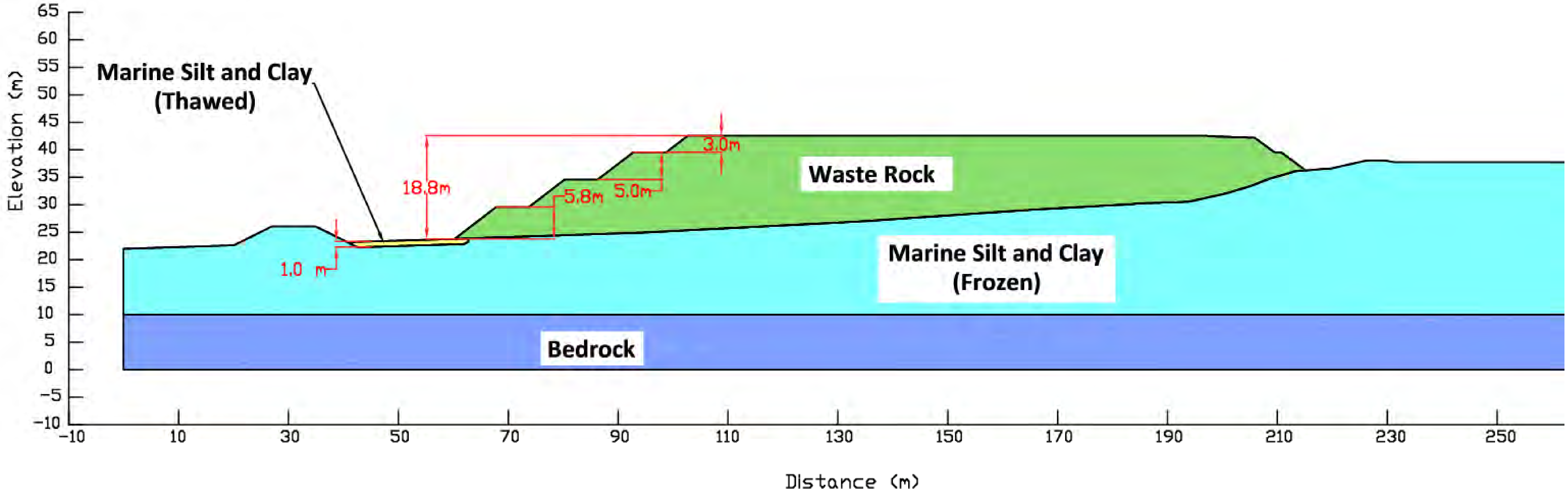
Target Gross Machine Weight (TGMW) -Empty Operating Weight (EOW) = Target Payload (TP)

Target Payload (TP) x 1.2 + Empty Operating Weight (EOW) < Maximum Gross Machine Weight (MGMW)

(3) Bridgestone tire specifications VLTS (26.5R25)

http://www.bridgestone.com/products/speciality_tires/off_the_road/products/pdf/brochure_earth_010.pdf

Attachment 2: Waste Rock Pile Stability Analysis



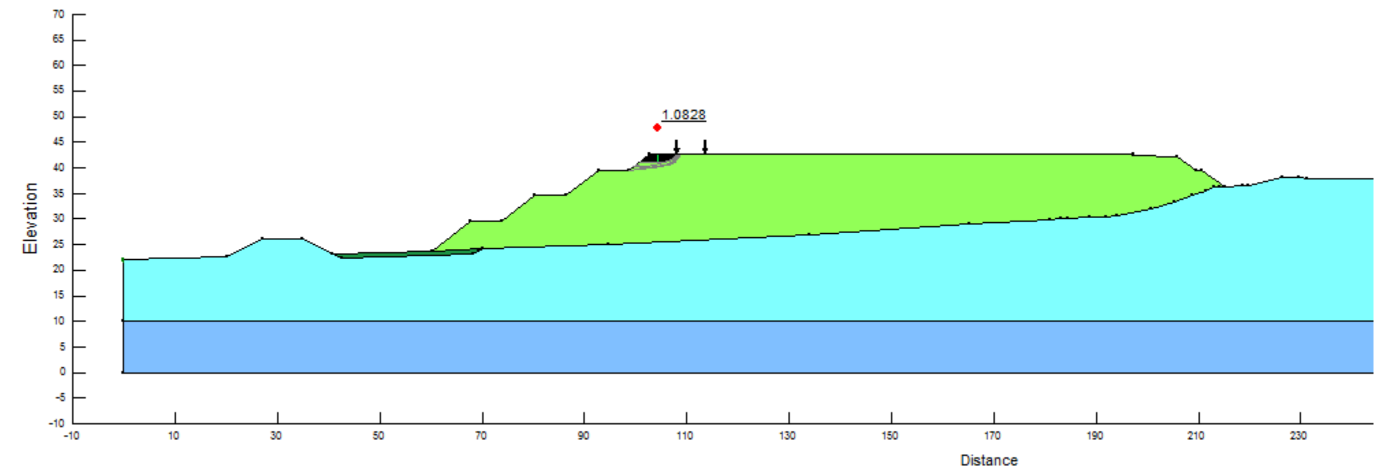
LEGEND

- Waste Rock
- Marine Silt and Clay (Frozen)
- Marine Silt and Clay (Thawed, Undrained)
- Marine Silt and Clay (Thawed, Drained)
- Bedrock

Note: Only undrained loading condition is shown in this figure. The drained analysis were completed by simply changing the material properties for the same boundary.

srk consulting	TMAC RESOURCES	WASTE ROCK PILE STABILITY ANALYSIS		
		Madrid South Waste Rock Pile Analyzed Cross Section		
Job No: 1CT22.004 Filename: HopeBay_MadridSouth_WRD_SlopeStability_160419_sa	HOPE BAY PROJECT	Date: April 2016	Approved:	Figure: 1

Madrid South Waste Rock Pile Stability Analysis Results – Stability of Dump Surface

