Integrated Geotechnical Feasibility Analysis for an Open Pit Mine in the Canadian Arctic

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ABSTRACT

This paper presents the geotechnical results of a comprehensive feasibility case study for the Tiriganiaq pit of the Meliadine project in the Canadian arctic. A geomechanical model was constructed based on the ore reserve block model, and field and laboratory data collection campaigns. The results of oriented core logging were used to identify and zone the mine. Working in parallel with the pit optimization process a series of comprehensive slope stability analyses were undertaken on a block by block basis on selected optimised pits. The innovation of the undertaken approach resides in the smooth integration of the geological and geotechnical models with the optimised ultimate pit geometry to provide input for the slope stability analysis. Rather than working with “representative” slope geometries the actual planned slope geometries were used to compute the slope orientation at bench and inter-ramps levels using Geographical Information Systems (GIS) algorithms. This has allowed the stability analysis of the complete 3D numerical mine model and facilitates the identification of potentially unstable zones. The stability analyses were based on deterministic and probabilistic limit equilibrium techniques. It was possible to investigate the stability of all benches and inter-ramps for the ultimate pit defined by the block model. The factor of safety (FS) and probability of failure (PF) were assessed for every block of the optimised pit. In order to quantify the impact of the prevailing geotechnical conditions on the proposed pit shells a series of multi-criteria stability analyses were employed to assess the potential for localised instability.

KEYWORDS

Open-pit mining, Interramp orientation, Bench face orientation, GIS, Integrated design, Block modeling

CITATION

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1 INTRODUCTION

The Meliadine gold project in Nunavut Territory in northern Canada (Fig. 1) is Agnico-Eagle’s second major project in Canada’s Low Arctic following the Meadowbank gold mine. The project has probable gold reserves of 2.6 million ounces (9.5 million tonnes grading 8.5 grams/tonne gold) as of December 31, 2010. Meliadine is located on a large property with significant exploration upside and has regional synergies with Meadowbank, just 290 kilometres to the northwest.

A good part of the Tiriganiaq deposit will be mined through open pit. The projected ultimate pit, obtained at the feasibility study, was 1050 m by 390 m in size with a depth of 155 m. Within the block model, the pit surface is defined by 2376 blocks of a size of 25 m by 5 m by 5 m.

This paper reports on how a series of stability analyses were integrated during the feasibility studies of the Tiriganiaq pit of the Meliadine project. Instead of analysing the stability at selected locations along the pit, as it is often the case, stability analyses were undertaken on all blocks of the optimised pit. This allowed the construction of stability maps identifying zones of concern for a given pit design for both the bench and inter- ramp level. This methodology makes use of the detailed pit design geometry and all available geotechnical data. Furthermore, multi-criteria stability can be applied to identify potential instability zones.

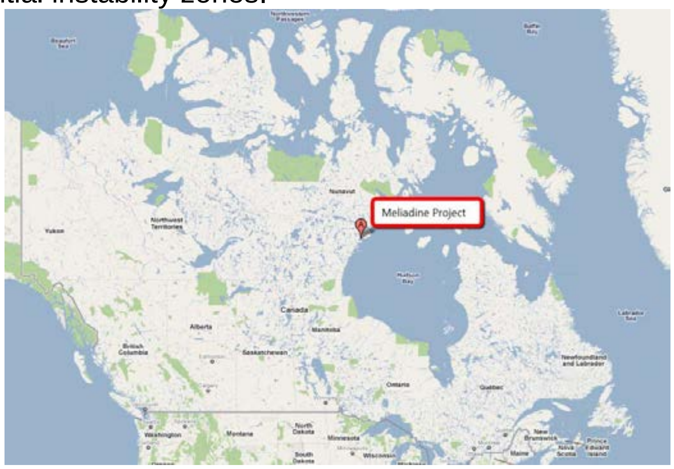


Figure 1. Map of Meliadine advanced-stage gold project in Nunavut Territory, constructed from Google.

2 SLOPE STABILITY ANALYSES

2.1 Mechanical and structural properties

The mechanical and structural properties were obtained through line mapping, oriented drilling and laboratory testing. The pit walls were in mafic volcanic rocks and graywacke. A summary of the structural and mechanical properties within the Tiriganiaq pit is presented in Table 1. The entire pit area, however, was defined by a single structural domain. Four fracture sets were identified with the most dominant one being the foliation. In order to be consistent with the way structural data were collected on-site, both J1 and J3 are represented in Table 1. Fracture orientations were defined by their mean dip and dip direction and a Fisher coefficient, while the mean trace length values were obtained through face mapping in underground access drifts. Finally, laboratory direct shear tests were used to estimate the friction angle and cohesion for selected fractures. An average value of 2.8 t/m^3 was used for the specific gravity of the rock masses.

Table 1. Structural properties within the Tiriganiaq pit, after Golder (2010).

