



**Mount Polley Mine  
Tailing Storage Facility  
2012 Stage 8a (965m) Expansion Stability Analyses**



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## 1.0 ANALYSIS PARAMETERS AND METHODOLOGY

### 1.1 General

Two-dimensional limit equilibrium stability analyses were carried for representative sections of the proposed 2012 configuration of the Mt. Polley tailings dam, raised to the 2012 target crest elevation of 965 m. This represents a crest elevation 5 m higher than the 2011 dam configuration.

The analyses were conducted using the computer code SLOPE/W (GeoStudio, 2012), incorporating 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 and glaciofluvial sediments, and bedrock. The material properties used for the analyses are based on previously established parameters assumed by KP (2007) with minor modifications deemed appropriate by AMEC in more recent analyses. The parameters used in the stability analyses presented herein are summarized in Table 1.1.

The stability of the three dam sections selected as representative is dependent on the shear strength of the downstream rockfill shell and foundation materials. The compacted till core is supported by the downstream rockfill shell and does not significantly contribute to the stability of the embankment from a slope stability perspective. The centerline raise geometry of the dam is such that stability is not significantly affected by the shear strength of the upstream impounded tailings.

### 1.2 Material Parameters

In the fall of 2011, AMEC conducted a field investigation, involving sonic drilling, with the objectives of:

- replacement of inoperative instrumentation;
- expansion of the instrumentation network; and
- acquisition of additional geotechnical information around the base of the embankment, with specific focus on the extent and geotechnical characteristics of glaciolacustrine and glaciofluvial sediments within the glacial till units that predominate within the dam foundations.

The following is the summary of the foundation soil stratigraphy below the representative stability analysis sections as presented in the AMEC Site Investigation Report (AMEC, 2012-1):

#### Main embankment section

Glaciolacustrine and glaciofluvial soils exist between an upper and lower till unit, with thicknesses ranging from approximately 5 to 33 m.

#### Perimeter embankment section

Glaciolacustrine and glaciofluvial units exist within the glacial till units. At Stn.4+000 the thicknesses are approximately 3 to 4 m, while at Stn.3+300 the thickness of the unit is

approximately 4 m. Glacial till was the only soil unit encountered in the drill hole at Stn.4+500.

South embankment section

Only a thin unit of glaciolacustrine soil, in the order of 0.6 m, was encountered within foundation soils near Stn.1+100.

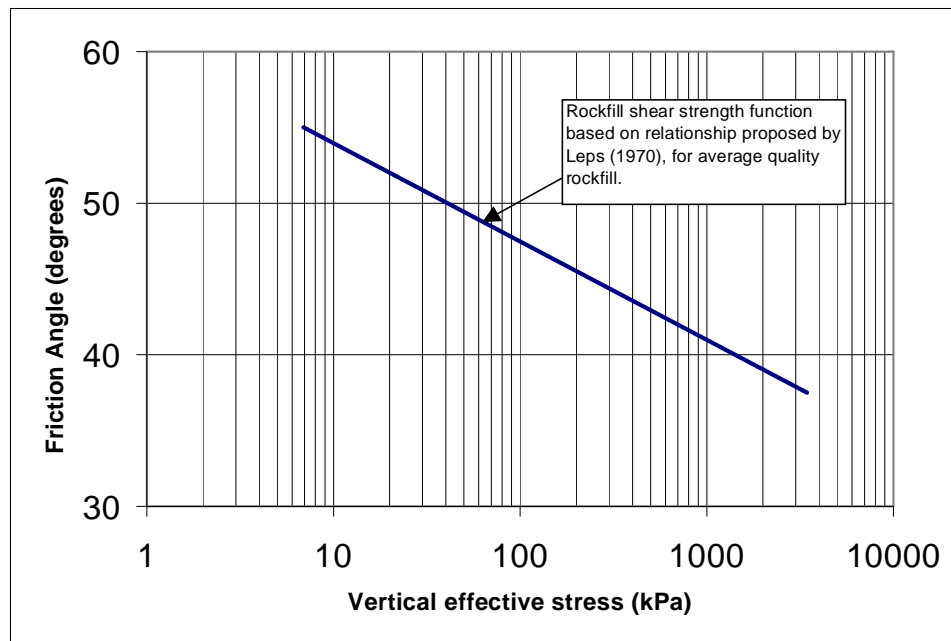
The glaciolacustrine/glaciofluvial unit generally was found to be varved with predominantly silt and clayey silt of low plasticity, interbedded with more granular glaciofluvial deposits. Evidence of pre-shearing within the glaciolacustrine unit, checked for by peeling the sonic borehole cores apart along varves for close visual examination, specifically looking for slickensided surfaces, was not encountered. Thus for this unit, a shear strength of  $c' = 0$ , and  $\phi' = 28^\circ$  is judged reasonable, although sensitivity analyses were carried out within the range given in Table 1.1.