<table>
<tr><th>Model/Stability Condition</th><th>Factor of Safety</th><th>Slip Surface Option</th><th>Figure</th></tr>
<tr><td rowspan="3">Madrid South/Short Term (Undrained Static Condition)</td><td>1.1</td><td>Fully Defined</td><td>
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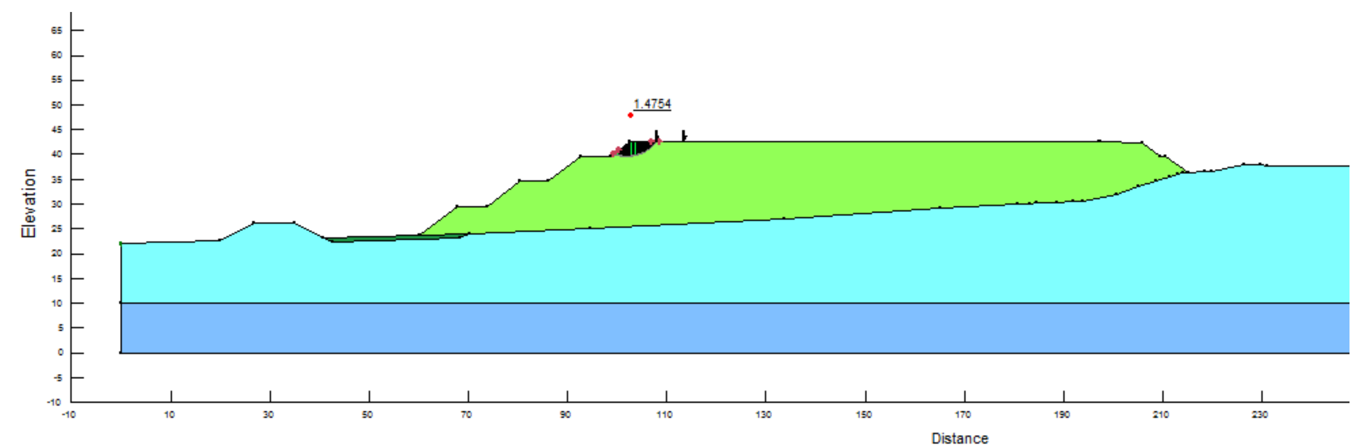
<tr><td>1.5</td><td>Entry Exit</td><td>
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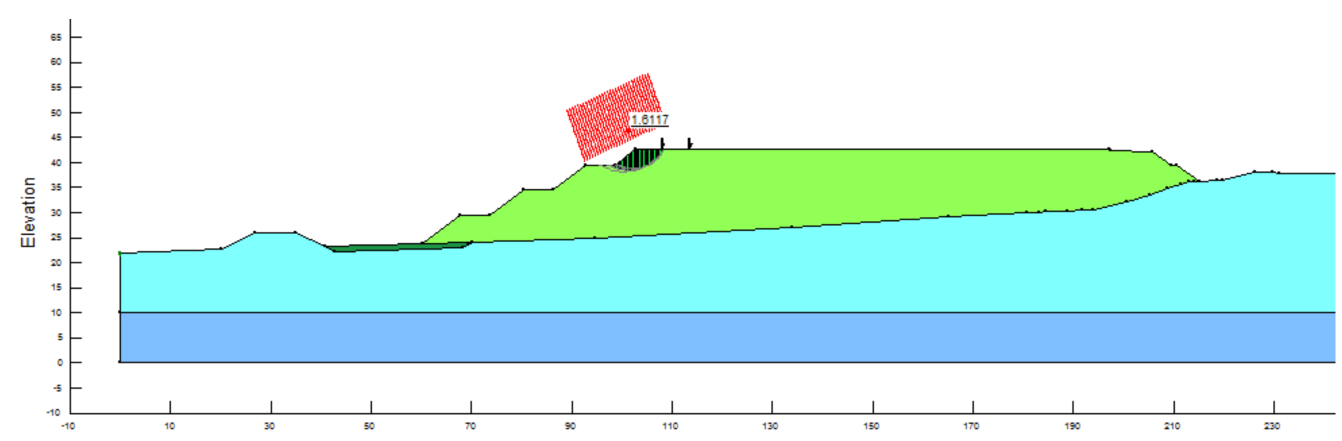
<tr><td>1.6</td><td>Grid and Radius</td><td>
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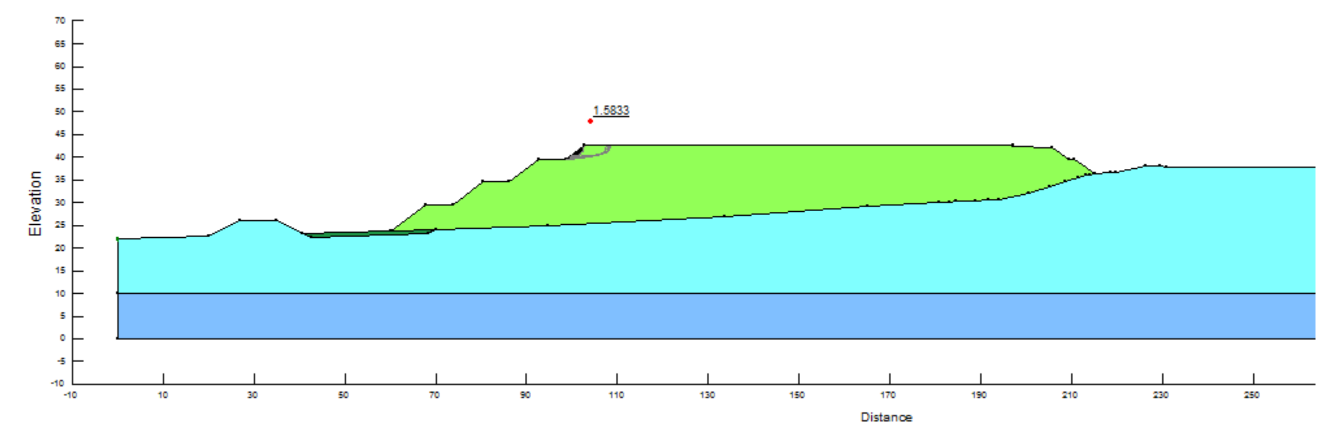
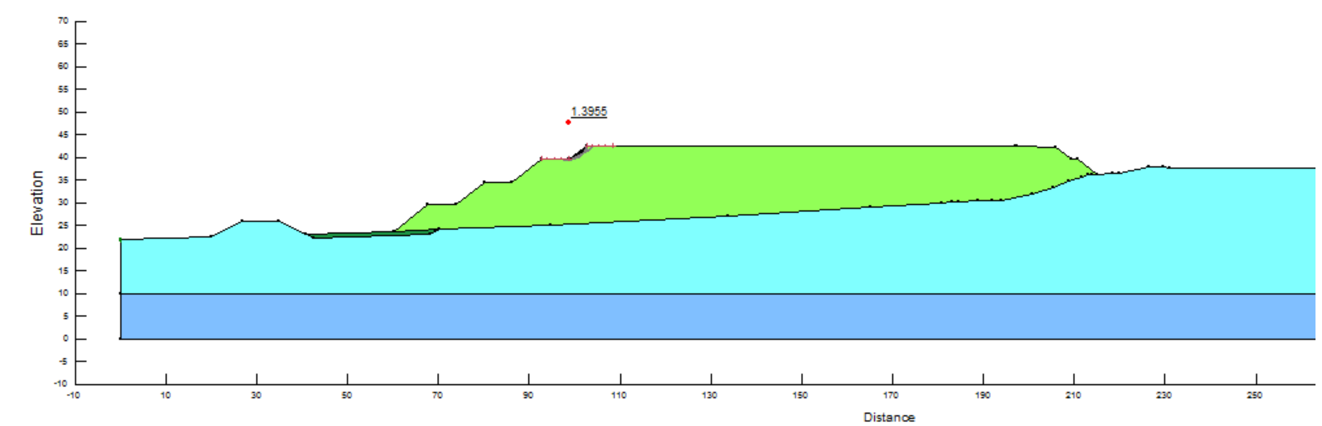
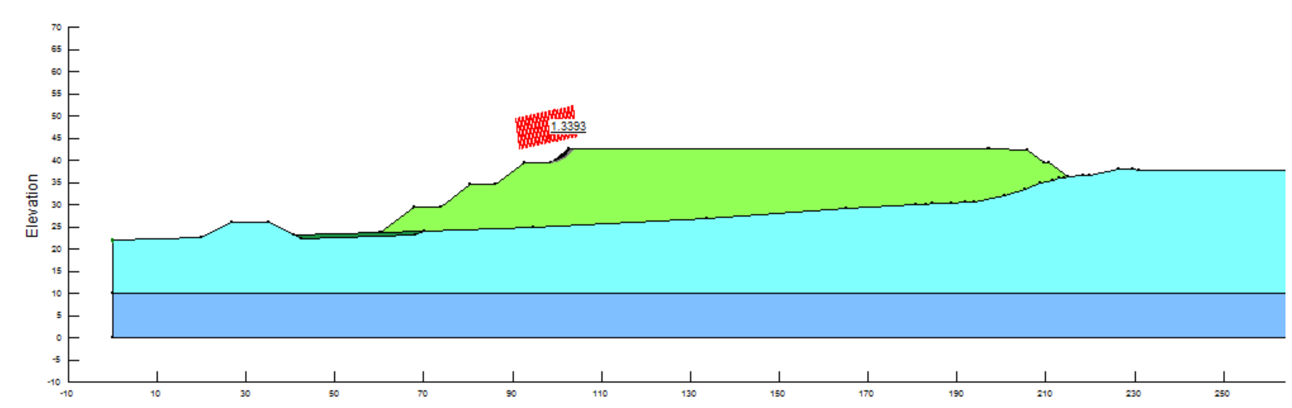
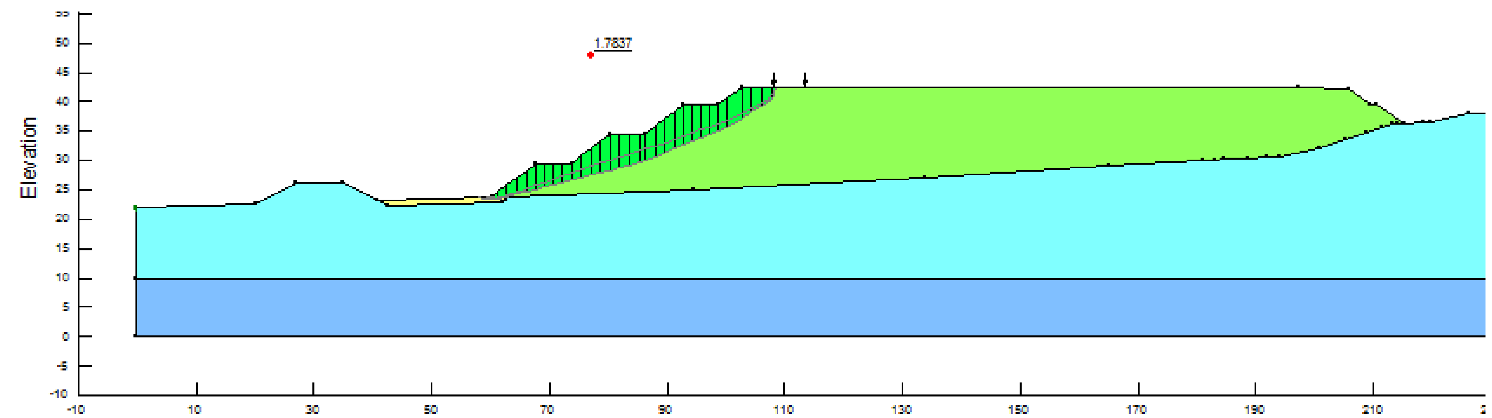
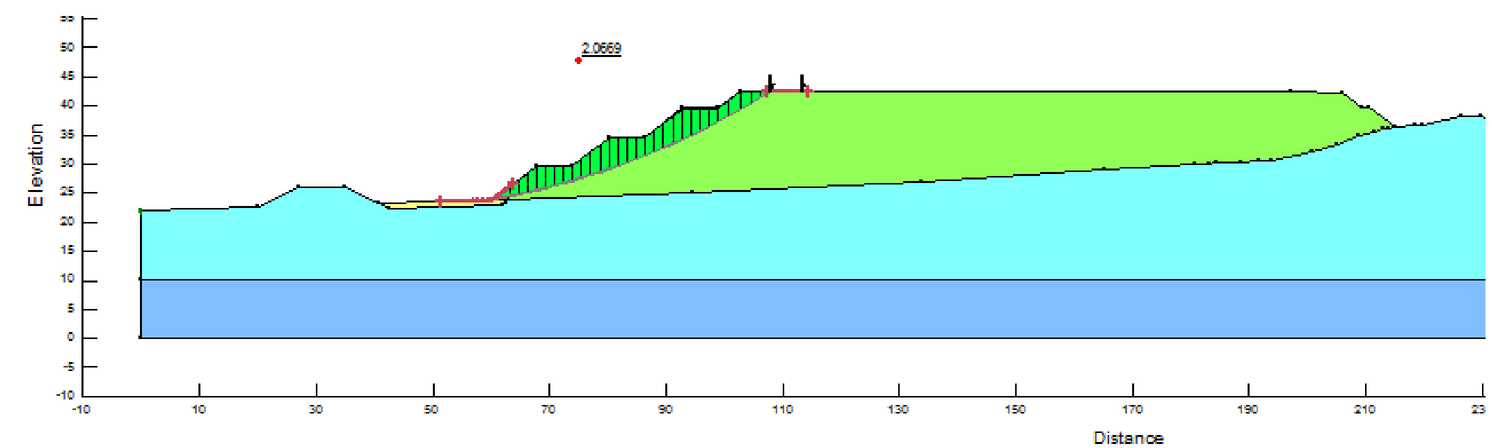
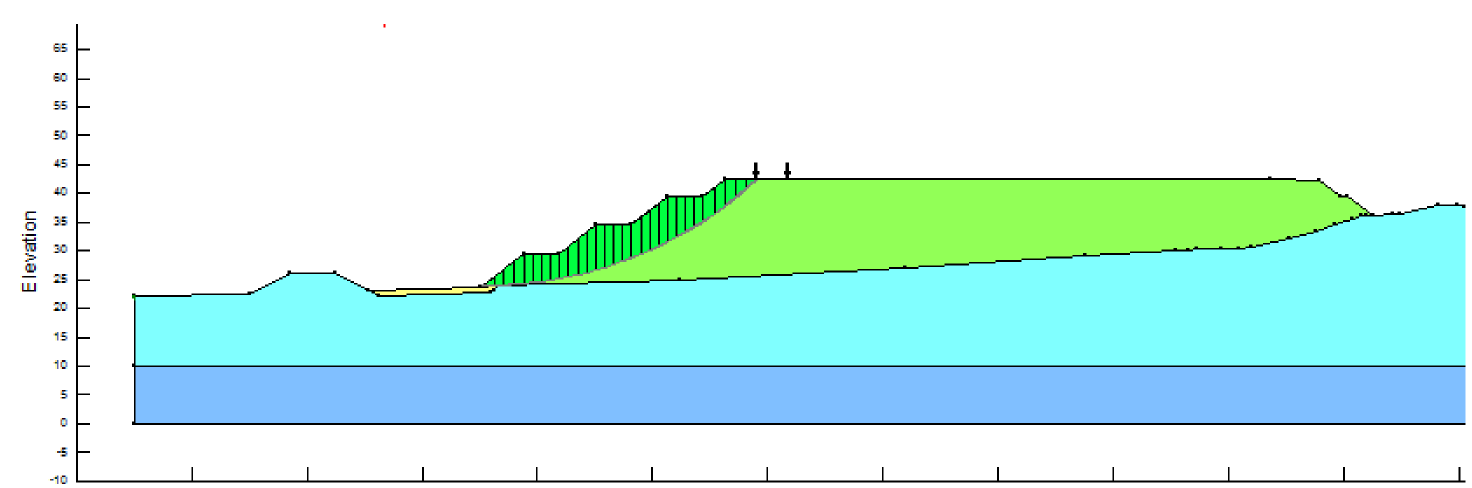
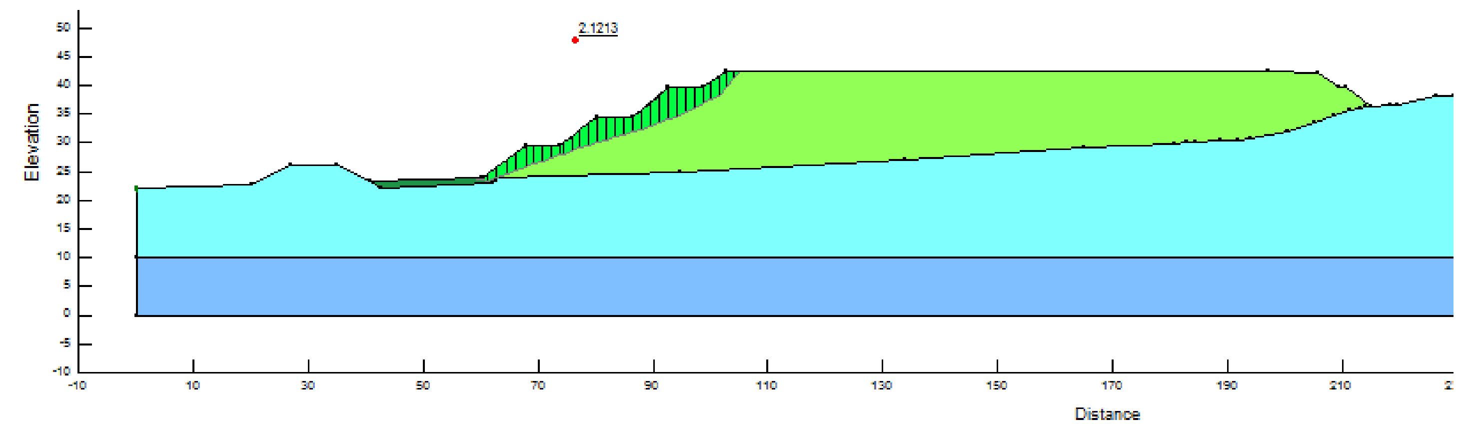
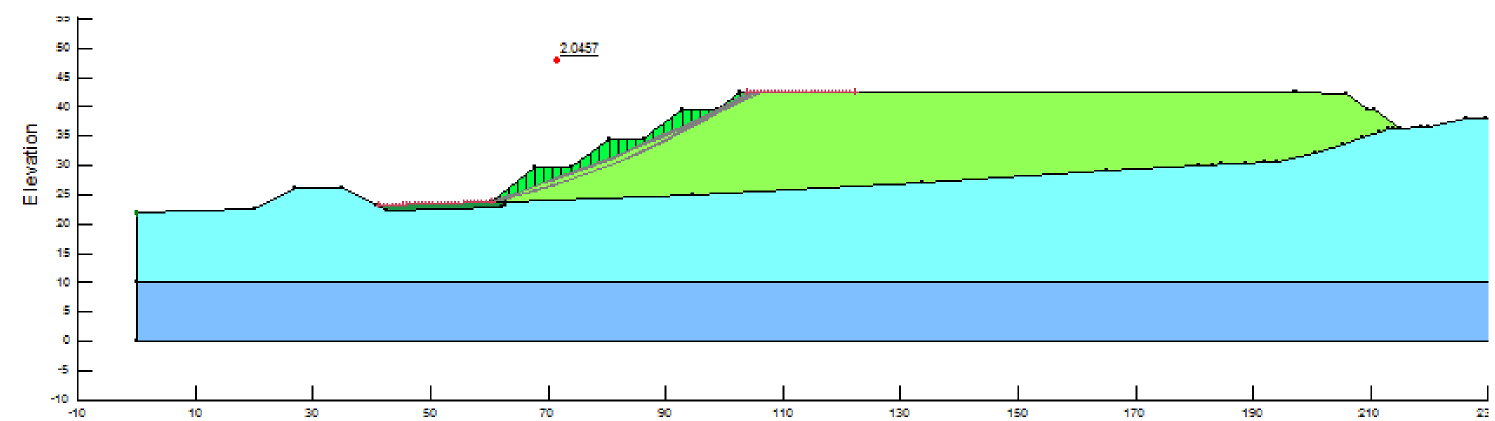
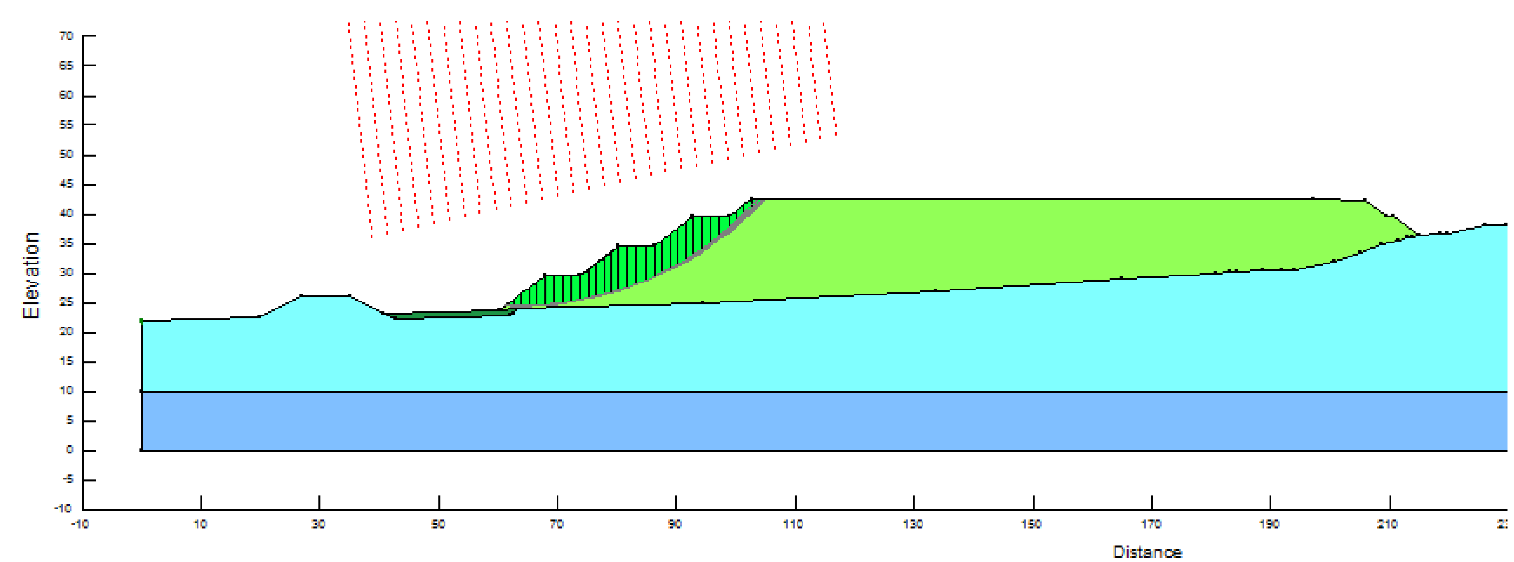
Madrid South Waste Rock Pile Stability Analysis Results – Stability of Dump Surface

