Mount Polley Mine
Tailing Storage Facility
2011 Stage 7 Expansion Stability Analyses

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1.0 INTRODUCTION

Mount Polley Mine is wholly owned by Imperial Metals Corporation and operated by the Mount Polley Mining Corporation (MPMC), and is located 56 kilometres northeast of Williams Lake. Mount Polley began mine production in 1997 and operated until October 2001 when operations were suspended for economic reasons. In March 2005, the mine restarted production and has been in continuous operation since. Currently, it is estimated that the mill throughput is approximately 20,000 tpd. Tailings are deposited as slurry into the tailings storage facility (TSF). The TSF is comprised of one overall embankment that is approximately 4.2km in length. The embankment, based upon original separate embankments, is subdivided into three (3) sections referred to as the Main Embankment, Perimeter Embankment and South Embankment. Heights vary along the embankment and are approximately 45m, 27m, and 17m respectively, (based upon the Main, Perimeter and South nomenclature). The design and construction monitoring of the TSF embankments to date has been completed under the direction of Knight Piésold Limited (KP). The overall embankment has incorporated a staged expansion design utilizing a modified centreline construction methodology. The latest expansion was completed in August 2010, which entailed a four (4) m embankment raise to a consistent crest elevation of 958 m.

AMEC Earth & Environmental (AMEC) was retained by MPMC to provide design and construction monitoring for future expansions. To facilitate the additional volume of tailings from planned operations, the next expansion (Stage 7) is scheduled for 2011 and entails a 2.5m embankment raise to a crest elevation of 960.5 m.

The objective of the analyses presented herein was to assess the short-term stability of the TSF under static loading conditions. The factor of safety (FoS) required for long-term conditions is 1.5, while for the short-term conditions the FoS required is 1.3; both values consistent with the Canadian Dam Association Guidelines (CDA) (CDA, 2007) which is an industry standard for these facilities and endorsed for use by regulatory authority in British Columbia.

The analyses presented herein considered the stability related only to the 2011 expansion. This analyses is considered consistent with the short-term conditions but, at the same time, we would use the long-term CDA requirements for target values. In order to complete the stability analyses, three as-built sections of the embankments were modeled. The locations of these sections are shown in Appendix A, Figure 1.1.

2.0 ANALYSIS PARAMETERS AND METHODOLOGY

2.1 General

Two-dimensional limit equilibrium stability analyses were carried out using the computer code SLOPE/W (GeoStudio, 2007). The analyses utilized the Morgenstern-Price method of slices solution. There are seven main materials incorporated into the analyzed sections, Zone S (compacted till fill), Zone C (rockfill), tailings, foundation tills (ablation, basal), glaciolacustrine/glaciofluvial sediments, and bedrock. The material properties used for the analyses are based on previously established parameters assumed by KP (2005) with minor modifications deemed appropriate by AMEC. The parameters used in the stability analyses presented herein are summarized in Table 2.1.

The stability of the three dam sections is dependent on the strength of the downstream rockfill shell and foundation materials. The compacted till core is supported by the downstream rockfill shell and does not directly contribute to the stability of the embankment.

2.2 Material Parameters

Compacted Till Fill

Not enough information is currently available to confirm or modify the material parameters, thus the material properties assumed by KP are utilized.

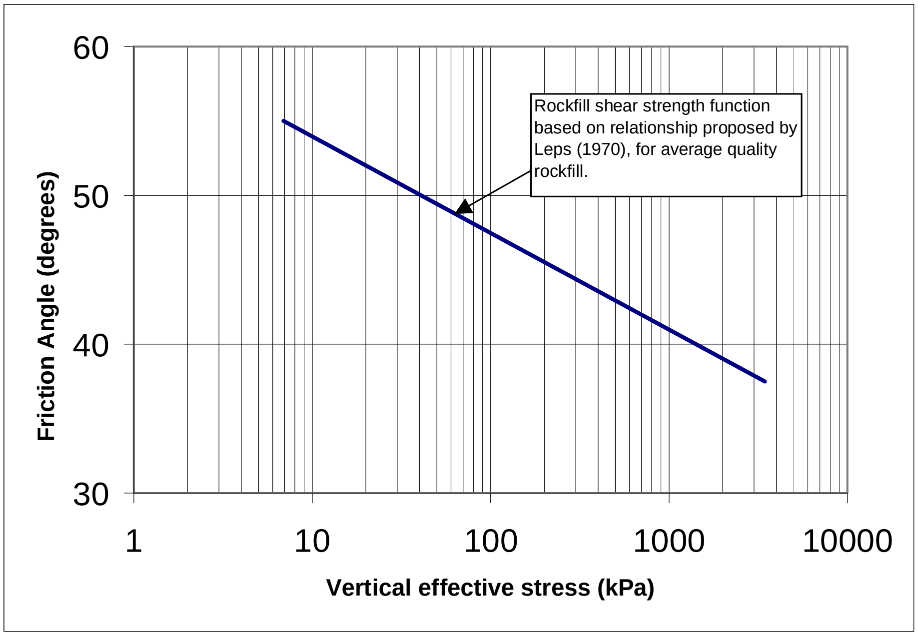
Rockfill

The rockfill shear strength is taken as stress-level dependent as per Leps (1970), as illustrated in Figure 2.1.