The foundation till unit comprises silty sand and gravel with occasional interbedded sand seams at depth. This unit is of higher shear strength than the glaciolacustrine/glaciofluvial unit.

The rockfill shear strength is taken as stress-level dependent as per Leps (1970), as illustrated in Figure 1.1. It is anticipated that the rockfill used for construction of the 2012 expansion will be comparable to that used for the past dam raises. As such, the trend for average quality rockfill was used because the rockfill:

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

**Figure 1.1 Shear Strength Relationship Used for Rockfill**



During the 2011 construction season, AMEC observed, on the basis of field density test results, the bulk unit weight of the till averages about 20.5 kN/m<sup>3</sup>, so this is now adopted for the purposes of stability analyses.

The material strength parameters used in the stability analyses are as summarized in Table 1.1.

**Table 1.1 Material Strength Parameters**

Material	$\gamma_b$ (Bulk Unit Weight) (kN/m <sup>3</sup> )	$\phi'$ (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)	20.5	35	0
Glaciolacustrine/Glaciofluvial	20	28 Sensitivity analysis (24 through 33)	0
Basal Till	21	33	0
Tailings	18	30 (drained) $S_u/\sigma_v' = 0.1$ (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 ( $\phi'$ ) – see Figure 1.1.

### 1.3 Pore Pressure Assumptions

Where possible, the current phreatic surfaces used for the stability analysis sections were derived from vibrating wire piezometer readings installed in the embankments or into the embankment foundation. Where no piezometric pressure data was available, the phreatic surface was estimated based on trends on monitored sections, interpolation of piezometer data, observed piezometric trends over the years at this facility, and experience from other tailings dams of similar design with similar foundation conditions.

The phreatic surface for the 2012 (crest El. 965 m) raise was estimated by increasing the phreatic surface on the upstream side to an elevation of 965 m. equivalent to the maximum Stage 8a raise, while maintaining the phreatic surface downstream of the core as indicated by interpolation of piezometric data, which shows essentially zero foundation piezometer response, neither to the rising tailings pond elevation, nor in response to increased embankment loading associated with the construction of the annual stage raises.

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.

The phreatic surface modeled in the analyses reflects the pore pressures observed in the glaciolacustrine/glaciofluvial unit.

### 1.4 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. Currently, “during construction” conditions are applicable.

## 2.0 STABILITY ANALYSES RESULTS

The stability analyses of the 2012 Stage 8a expansion were carried out for three representative cross sections of the embankment (Perimeter, Main, and South). These are the same sections analyzed in previous reports. The stability results are presented in Figure 2.1 through Figure 2.3 and are summarized below in Table 2.1.

To analyze stability of the embankment two shear strength cases were considered for each cross section: one considering drained shear strength within the tailings, and the other considering residual undrained shear strength (i.e. post-liquefaction conditions) within the tailings.