Model/Stability Condition	Factor of Safety	Slip Surface Option	Figure
Madrid South/Long Term (Drained Static Condition)	1.6	Fully Defined	
	1.4	Entry Exit	
	1.3	Grid and Radius	

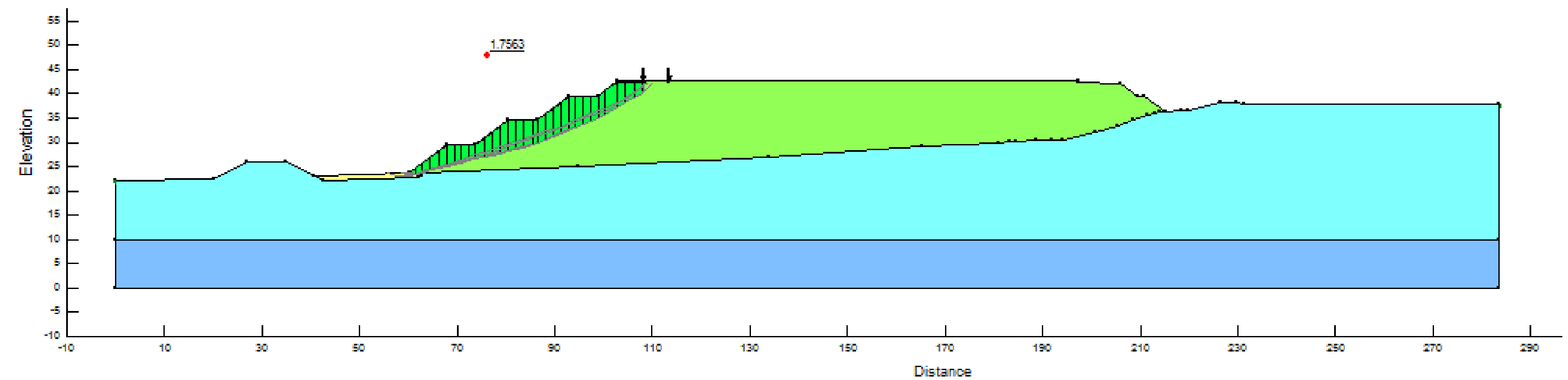
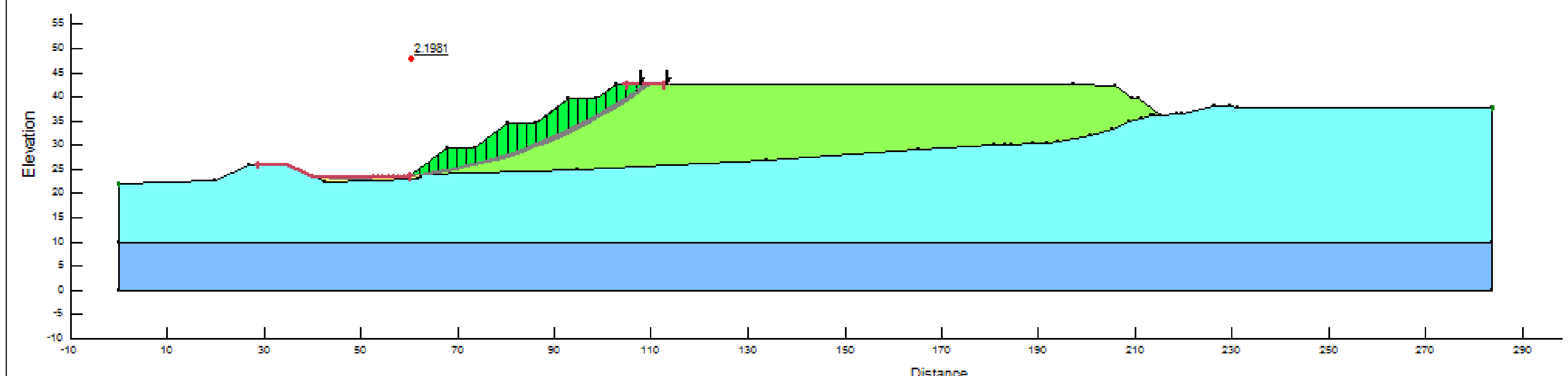
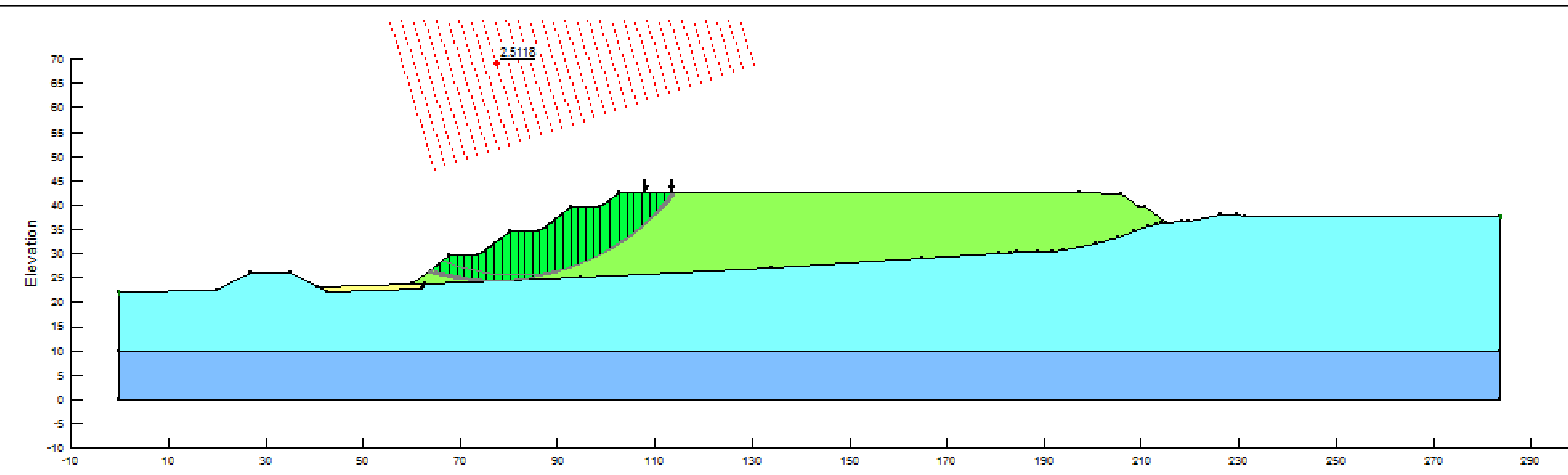
Madrid South Waste Rock Pile Stability Analysis Results – Overall Stability