Joint Set	Dip (deg)	Dip Direction (deg)	Fisher Coefficient	Friction Angle (deg)	Cohesion (t/m^2)	Mean trace Length (m)
Foliation	67	003	172	35	0	29
J0	24	184	64	42	8	3
J1/J3	75	100	36	42	8	3
J2	36	271	17	42	8	4

2.2 Integrated stability analysis methodology

The fundamental basis of the approach chosen for the stability analysis for the Meliadine gold project lies in the data management of geological and geomechanical data, outlined in Figure 2. Pertinent geometric, geological and geomechanical data, including: x, y and z block coordinates, material density, rock type, structural domain etc., were stored in the block model. Furthermore, a digital elevation model (DEM) was constructed to provide a three-dimensional representation of the planned ultimate pit topography.

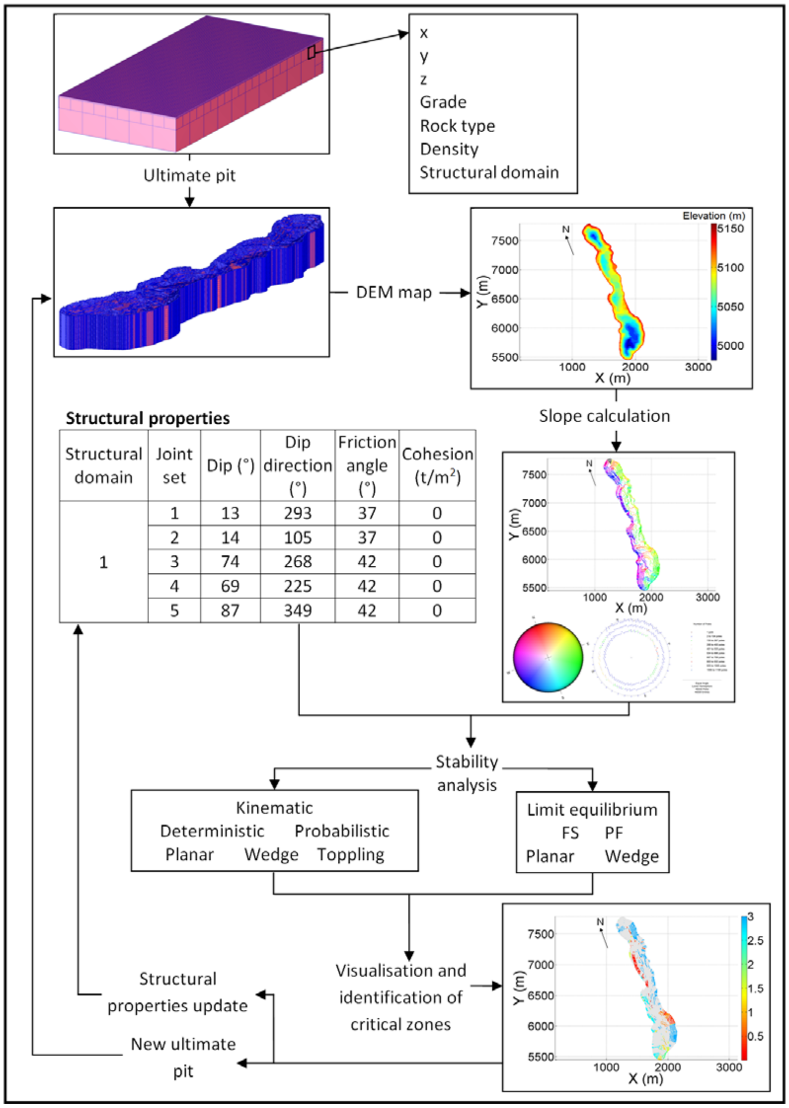


Figure 2. General methodology for integrated slope stability analysis.

Previous work had indicated that assessing slope orientation at bench and inter-ramp levels was a complex process. For the purposes of the Meliadine pit, the principal component analysis, in the Matlab statistics toolbox (MathWorks, 2010), was used. This allows better determination of the true slope orientation at individual bench and inter-ramp levels.

Bench and inter-ramp stability analyses for the whole pit were undertaken using the defined pit slopes and by retrieving the information from the geotechnical database. Slope stability at the bench and inter-ramp levels was undertaken using both kinematic and limit equilibrium algorithms. The developed algorithms allowed the visualisation of the results of stability on susceptibility maps, identifying areas of concern. Whenever the geotechnical or geometrical information is updated, new analyses can be performed, and the stability can be reassessed. This is an on-going process.

2.3 Slope orientation using Geographical Information Systems (GIS)

GIS-based algorithms can be used to determine the orientation of slopes in open-pit mines. Based on previous work reported by Grenon & Laflamme (2011) principal component analysis (PCA)-based methods were used to evaluate slope orientations at the inter-ramp and bench level. In PCA the sampling window, centered on the cell where slope orientation is computed, is not limited to nine cells like in most of the common GIS techniques.

For the purposes of the Meliadine pit, the principal component analysis, in the Matlab statistics toolbox (MathWorks, 2010), was used to fit a plane on a set of 3D points. Based on the calibration methodology discussed in Grenon & Laflamme (2011) a sampling window of 13 x 3 cells was used for inter-ramp orientation and a sampling size of 5 x 3 cells for bench orientation.