**Table 2.1 Factor of Safety Summary**

Section Embankment	Original 2012 Stage 8 (963.5 m)	2012 Stage 8a (965 m)	Approximate FoS Reduction
Tailings shear strength: drained ( $c' = 0$ , $\phi' = 30^\circ$ ), minimum acceptable FoS = 1.3			
Main (Ch. 20+60)	1.33	1.31	1.5%
Perimeter (Ch. 39+90)	1.85	1.81	2.2%
South (Ch. 7+20)	2.03	1.95	3.9%
Tailings shear strength: post-liquefaction, undrained ( $S_u/\sigma_v' = 0.1$ ), minimum acceptable FoS = 1.1			
Main (Ch. 20+60)	1.29	1.27	1.6%
Perimeter (Ch. 39+90)	1.82	1.77	2.7%
South (Ch. 7+20)	2.00	1.92	4.0%

Sensitivity analyses were undertaken for the main embankment (the one with the lowest factors of safety) considering a range of shear strengths within the glaciolacustrine/glaciofluvial unit, for peak (drained) and post-liquefaction residual (undrained) shear strength conditions within the tailings. The results of these analyses are summarized on Figure 2.4. For the 2012 stage 8 raise configuration, an acceptable factor of safety ( $\geq 1.3$ ) is obtained for a glaciolacustrine/glaciofluvial unit  $\phi'$  value of  $28^\circ$ .

To analyze the 2012 Stage 8a embankment expansion impact on the overall stability of the embankment, analyses comparing the originally proposed 2012 Stage 8 raise (to crest El. 963.5 m) stability analyses to the currently proposed 2012 Stage 8a (to crest El. 965 m) were performed. The critical section (i.e. yielding the lowest factor of safety) for the 2012 Stage 8a expansion remains the main embankment. A FoS reduction was observed in the main embankment for the case of peak (drained) strength within the tailings, while reduction of about 1.6% was observed for the post-liquefaction residual (undrained) strength within the tailings. Similarly, due to the negligible reduction in FoS under static loading conditions, it is reasonable to infer that the seismic stability situation would remain essentially unchanged relative to KP's 2007 analyses, which predicted earthquake-induced deformations, under the design earthquake loading, to be well within tolerable limits. Thus, stability requirements are satisfied for the 2012 Stage 8a expansion.

A stability analyses for the ultimate embankment configuration is currently underway with a design change from modified centerline raising to centerline raising, beginning at El. 963.5 m. These analyses will address potential raising of the embankment to crest El. 990 m. In addition, during the ultimate design stability analysis the timing of flattening/extending of the overall downstream slope is being assessed to maintain a FoS during construction above 1.3 and ultimately achieve the minimum closure requirement of 1.5, under static loading conditions, once the embankment is completed to its final configuration.

## 2.1 Pore Pressure Trigger Levels

Pore pressure trigger levels are a useful means of relating monitored piezometer data to the stability analyses and the achieved factors of safety. In this way, piezometric alert levels can be quantified, with pre-set actions to be taken if defined trigger levels are approached or exceeded.

To determine the pore pressure trigger levels in the foundation piezometers additional stability analyses were performed. As the main embankment cross section was determined to be the critical section, as stated above, this cross section and the pore pressures associated with this section were utilized to assess and assign trigger levels. A red, yellow, green “stoplight” approach was utilized and the threshold conditions are defined as follows:

- Red (factor of safety at or below 1.1) – If the foundation piezometers indicate a red condition, crest raising is to cease. AMEC’s Senior Technical Engineer is to be informed immediately, and a corrective course of action will be implemented as per direction of the AMEC’s Senior Technical Engineer, including intensified monitoring, and placement of a stabilization buttress to flatten the overall slope in the embankment area of concern.
- Yellow (factor of safety above 1.1 and below 1.3) – If the foundation piezometers indicate a yellow condition, work should be temporarily suspended in around the embankment, AMEC’s Senior Technical Engineer is to be informed, and a corrective action will be implemented as per direction of the AMEC’s Senior Technical Engineer. Access to the embankment should be limited to essential personnel.
- Green (factor of safety above 1.3) – If the foundation piezometers indicate a green condition, work in and around the embankment is to continue as needed.

It should be noted that a yellow or red condition is not automatically triggered by a single piezometer on a given instrumentation section yielding a reading of concern. Such conditions will only be triggered if most or all foundation piezometers on a given section reach the requisite trigger levels. If individual piezometers on a section approach or reach threshold levels while the remainder do not, additional and/or intensified monitoring may be specified, but the threshold levels described above will not be deemed as having been triggered.