Model/Stability Condition	Factor of Safety	Slip Surface Option	Figure
Madrid South/Short Term (Undrained Static Condition)	1.8	Fully Defined	
	2.1	Entry Exit	
	2.2	Grid and Radius	

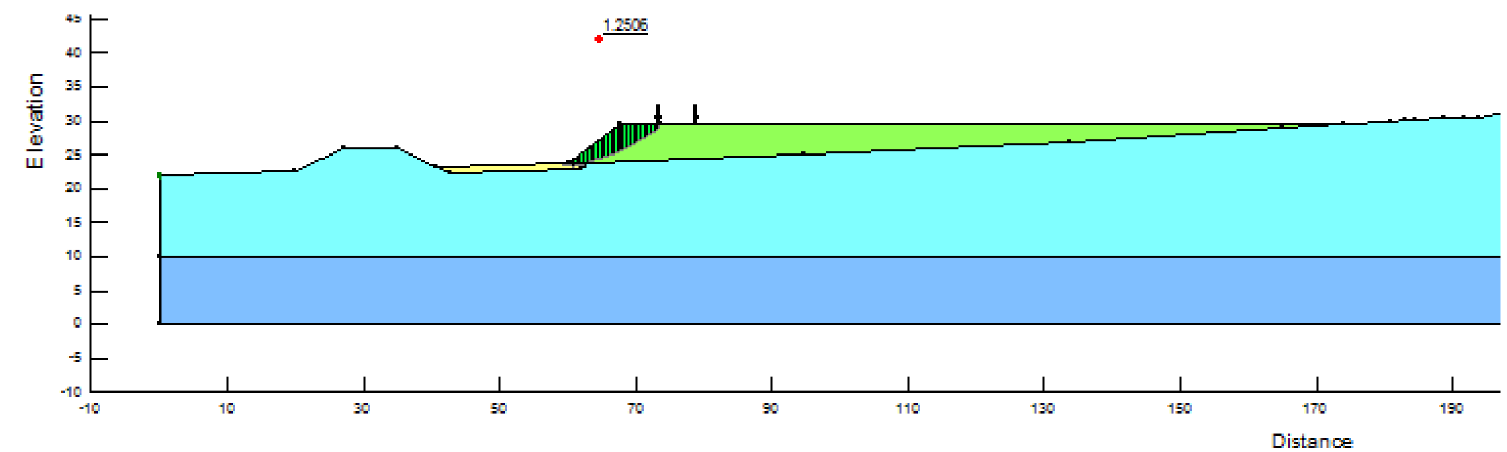
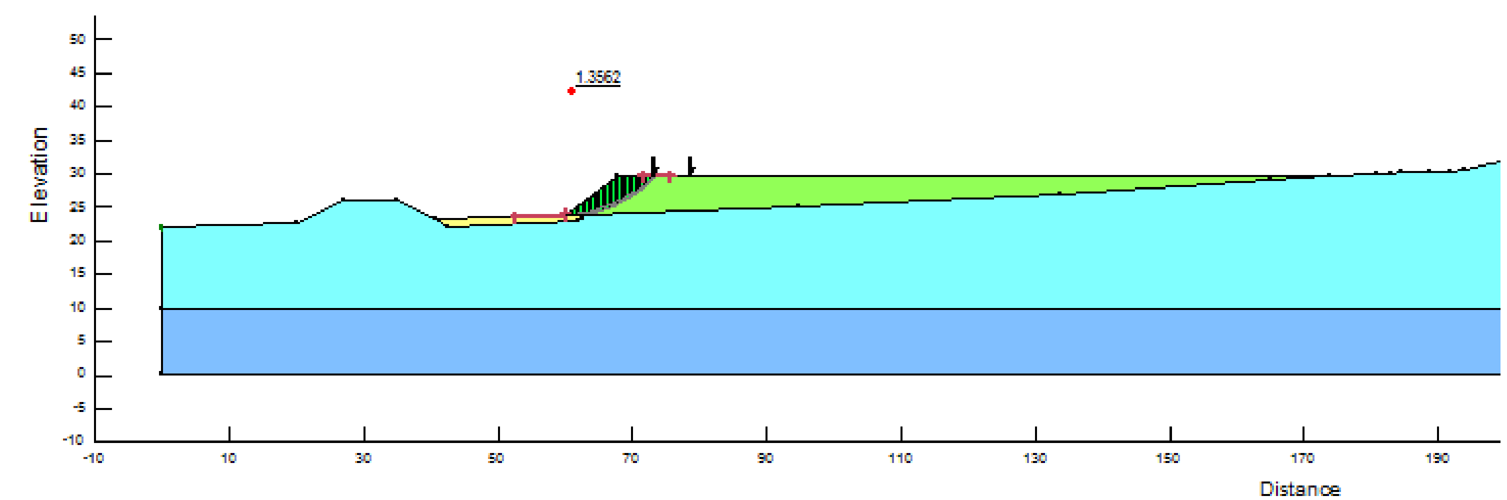
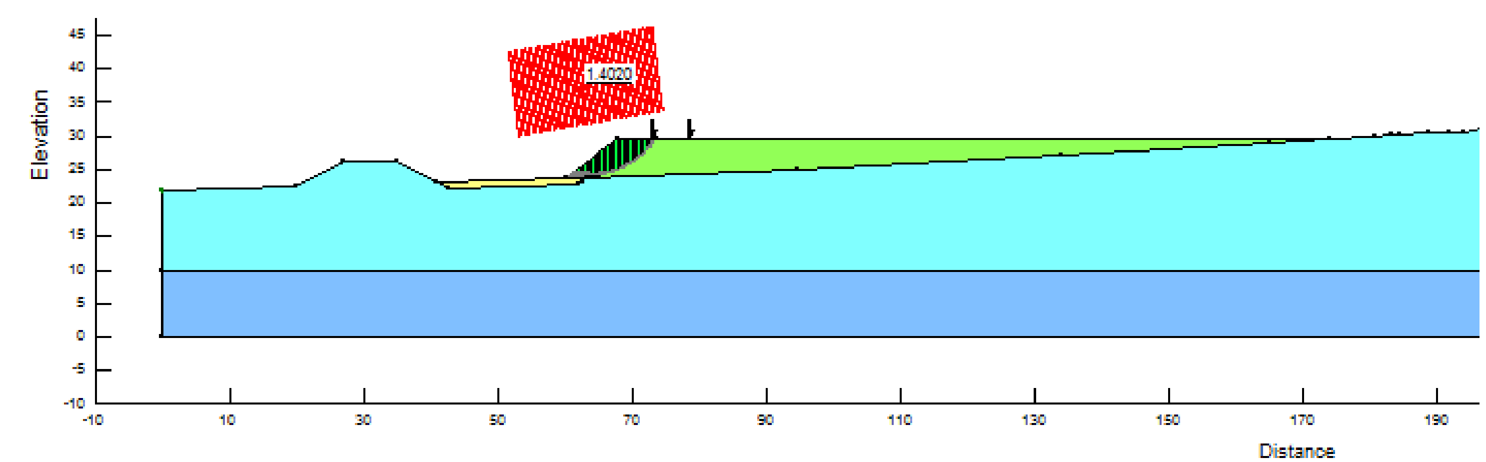
Madrid South Waste Rock Pile Stability Analysis Results – Overall Stability

Model/Stability Condition	Factor of Safety	Slip Surface Option	Figure
Madrid South/Long Term (Drained Static Condition)	2.1	Fully Defined	
	2.0	Entry Exit	
	2.3	Grid and Radius	