The inter-ramp slope orientation is presented in Figure 3 employing the color-coding legend proposed by Jaboyedoff et al. (2009). The stereonets present inter-ramp slope orientations at every cell of the pit DEM. It is easy to identify the north and south walls of the pit using pole clusters. Using these visualization techniques, it is possible to establish the variability in inter-ramp angles.

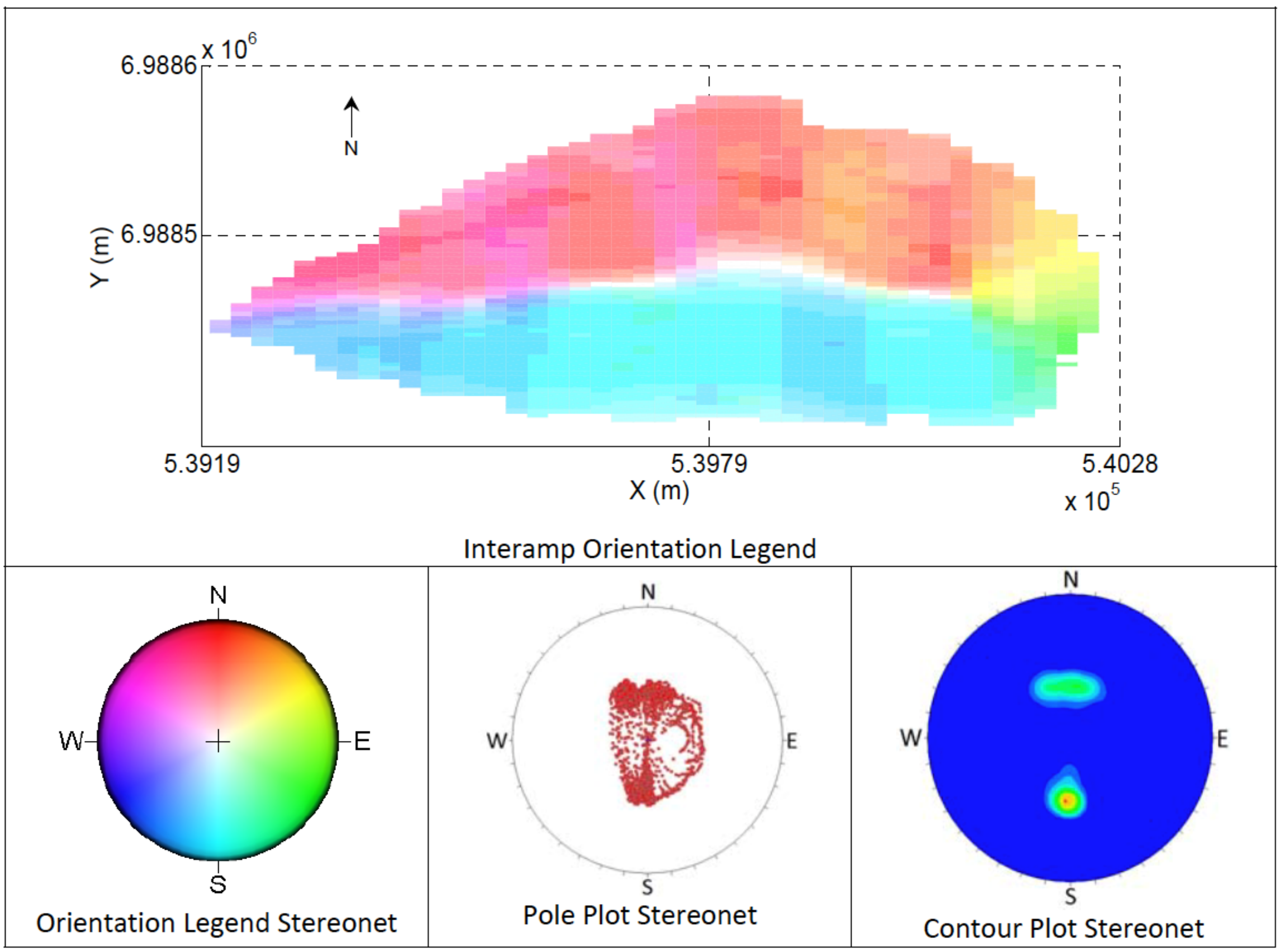
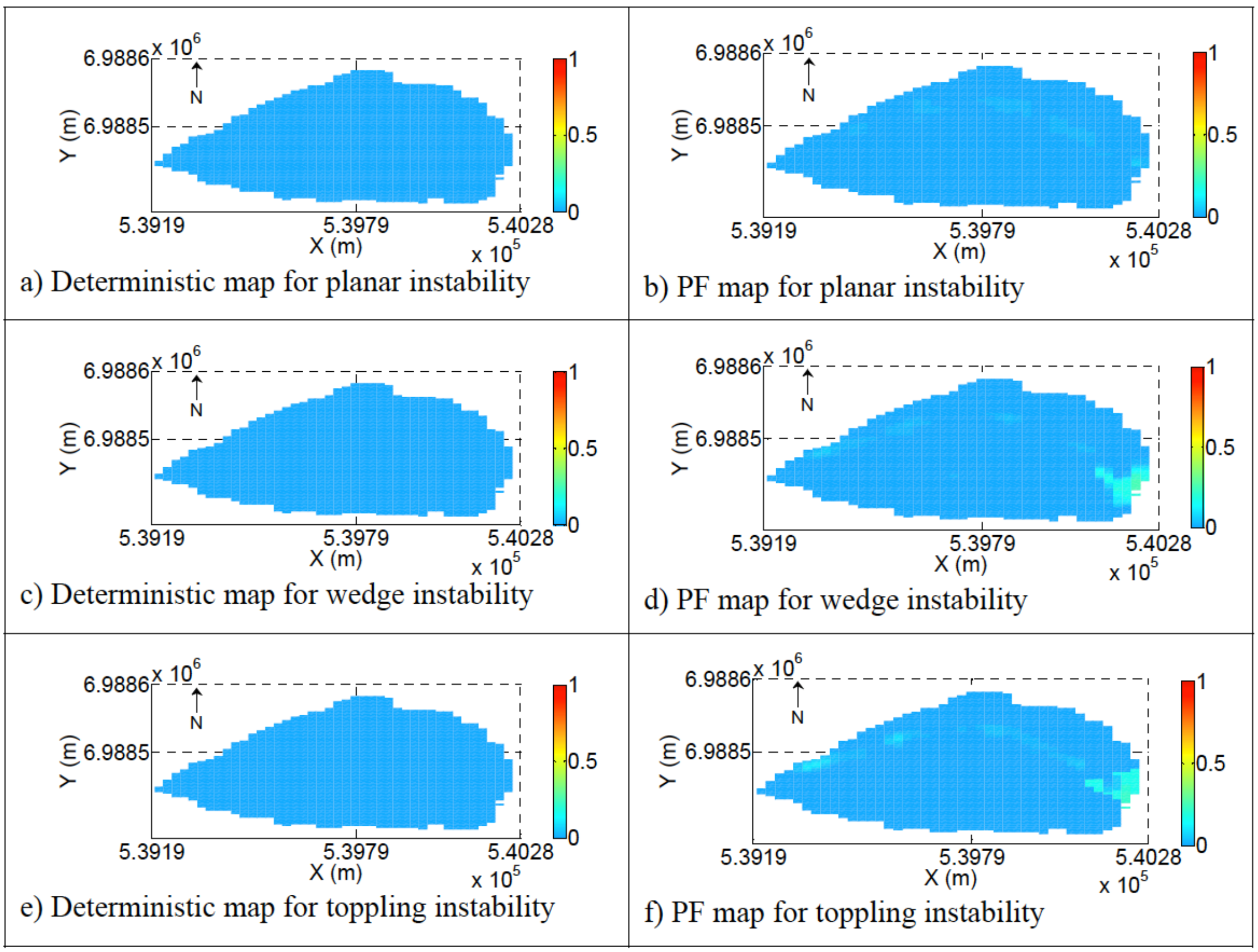