Besides the specified trigger levels, piezometric trends are to be closely monitored in the foundation piezometers. Small variations in the piezometric readings are expected, however if a spike occurs in any of the foundation piezometers, and/or an unexpected a consistent trend of increasing pore pressure is noted, AMEC’s Support Engineer is to be informed immediately to assess the situation.

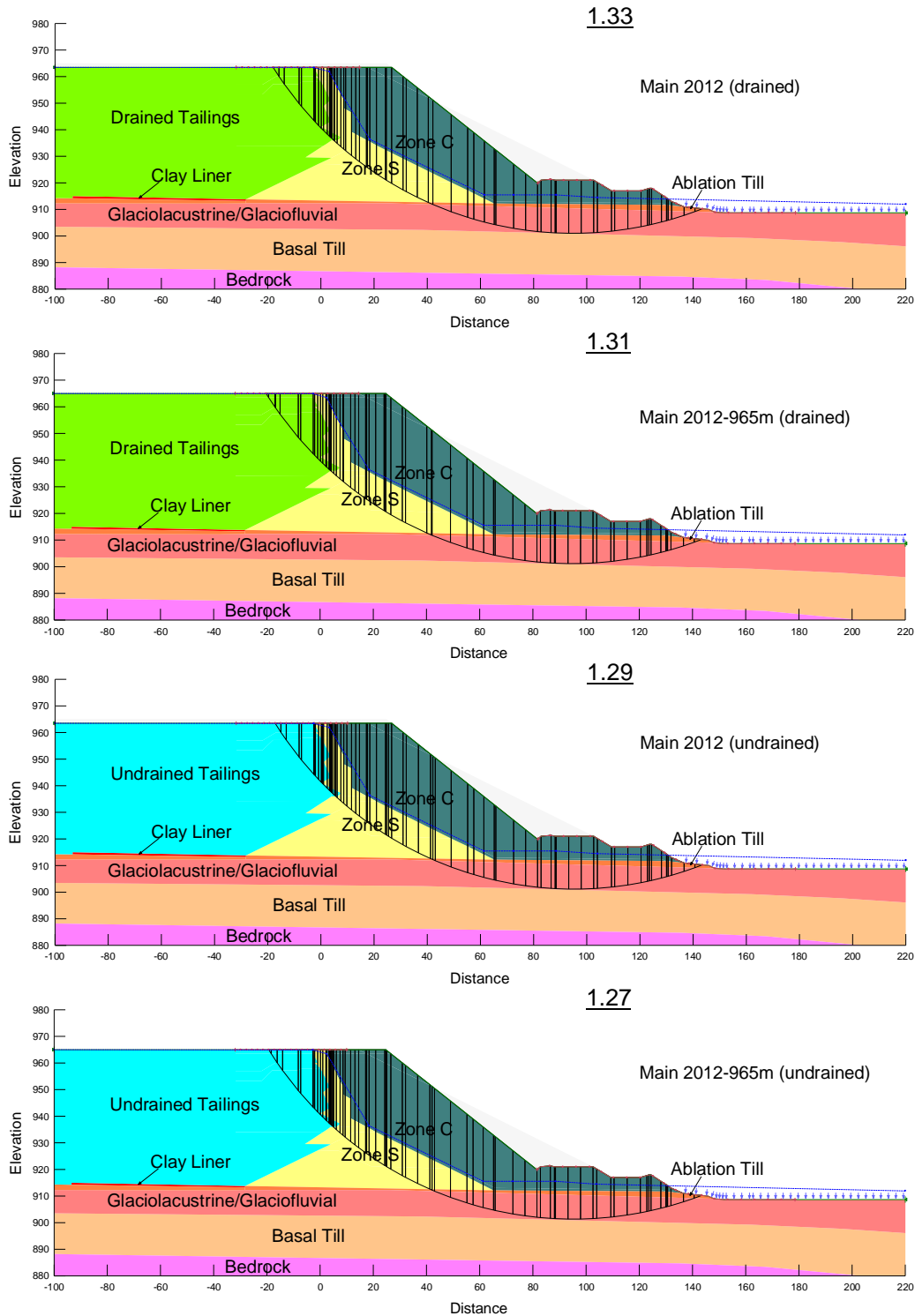
The results of the pore pressure trigger level stability analyses are presented in Figure 2.5 and are summarized in the Table 2.2 below, which applies only for the main embankment piezometers. Factor of safety values for the perimeter and south embankments are sufficiently high that monitoring of piezometric trends, without defined trigger levels, is deemed sufficient.