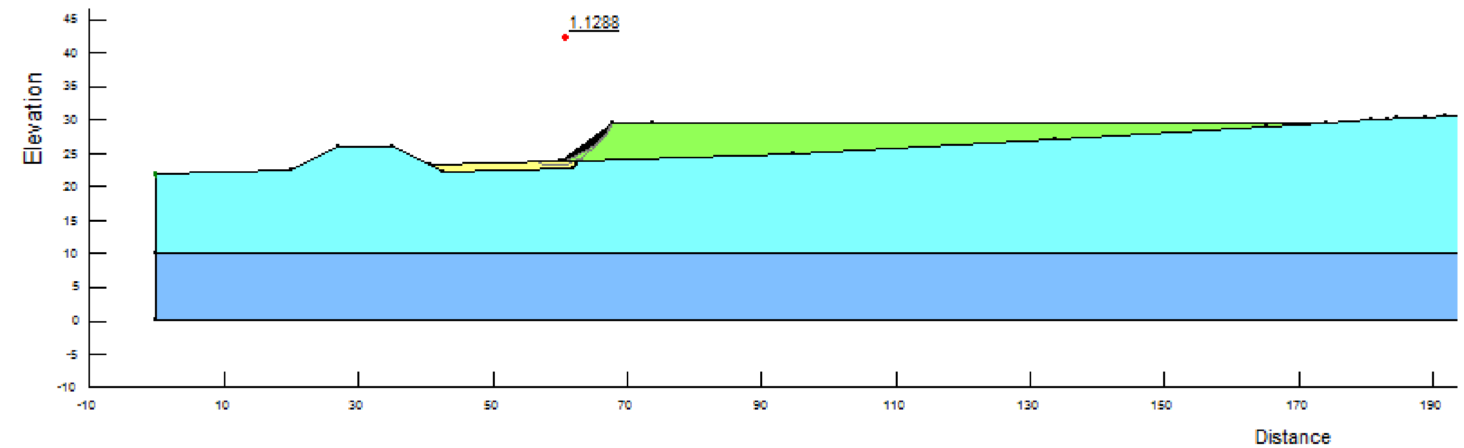
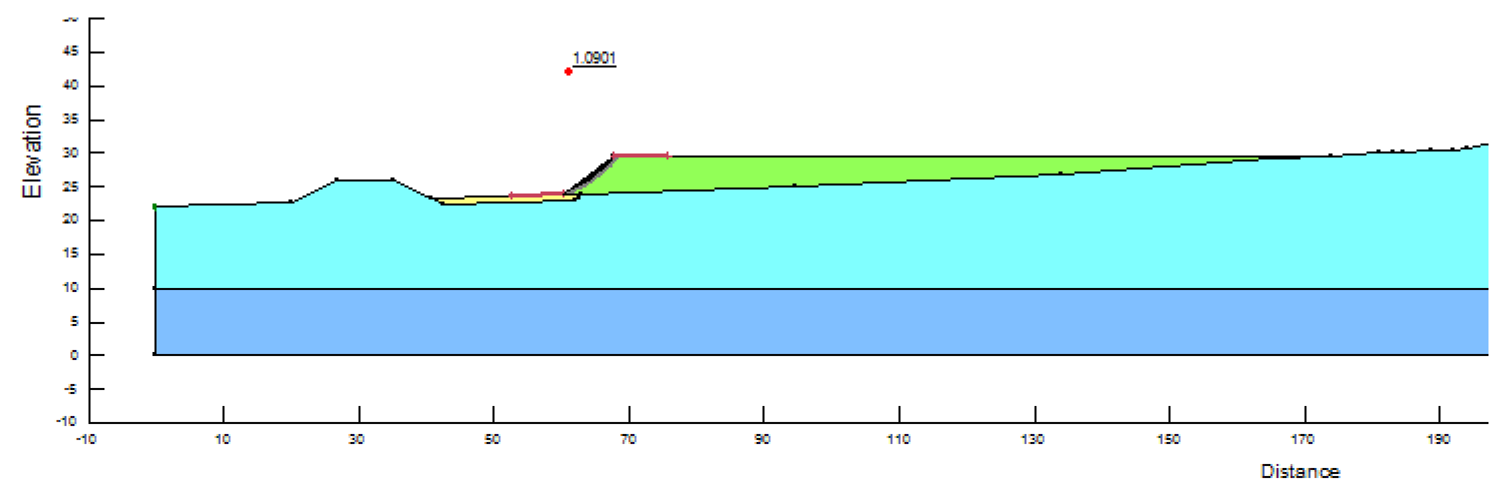
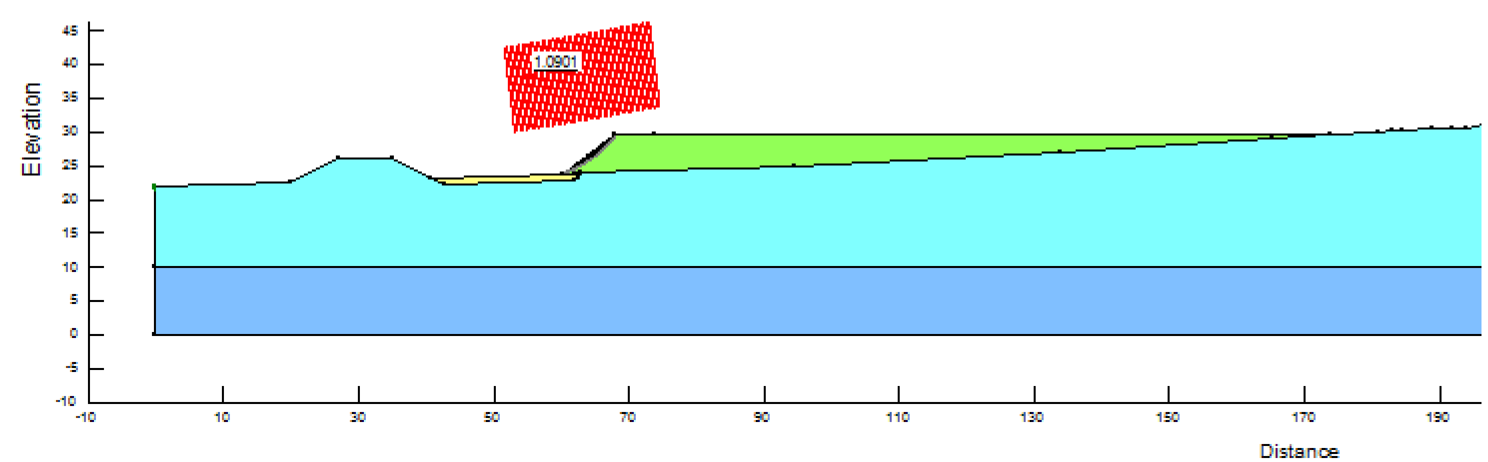
Madrid South Waste Rock Pile Stability Analysis Results – Overall Stability

Model/Stability Condition`	Factor of Safety	Slip Surface Option	Figure
Madrid South/Pseudo-Static (Earthquake)	1.8	Fully Defined	
	2.2	Entry Exit	
	2.5	Grid and Radius	

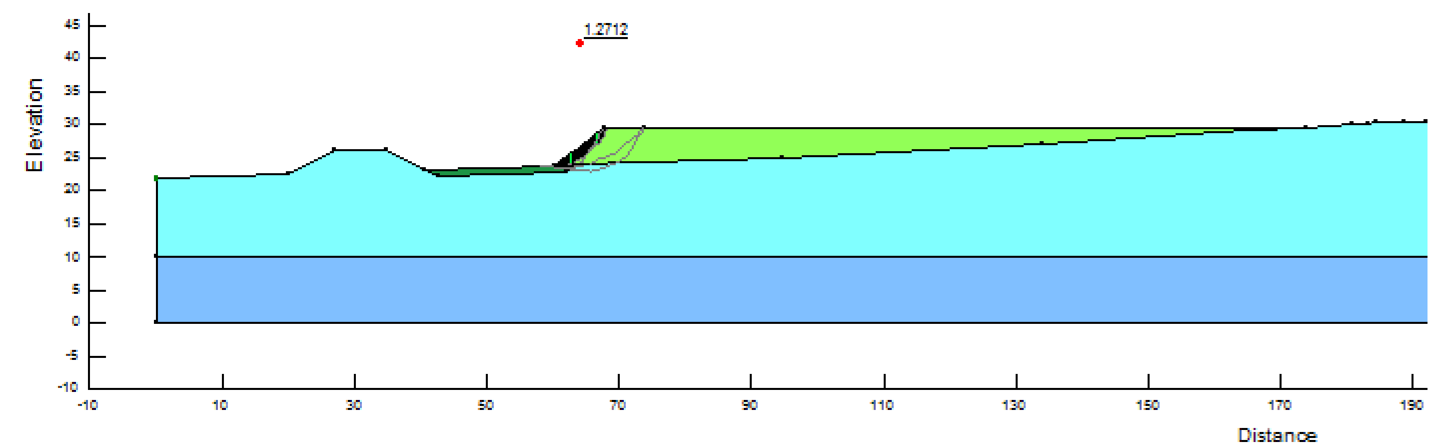
Madrid South Waste Rock Pile Stability Analysis Results – Overall Stability

Model/Stability Condition	Factor of Safety	Slip Surface Option	Figure
Madrid South/1st Bench – Short Term (Undrained Static Condition)	1.3	Fully Defined	
	1.4	Entry Exit	
	1.4	Grid and Radius	

Madrid South Waste Rock Pile Stability Analysis Results – Overall Stability

Model/Stability Condition	Factor of Safety	Slip Surface Option	Figure
Madrid South/1st Bench –Short Term (Undrained Static Condition) **Without Load**	1.1	Fully Defined	
	1.1	Entry Exit	
	1.1	Grid and Radius	