Figure 3. Inter-ramp slope orientation map.

2.4 Kinematic analysis

The stability of the pit can be assessed once the slope orientation and structural characteristics are defined for every cell. The first stage in the stability investigation of the pit is a kinematic analysis. Hudson & Harrison (1997) describe the necessary steps to establish whether any slope geometry and fracture orientation combination results in a kinematically feasible slope failure system. Kinematic analyses are commonly employed to establish the plausibility of planar, wedge and toppling instability slope failure mechanisms. Based on the available geomechanical data at Meliadine, the kinematic analysis employed the mean values for all structural properties (deterministic analysis) and was complemented by a probabilistic analysis using Monte-Carlo sampling on the probability density function (PDF) of the various structural properties listed in Table 1.

Figure 4 illustrates a series of maps identifying areas susceptible to planar, wedge and toppling mechanisms at the inter-ramp bench levels. The results of the deterministic analyses are presented in Figures 4a, 4c and 4e where every cell can either be stable or unstable. Based on the available structural data all areas of the pit were stable. All potential failure modes analysis are indicating that no such potential exist for mean structural properties. Figures 4b, 4d and 4f illustrate stability maps based on the probabilistic analyses. Only a very limited number of cells, along the south wall of the pit have a small probability of instability (PF).



a) Deterministic map for planar instability

b) PF map for planar instability

c) Deterministic map for wedge instability

d) PF map for wedge instability

e) Deterministic map for toppling instability

f) PF map for toppling instability

Figure 4. Slope stability at the inter-ramp level.