**Table 2.2 Foundation Piezometer Trigger Levels**

Condition	Modeled Pore Pressure Elevation Head (m)	Above Original Ground Elevation (912m) (m)
RED	Above 925	>13
YELLOW	Between 921 and 925	9 to 13
GREEN	Less than 921	<9

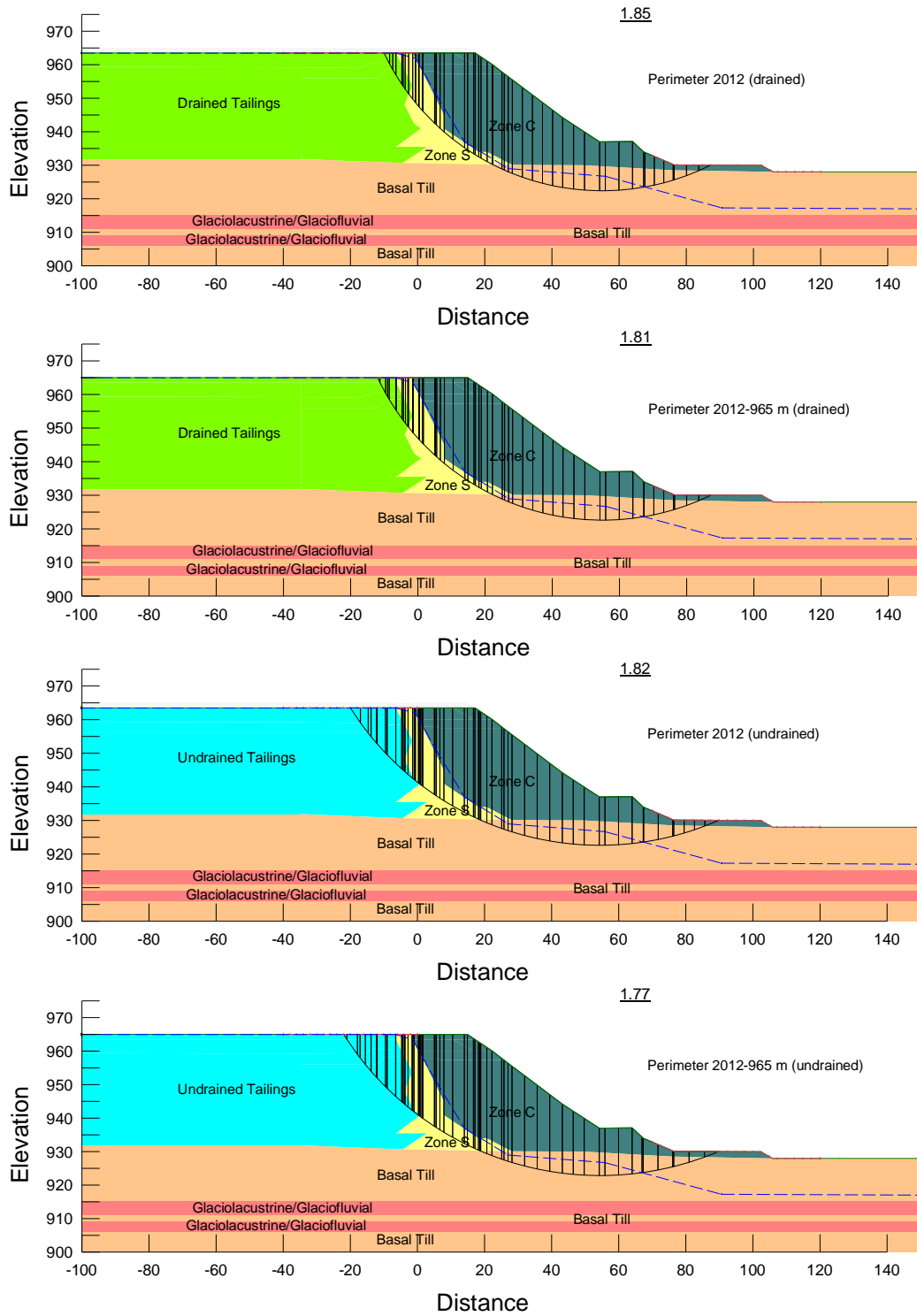


**Figure 2.1 Main Embankment Stability Analysis**



Please note that phreatic surface indicated is applied for the tailings, the till core, and the foundation soils only. Rockfill shell is assumed fully drained