Madrid South Waste Rock Pile Stability Analysis Results – Overall Stability

<table>
<tr><th>Model/Stability Condition</th><th>Factor of Safety</th><th>Slip Surface Option</th><th>Figure</th></tr>
<tr><td rowspan="3">Madrid South/1st Bench –Short Term (Drained Static Condition)</td><td>1.3</td><td>Fully Defined</td><td>
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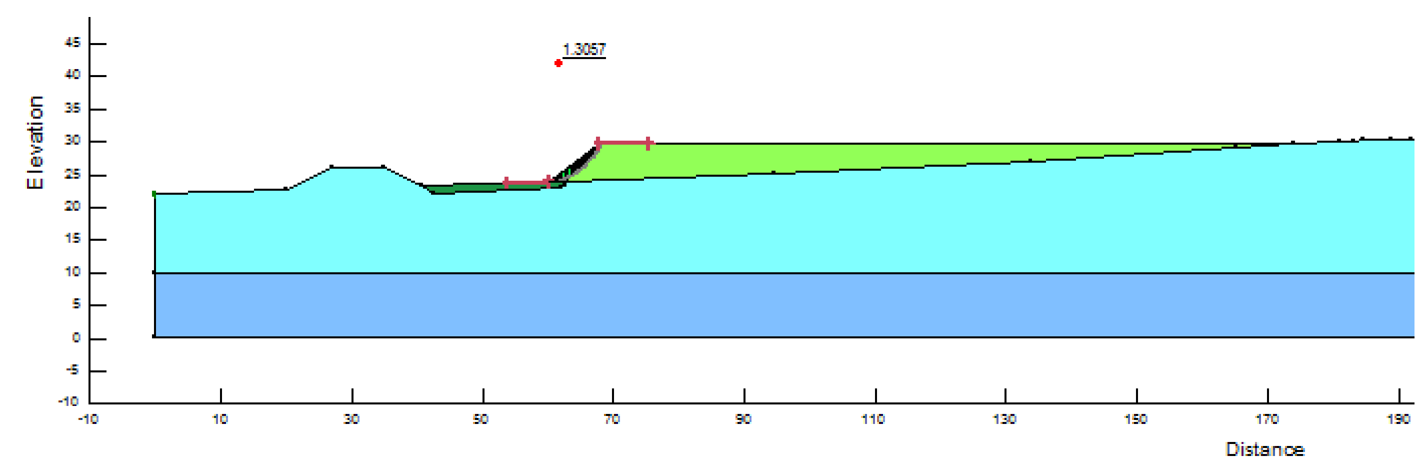
<tr><td>1.3</td><td>Entry Exit</td><td>
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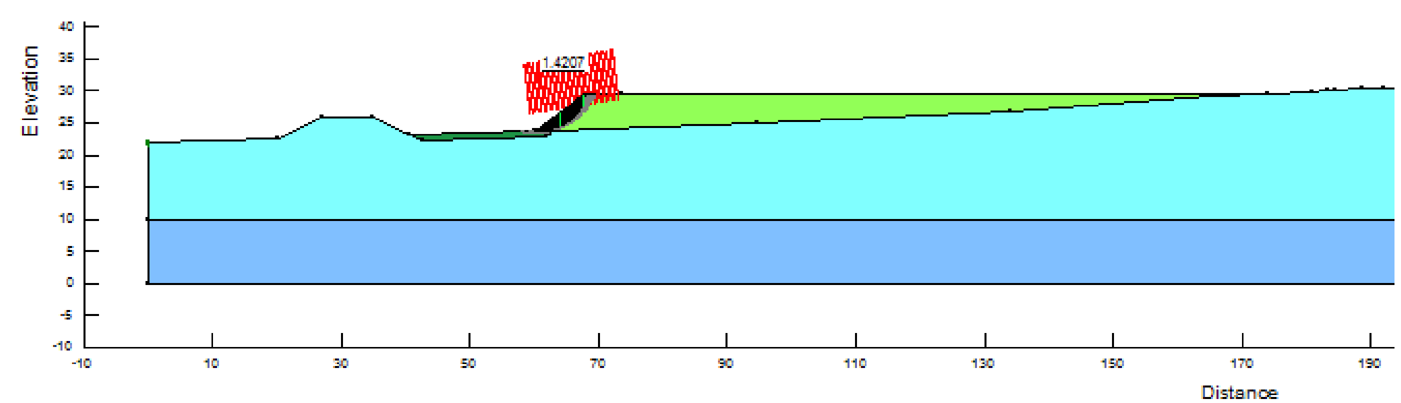