The results of the kinematic analysis for the inter-ramp levels are summarised in Table 2. The deterministic analyses do not identify any areas of instability throughout the pit, supporting the interpretation that there are no major stability concerns. In fact, only a very limited number of cells show a PF above 10% for either planar, wedge or toppling failure. The practical implication of this analysis is that if production requirements dictate it, it is possible to pursue a more aggressive approach.

Table 2. Analysis results at the inter-ramp level for kinematic instability.

	Total number of cells	**Planar instability**		**Wedge instability**		**Toppling instability**	
		Number of cells	% in overall pit surface	Number of cells	% in overall pit surface	Number of cells	% in overall pit surface
Deterministic analysis	2376	0	0	0	0	0	0
PF>10%	2376	0	0	64	2.7	64	2.7
PF>20%	2376	0	0	11	0.5	6	0.25
PF>25%	2376	0	0	2	0	0	0

2.5 Limit equilibrium analysis

Limit equilibrium analysis (LEA) are often the next step in investigating the stability of slopes. LEA can be used to evaluate either the factor of safety (FS) or the probability of failure (PF). The theory of LEA is well described in standard textbooks such as Hudson & Harrison (1997) and Wyllie & Mah (2004). At Meliadine the rock mass is of good quality, not fragmented, therefore making the analysis for a circular type of failure unlikely. A toppling type of LEA analysis is currently not implemented in the developed platform.

Figure 5 presents the LEA maps for planar and wedge instabilities. The undertaken analysis did not investigate variations in the hydrogeological model. It can be observed in Figure 5a that for mean structural properties, the resulting FS for planar instabilities, over the entire pit area, is greater than 1.5. Gray cells indicate that no kinematically feasible planar failures were possible. In Figure 5c, the FS is above 3 when considering mean structural properties for wedge type instabilities. Figures 5b and 5d are stability maps reporting the resulting PF values at every cell. Figure 5b and 5d demonstrate that that the PF for planar wedge instabilities, based on the available information, is nil throughout the entire pit area.

2.6 Acceptance design criteria

It is increasingly popular to use FS and PF values to determine whether a particular mine slope design is acceptable. Recently, Read & Stacey (2009) suggested a series of typical acceptance criteria values for various slope scales, based on the consequence of failure, Table 3. For example, where the consequence of failure of an inter-ramp slope is moderate, the minimum FS should be 1.2 and the maximum PF should be 20%. Figure 6 presents the results for the various criteria at the inter-ramp level for static conditions. The maps in Figures 6a to 6f indicate that the design is acceptable.