**Figure 2.2 Perimeter Embankment Stability Analysis**

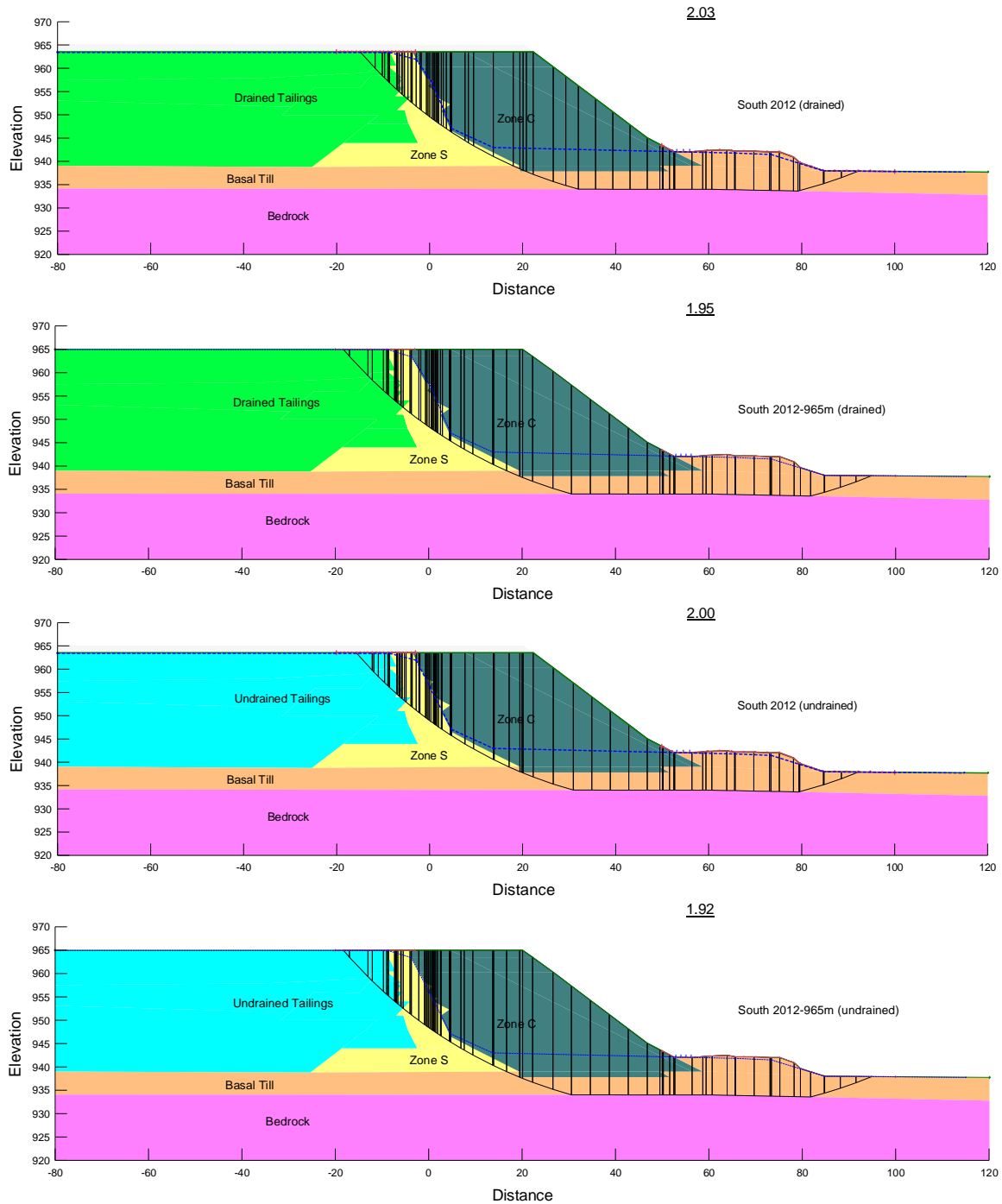


Please note that phreatic surface indicated is applied for the tailings, the till core, and the foundation soils only. Rockfill shell is assumed fully drained