<tr><td>1.4</td><td>Grid and Radius</td><td>
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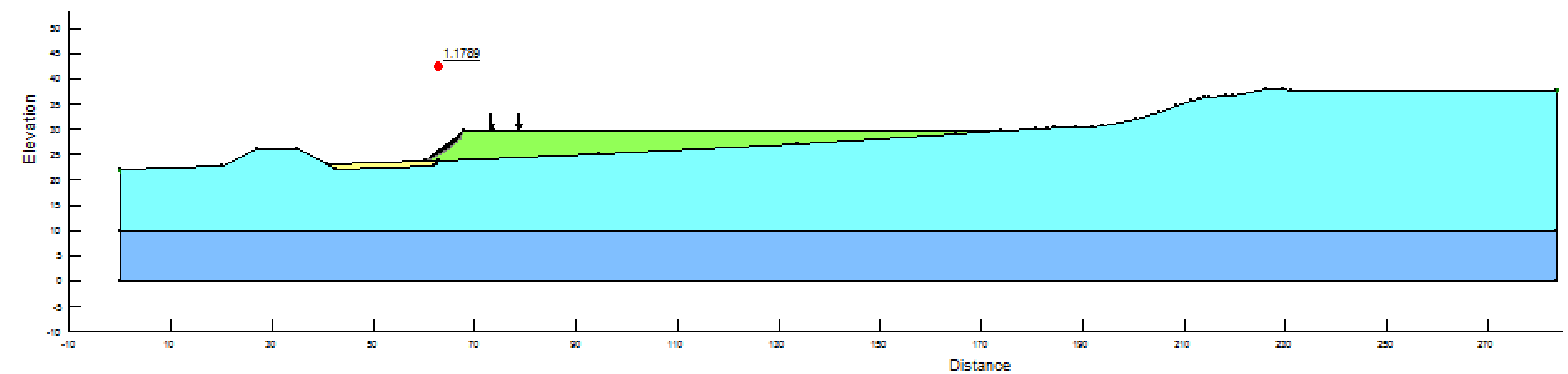
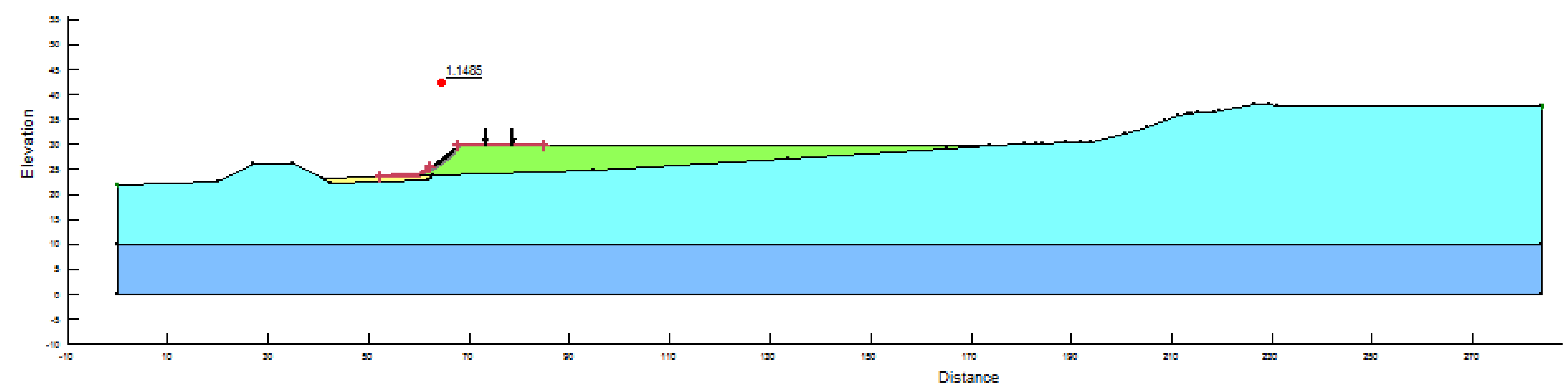
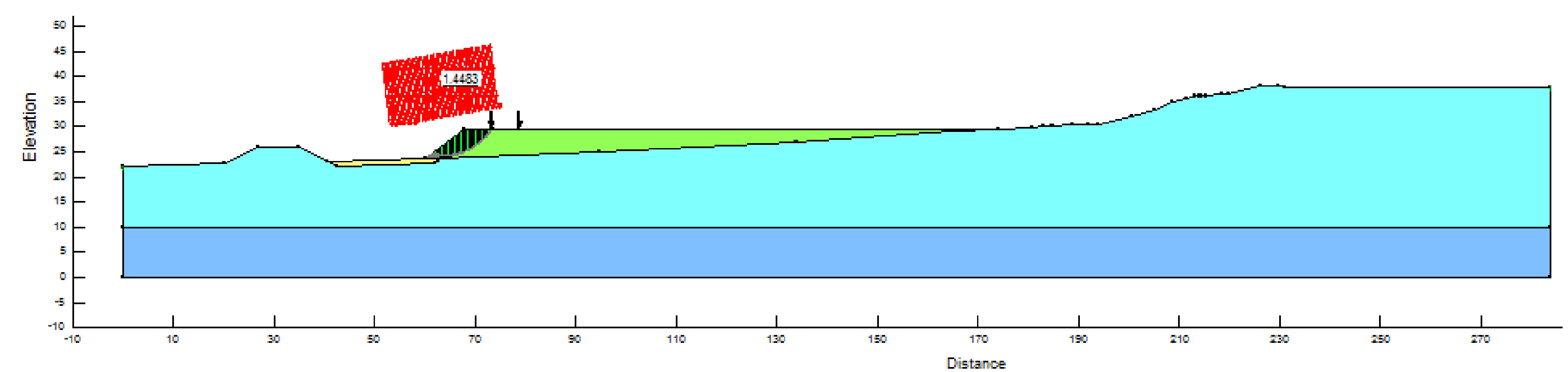
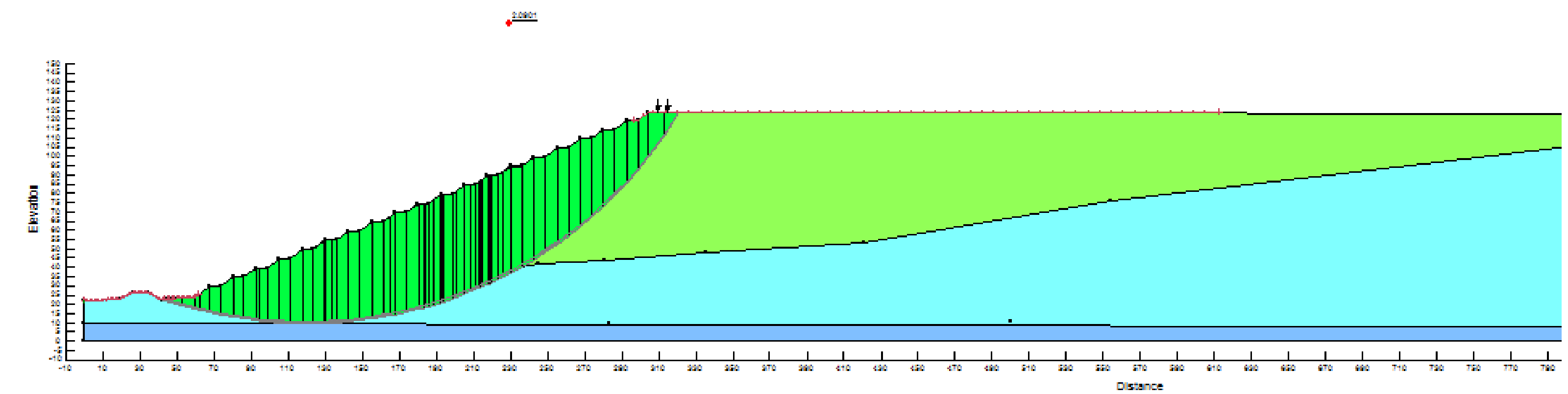
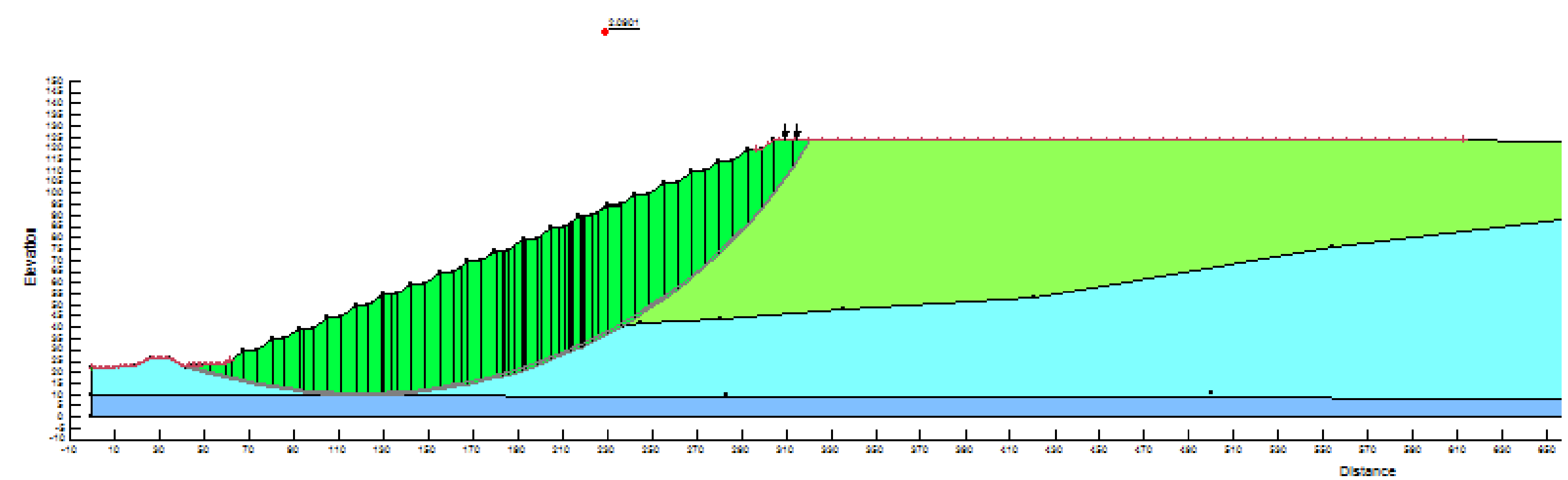
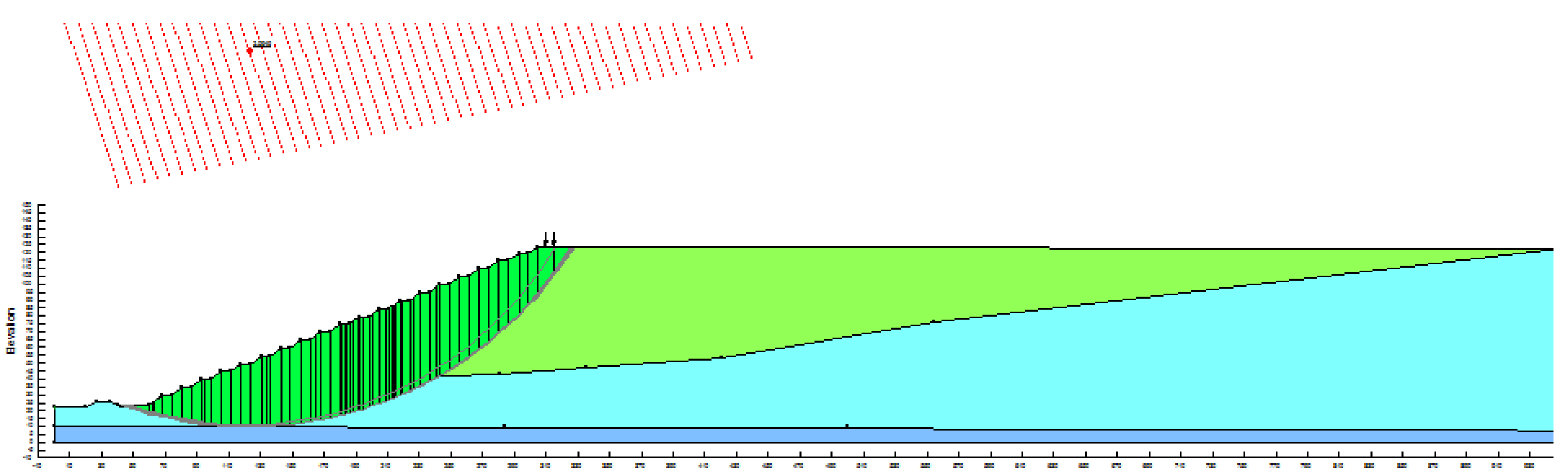
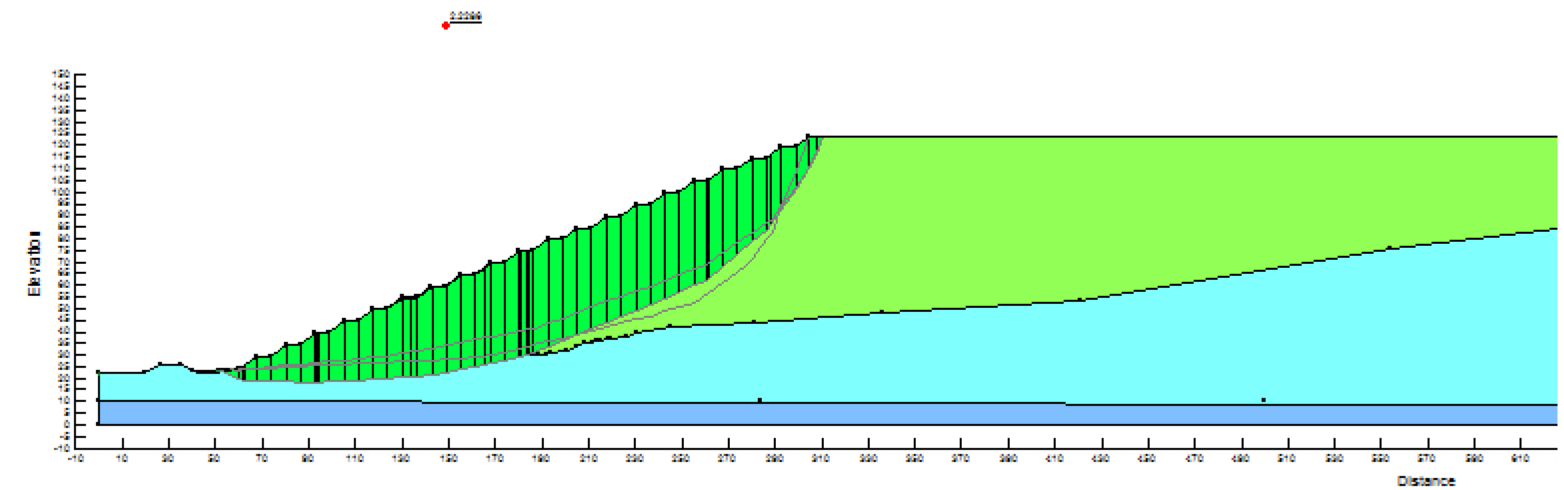
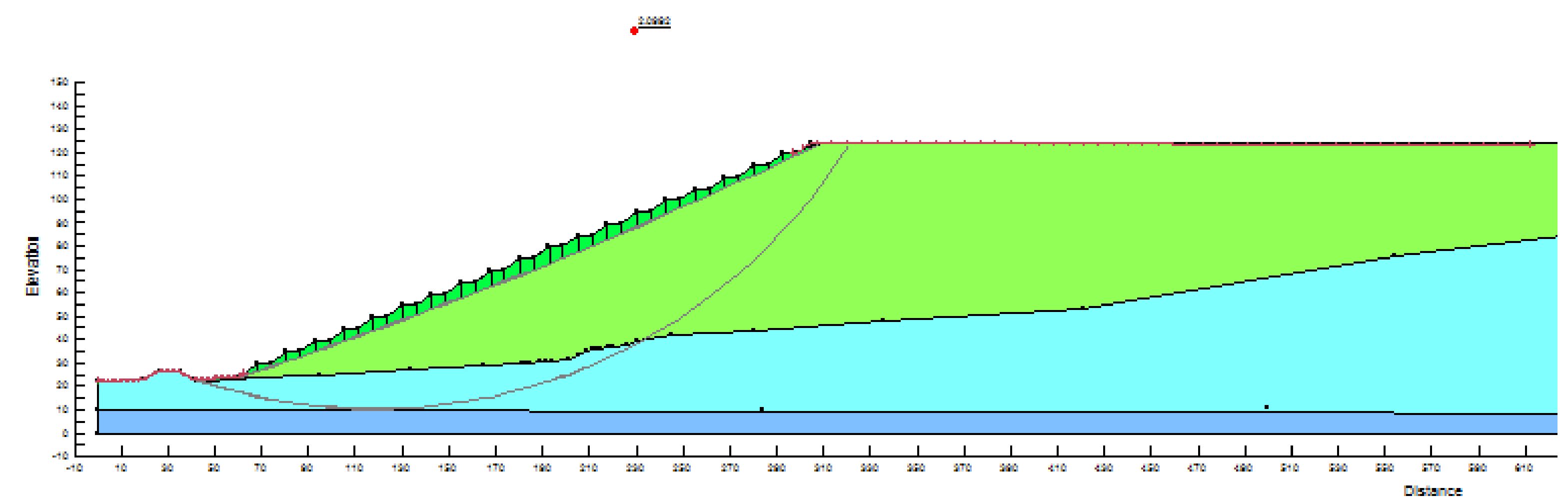
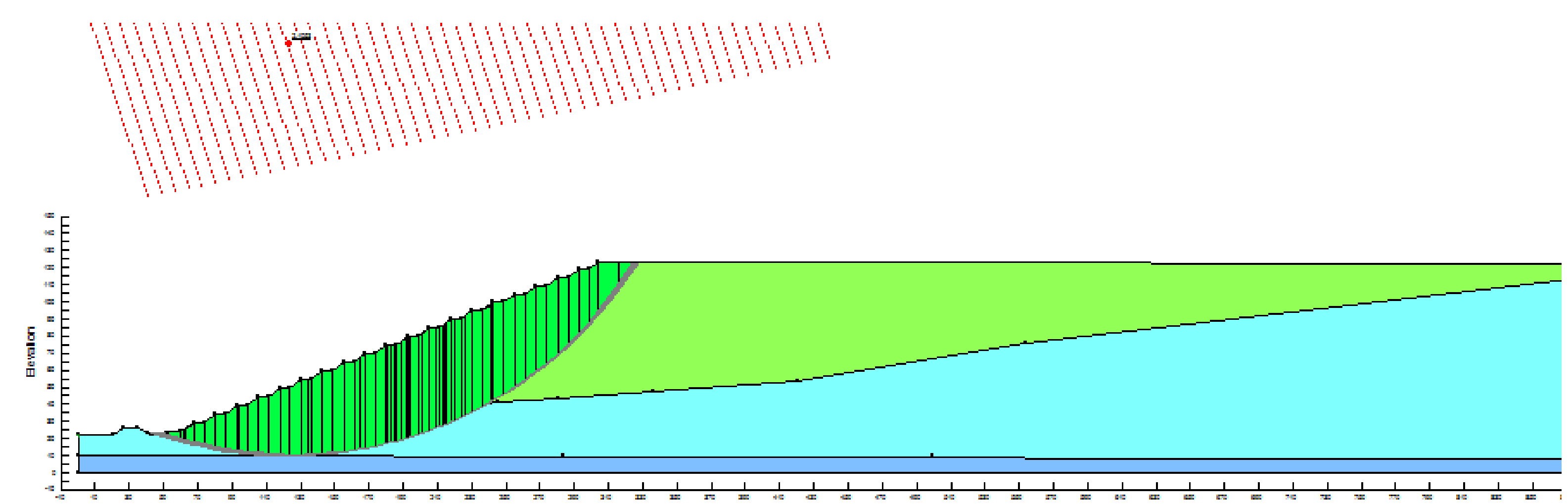
Madrid South Waste Rock Pile Stability Analysis Results – Overall Stability