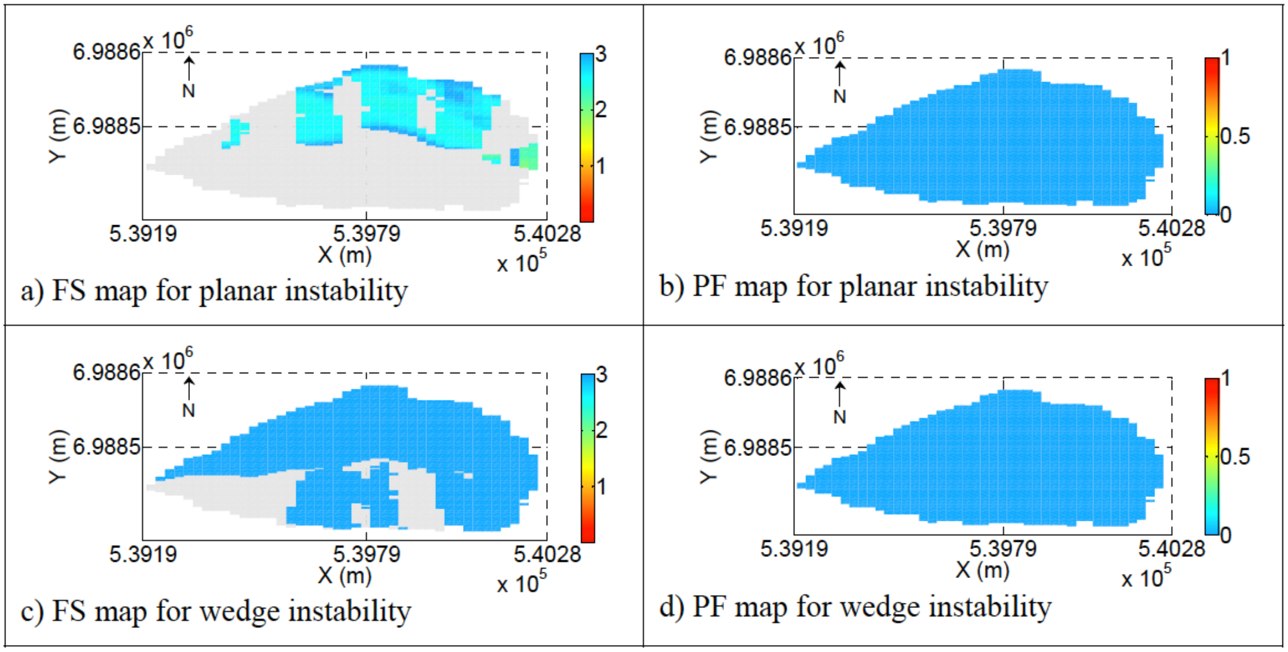
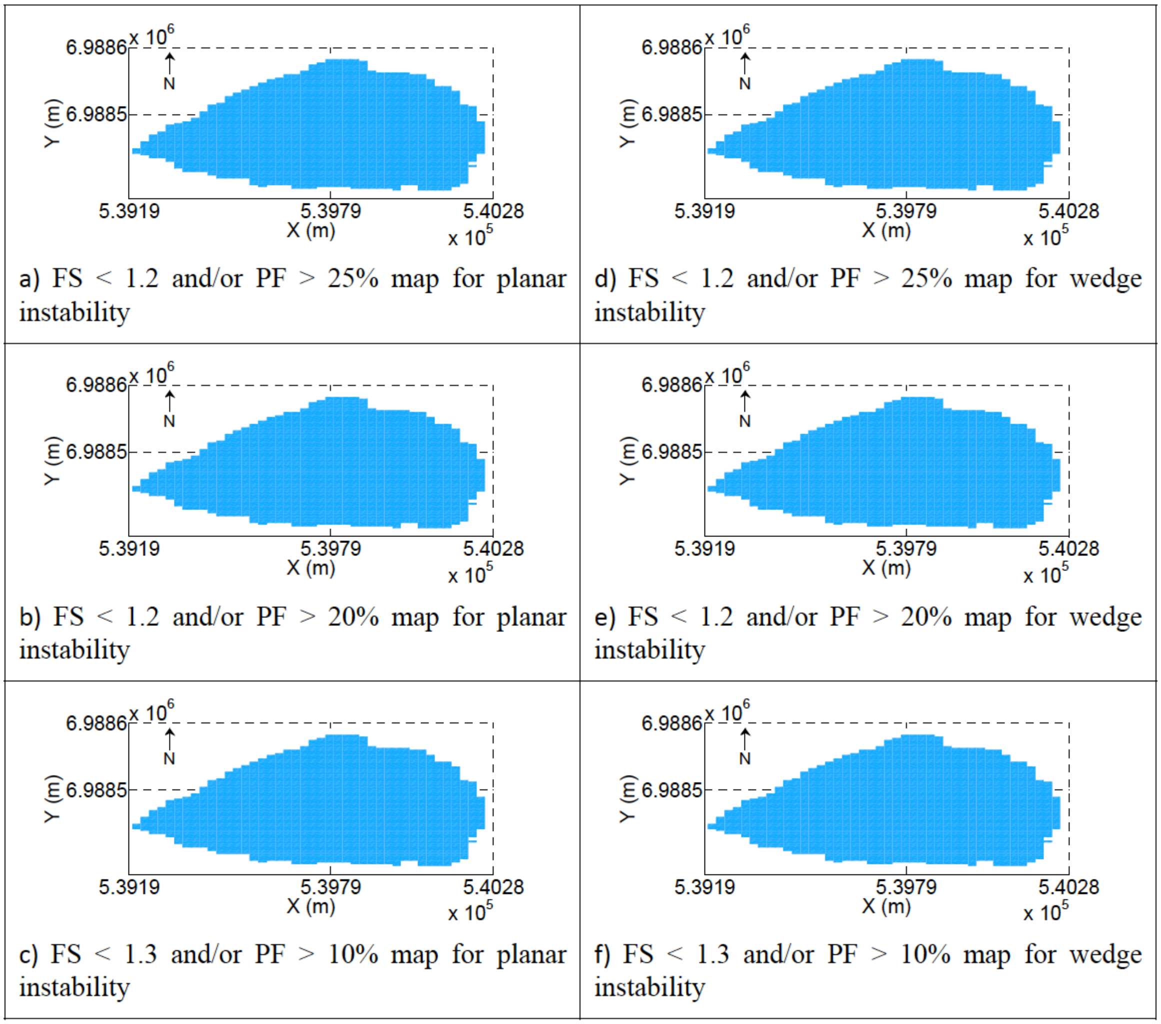


Figure 5. Slope stability at the inter-ramp level.

Table 3. Typical FS and PF acceptance criteria values from Read & Stacey (2009).