It is anticipated that the rockfill used for construction of the 2011 expansion will be comparable to that used for the past dam raises. As such, the trend for average rockfill was used because the rockfill is anticipated to be:

- strong and durable with high compressive strength;
- well-graded, and comprised of highly angular rock; and
- will receive moderate compactive effort.

Figure 2.1 Shear Strength Relationship Used for Rockfill



In-Situ Foundation

KP (2005) summarized the foundation conditions for the dam as follows:

> *"The tailings basin is generally blanketed by naturally occurring well graded low permeability glacial till which functions as an in-situ soil liner and precludes seepage loss from the facility. However, a basin liner was constructed just upstream of the Main Embankment to ensure that the basin liner had a minimum thickness of 2 meters throughout the tailings basin. The constructed basin liner was tied into the Main Embankment core zone and the existing basin liner where the in-situ thickness exceeded 2 m.*
>
> *The foundation conditions at the Main Embankment consist of low permeability glacial till material at surface underlain by fluvial and lacustrine silts up to 20 m thick. The foundation conditions at the Perimeter Embankment consist of low permeability glacial till throughout that is generally in excess of 5 m. The foundation conditions at the South Embankment consist of a relatively thin, low permeability glacial till material overlying bedrock. The glacial till is a few meters thick but its thickness is not consistent throughout the South Embankment foundation. It is important not to expose the fractured bedrock and to ensure that the glacial till cover is at least 2 m thick throughout the foundation and that it is tied into the core zone.*
>
> *Laboratory testwork on the foundation soils indicates that the materials have adequate shear strength to ensure foundation stability of the embankments. Artesian pressures exist at the base of the Main Embankment. Pressure relief wells trenches have been installed at this location to depressurize the underlying glaciofluvial deposits."*

Summary of Material Strength Parameters

The material strength parameters used in the stability analyses are as summarized in Table 2.1.

Table 2.1 Material Strength Parameters

Material	γ_b (Bulk Unit Weight) (kN/m^3)	ϕ' (Friction Angle) (degrees)	c' (Cohesion) (kPa)
Rockfill (Zone C)	22	Defined by Lep's (1970) shear normal function for average quality rockfill (Note 1)	0
Compacted Till Fill (Zone S)	22	35	0
Ablation Till	21	26	0
Glaciolacustrine/Glaciofluvial	20	33 24 (residual)	0
Glacial Till	21	33	0
Tailings	18	30 (drained) S_u/σ_v' = 0.1 (undrained)	0

Note 1. The shear normal function used for the rockfill accounts for the stress-level dependency of the normalized shear strength as expressed by the effective friction angle (ϕ') – see Figure 2.1.

2.3 Pore Pressure Assumptions

Where possible, the current phreatic surfaces were derived from vibrating wire piezometer readings installed in the embankments or into the embankment foundation, as reported in Stage 6B construction report (KP, 2011). Where no piezometric pressure data was available, the phreatic surface was estimated using typical phreatic surfaces observed from similar projects.

The phreatic surface for the 2011 expansion was estimated by increasing current phreatic surface on the upstream side of the core by 2.5 m, equivalent to the Stage 7 raise, while maintaining the phreatic surface downstream of the core.

The rockfill was assigned zero pore pressure except where located below the phreatic surface, below which pore pressures at any given point were taken as hydrostatic.

Artesian conditions are modelled in the main embankment to reflect the pore pressures observed in the glaciolacustrine/glaciofluvial sediment unit in that area. Note that as stated in KP's Stage 6 Construction Report (KP, 2007) piezometric trigger level of 15m above ground reduces the FoS to 1.1. For Stage 7 expansion the same piezometric trigger level is adopted.

2.4 Minimum Factor of Safety Criteria

The minimum FoS criteria for design is 1.3 for short-term (during construction) and 1.5 for long-term (closure) steady state conditions.

3.0 STABILITY ANALYSES RESULTS

The stability analyses of the TSF 2011 expansion were carried out for three sections of the embankment described above. These sections are typical as-built sections as reported in the Stage 6B Construction report (KP, 2011). In addition to the stability analysis of the expansion the current embankment stability was assessed to establish a FoS baseline for comparison. The sections modeled are shown in Figures 3.1 through 3.4 in Appendix A, with a summary provided below in Table 3.1 Factor of Safety Summary.

Table 3.1 Factor of Safety Summary

Section Embankment	Current Conditions	2011 Stage 7 Expansion	Approximate FoS Reduction
Main (Ch. 20+45)	1.8	1.7	3%
Main (Ch.20+45) Glaciolacustrine (Residual)	1.4	1.4	0%
Perimeter (Ch. 39+90)	2.1	2.0	5%
South (Ch. 7+15)	2.6	2.4	10%

The stability analyses identified that the main embankment was the critical stability section for the 2011 expansion. To analyze the 2011 expansion impact on the overall stability of the embankment, a comparison between the current conditions and 2011 expansion was performed. A FoS reduction of about 3% was observed in the main embankment, while a 0% reduction was observed utilizing residual strength in the glaciolacustrine unit and deemed insignificant to the overall stability of the embankment. Similarly, due to the negligible reduction in FoS, the seismic stability situation would remain unchanged relative to KP's 2007 analyses and the deformations would still be considered negligible. Thus, the stability requirement is satisfied for the 2011 expansion.

A more comprehensive embankment stability assessment will be carried out after additional information is gathered during the 2011 expansion and instrumentation installation program.

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