Model/Stability Condition	Factor of Safety	Slip Surface Option	Figure
Madrid South/1st Bench –Pseudo-Static (Earthquake)	1.2	Fully Defined	
	1.1	Entry Exit	
	1.4	Grid and Radius	

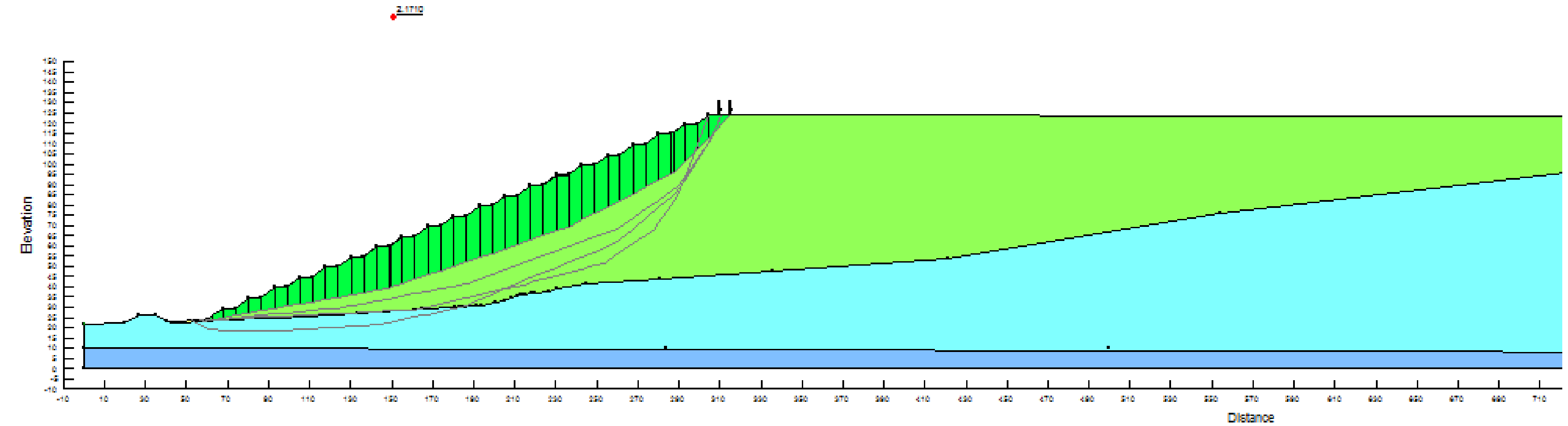
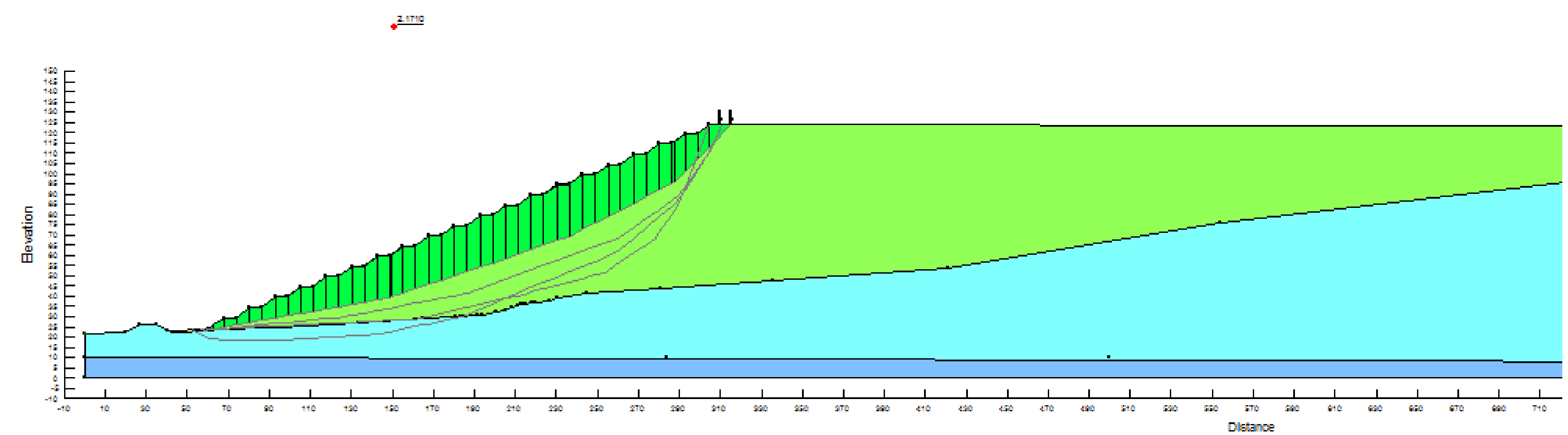
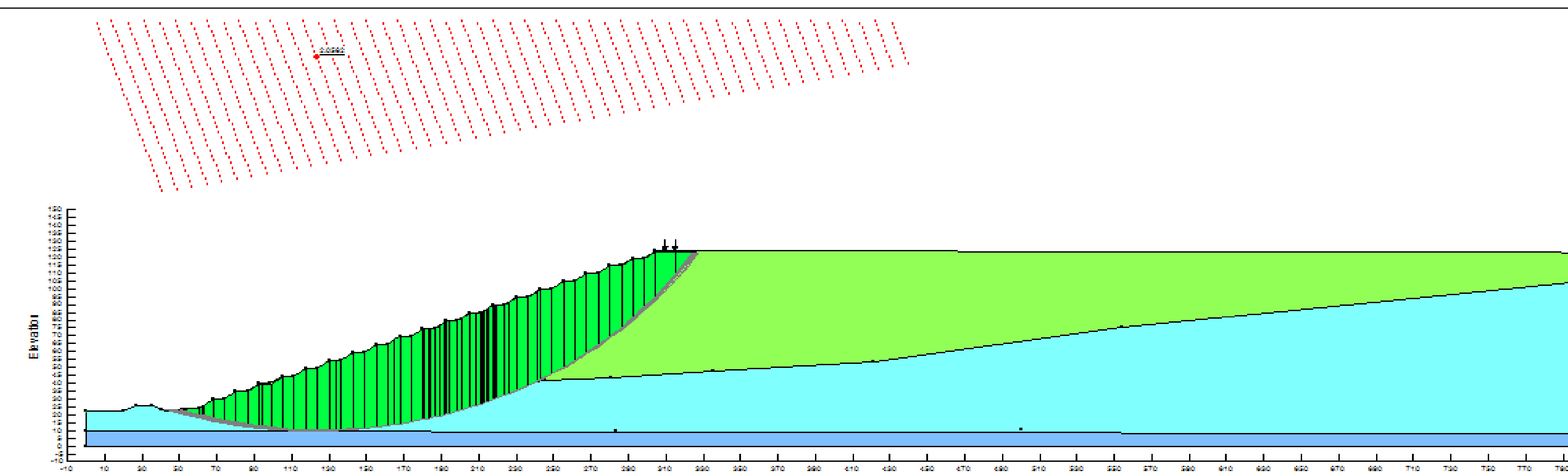
Madrid South Waste Rock Pile Stability Analysis Results – Max Height Overall Stability

Model/Stability Condition	Factor of Safety	Slip Surface Option	Figure
100m High WRD - Short Term (Undrained Static Condition)	2.2	Fully Defined	
	2.1	Entry Exit	
	2.1	Grid and Radius	

Madrid South Waste Rock Pile Stability Analysis Results – Max Height Overall Stability

Model/Stability Condition	**Factor of Safety**	**Slip Surface Option**	**Figure**
100m High WRD – Long Term (Undrained Static Condition)	2.2	Fully Defined	
	2.1	Entry Exit	
	2.1	Grid and Radius	

Madrid South Waste Rock Pile Stability Analysis Results – Max Height Overall Stability

Model/Stability Condition	Factor of Safety	Slip Surface Option	Figure
100m High WRD - Pseudo-Static (Earthquake)	2.2	Fully Defined	
	2.1	Entry Exit	
	2.1	Grid and Radius	

Hope Bay - Waste Rock Pile Stability Analysis Result Summary

Madrid South Waste Rock Pile (Maximum Height)

	Stability Condition	Loading Condition	Recommended FOS	Resulting FOS
Stability of Dump Surface	Short Term	Undrain (with truck load)	1	1.1
	Long Term	Drain	1.2	1.3
Overall Stability of Dump	Short Term	Undrain (with truck load)	1.3-1.5	1.8
	Long Term	Drain	1.5	2.0
	Pseudo-Static	Undrain (with truck load)	1.1-1.3	1.8

The distance of the truck load to the crest = 5.5m

Madrid South Waste Rock Pile (First Bench)

	Stability Condition	Loading Condition	Recommended FOS	Resulting FOS
First Bench Stability of Dump Surface	Short Term	Undrain (with truck load)	1	1.1
	Long Term	Drain	1.2	1.3
	Pseudo-Static	Undrain (with truck load)	1.1-1.3	1.1

100m High Waste Rock Pile

	Stability Condition	Loading Condition	Recommended FOS	Resulting FOS
WRD Height: 100m	Short Term	Undrain (with truck load)	1.3-1.5	2.1
	Long Term	Drain	1.5	2.1
	Pseudo-Static	Undrain (with truck load)	1.1-1.3	2.1

Mined Rock and Overburden Piles

Investigation and Design Manual

Interim Guidelines

May, 1991

British Columbia
Mine Waste
Rock Pile
Research
Committee

TABLE 6.4

INTERIM GUIDELINES FOR MINIMUM DESIGN FACTOR OF SAFETY [1]

STABILITY CONDITION	SUGGESTED MINIMUM DESIGN VALUES FOR FACTOR OF SAFETY	
	CASE A	CASE B
STABILITY OF DUMP SURFACE		
–Short Term (during construction)	1.0	1.0
–Long Term (reclamation – abandonment)	1.2	1.1
OVERALL STABILITY (DEEP SEATED STABILITY)		
–Short Term (static)	1.3 – 1.5	1.1 – 1.3
–Long Term (static)	1.5	1.3
–Pseudo–Static (earthquake) [2]	1.1 – 1.3	1.0

CASE A:
- –Low level of confidence in critical analysis parameters
- –Possibly unconservative interpretation of conditions, assumptions
- –Severe consequences of failure
- –Simplified stability analysis method (charts, simplified method of slices)
- –Stability analysis method poorly simulates physical conditions
- –Poor understanding of potential failure mechanism(s)

CASE B:
- –High level of confidence in critical analysis parameters
- –Conservative interpretation of conditions, assumptions
- –Minimal consequences of failure
- –Rigorous stability analysis method
- –Stability analysis method simulates physical conditions well
- –High level of confidence in critical failure mechanism(s)

NOTES: 1. A range of suggested minimum design values are given to reflect different levels of confidence in understanding site conditions, material parameters, consequences of instability, and other factors.

2. Where pseudo–static analyses, based on peak ground accelerations which have a 10% probability of exceedance in 50 years, yield F.O.S. < 1.0, dynamic analysis of stress–strain response, and comparison of results with stress–strain characteristics of dump materials is recommended.