Slope scale	**Consequence of failure**[2]	**Acceptance criteria**[1]		
		FS (min) (static)	FS (min) (dynamic)	PF (max) P[FS≤ 1]
Bench	Low-high	1.1	NA	25-50%
Inter-ramp	Low	1.15-1.2	1.0	25%
	Moderate	1.2	1.0	20%
	High	1.2-1.3	1.1	10%
Overall	Low	1.2-1.3	1.0	15-20%
	Moderate	1.3	1.05	10%
	High	1.3-1.5	1.1	5%

[1]Needs to meet all acceptance criteria.
[2]Semi-quantitatively evaluated.



a) FS < 1.2 and/or PF > 25% map for planar instability

d) FS < 1.2 and/or PF > 25% map for wedge instability

b) FS < 1.2 and/or PF > 20% map for planar instability

e) FS < 1.2 and/or PF > 20% map for wedge instability

c) FS < 1.3 and/or PF > 10% map for planar instability

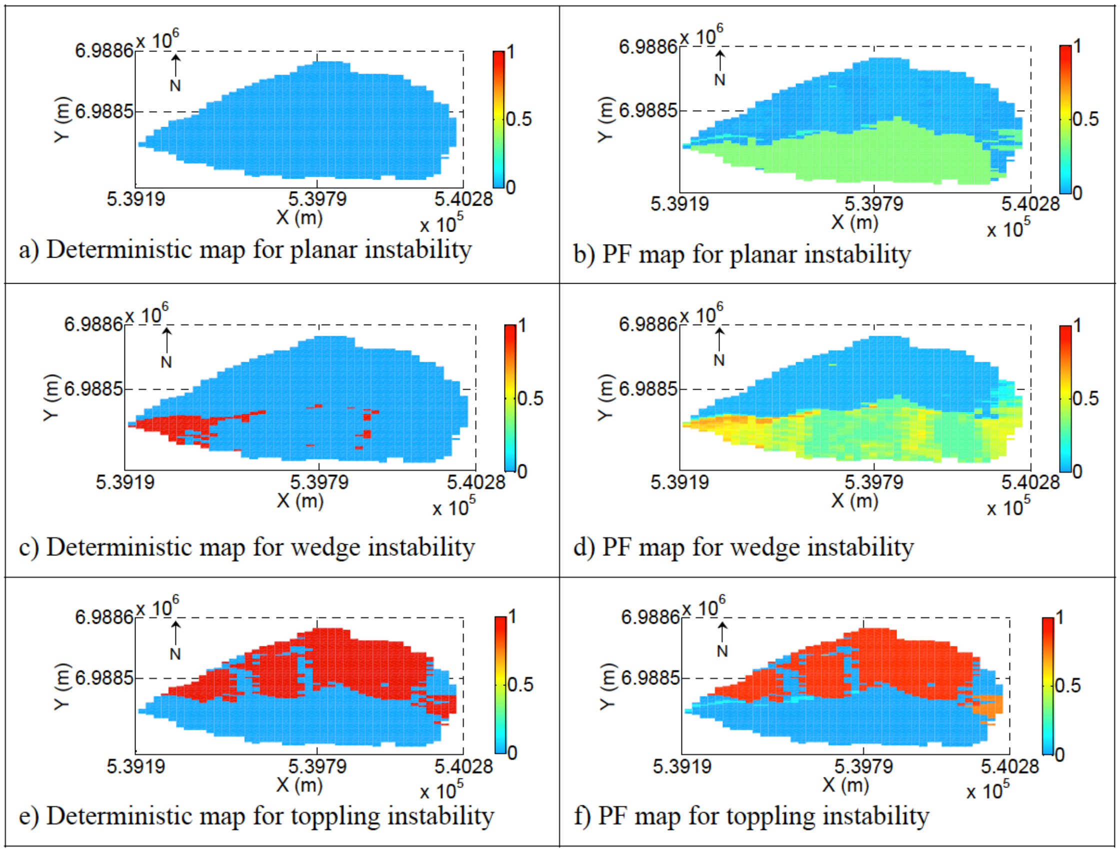
f) FS < 1.3 and/or PF > 10% map for wedge instability

Figure 6. Acceptability criteria at the inter-ramp level.

2.7 Kinematic bench design

The integrated stability analysis methodology was also used to investigate the stability of the proposed bench configurations. The results of the kinematic analysis are summarised in Figure 7. The mean structural values were used for the deterministic analyses. As clearly demonstrated in Figure 7a, when considering only mean values for all inputs, planar instabilities are not an issue, while Figure 7c indicates that only a very limited number of cells in the south west region of the pit are prone to wedge failure. Finally, it can be observed that the north wall is strongly subjected to toppling instabilities (Fig. 7e).

The probabilistic kinematic analysis results are illustrated in Figures 7b, 7d, and 7f. Figure 7b demonstrating that PF for planar instabilities is approaching 40% at bench level on the south side of the pit. This is of particular interest since the mean deterministic analysis did not identify any area of concern with respect to the stability of the slopes. In Figure 7d the map for wedge instabilities shows that the PF is between 40 and 50% on the south wall. These results are similar to the ones observed for planar instabilities. Figure 7f shows the PF for toppling instabilities suggesting that at bench level this is a concern. Numerical modelling can be employed to investigate the potential for localised planar and toppling instabilities to extend to large scale events. The results for kinematic analysis at bench level are shown in Table 4 where the number of potentially unstable cells varies greatly for the various failure modes when carrying deterministic analyses. The probability of failure is within a very limited range (42-50%) of cells for all those modes. This would have not been realised in the absence of a probabilistic analysis.



a) Deterministic map for planar instability
b) PF map for planar instability
c) Deterministic map for wedge instability
d) PF map for wedge instability
e) Deterministic map for toppling instability
f) PF map for toppling instability

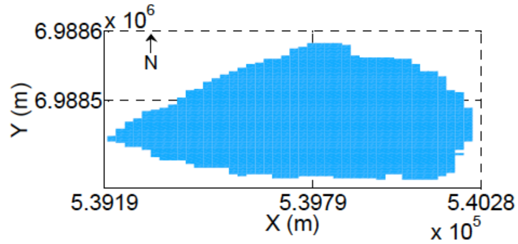
Figure 7. Slope stability at the bench level.

Table 4. Analysis results at the bench level for kinematic instability.

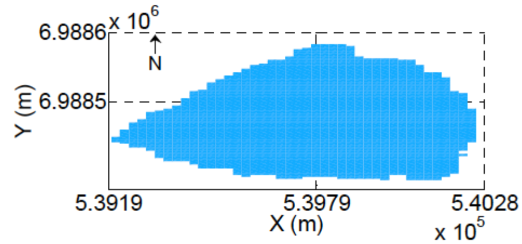
	Total number of cell	**Planar instability**		**Wedge instability**		**Toppling instability**	
		Number of cells	% in overall pit surface	Number of cells	% in overall pit surface	Number of cells	% in overall pit surface
Deterministic analysis	2376	0	0	149	6.27	1047	44.07
PF>10%	2376	1010	42.51	1194	50.25	1074	45.20
PF>20%	2376	1010	42.51	1149	48.36	1047	44.07
PF>25%	2376	988	41.58	1142	48.06	1047	44.07

2.8 Acceptability of proposed bench design

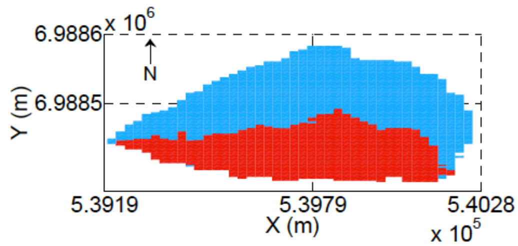
The acceptability of the proposed design has been established using LEA multi-criteria analysis. Referring to Table 3, the criteria were applied to planar and wedge instabilities at bench level. Figures 8a and 8c demonstrated that no cells exceeded the upper limit criteria for either planar and wedge instabilities. The lower bound criteria were not exceeded for wedge type instabilities, Figure 8d. On the other hand, the lower bound criterion was exceeded on the entire south wall for planar instabilities (Fig. 8b). The potential for toppling instabilities, identified by the kinematic analyses could not be addressed by LEA. It does however remain a concern, and it will be addressed by other techniques such as numerical modelling.



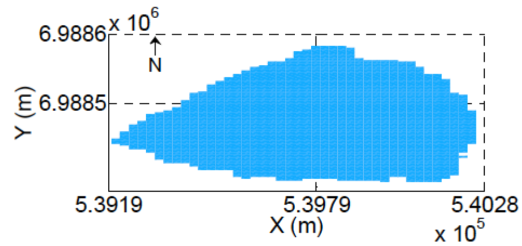
a) FS < 1.1 and/or PF > 50% map for planar instability



c) FS < 1.1 and/or PF > 50% map for wedge instability



b) FS < 1.1 and/or PF > 25% map for planar instability



d) FS < 1.1 and/or PF > 25% map for wedge instability

Figure 8. Acceptability criteria at the bench level.

A closer investigation of the information summarised in Table 5 reveals that 40% of the pit surface (the entire south wall) was not respecting the lower bound slope stability criteria. It can be seen that the limiting factor is not the FS but the PF criterion. This is a clear indication that at the feasibility stage it is imperative to introduce probabilistic analysis as part of the mine design. At bench level, these results suggest that the chosen design is within an acceptable range of values. Close geologic control and cost benefit analysis will be undertaken when the first benches will be excavated to further validate the design.

Table 5. Acceptability criteria analysis results at the bench level for planar instability

Total number of cell	**FS<1.1**		**PF>25%**		**FS<1.1 and/or PF>25%**	
	Number of cells	% in overall pit surface	Number of cells	% in overall pit surface	Number of cells	% in overall pit surface
2376	0	0	945	39.77	945	39.77

3 CONCLUSIONS

The projected ultimate pit of the Tiriganiaq deposit at the feasibility stage was 1050 m by 390 m in size at a depth of 155 m. Within the block model, the pit surface is defined by 2376 blocks of a size of 25 m by 5 m by 5 m. An integrated approach was used to bring together block modelling and slope stability analytical tools to determine the acceptability of the proposed stability design at the bench and inter-ramp slope levels.

The integration of both deterministic and probabilistic analyses allows for the operation to explore the significance of different strategies in developing the pit. The decision process was aided by the choice of multi-criteria stability analysis. This increased flexibility can contribute to improved design and allows the swift exploration of the various options.

4 ACKNOWLEDGEMENTS

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