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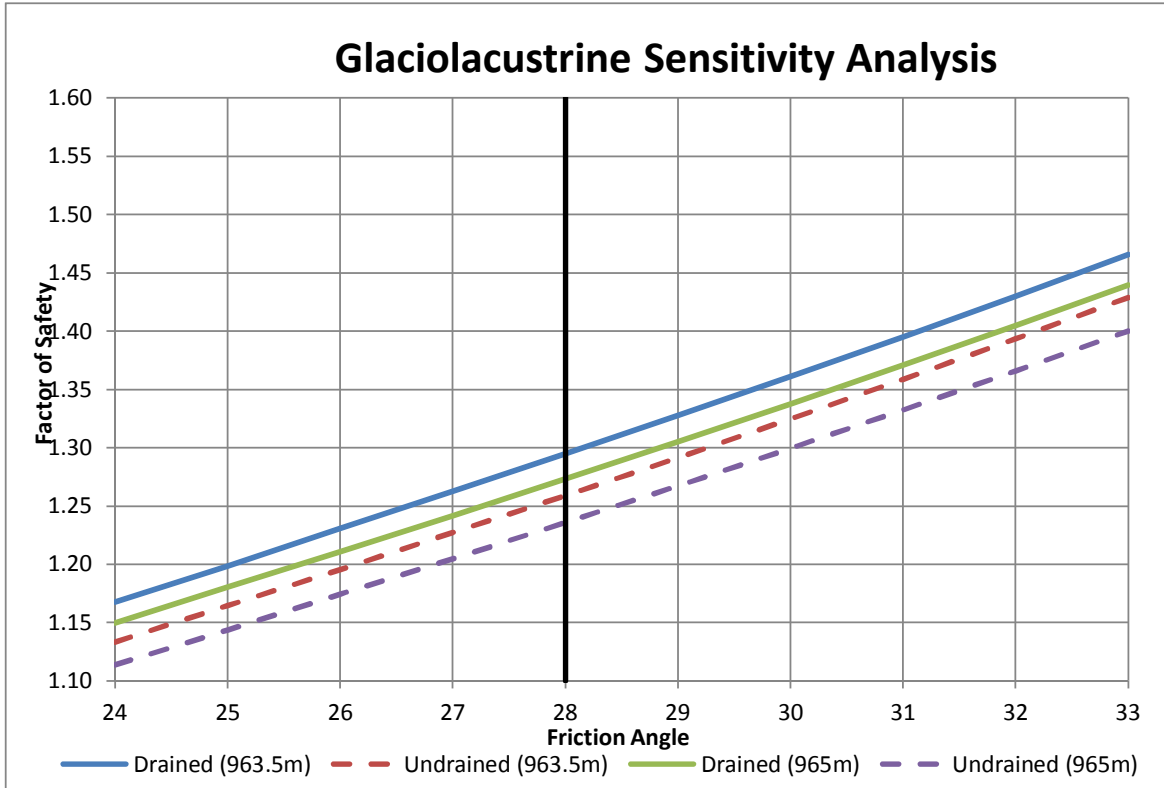
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**Figure 2.3 South Embankment Stability Analysis**

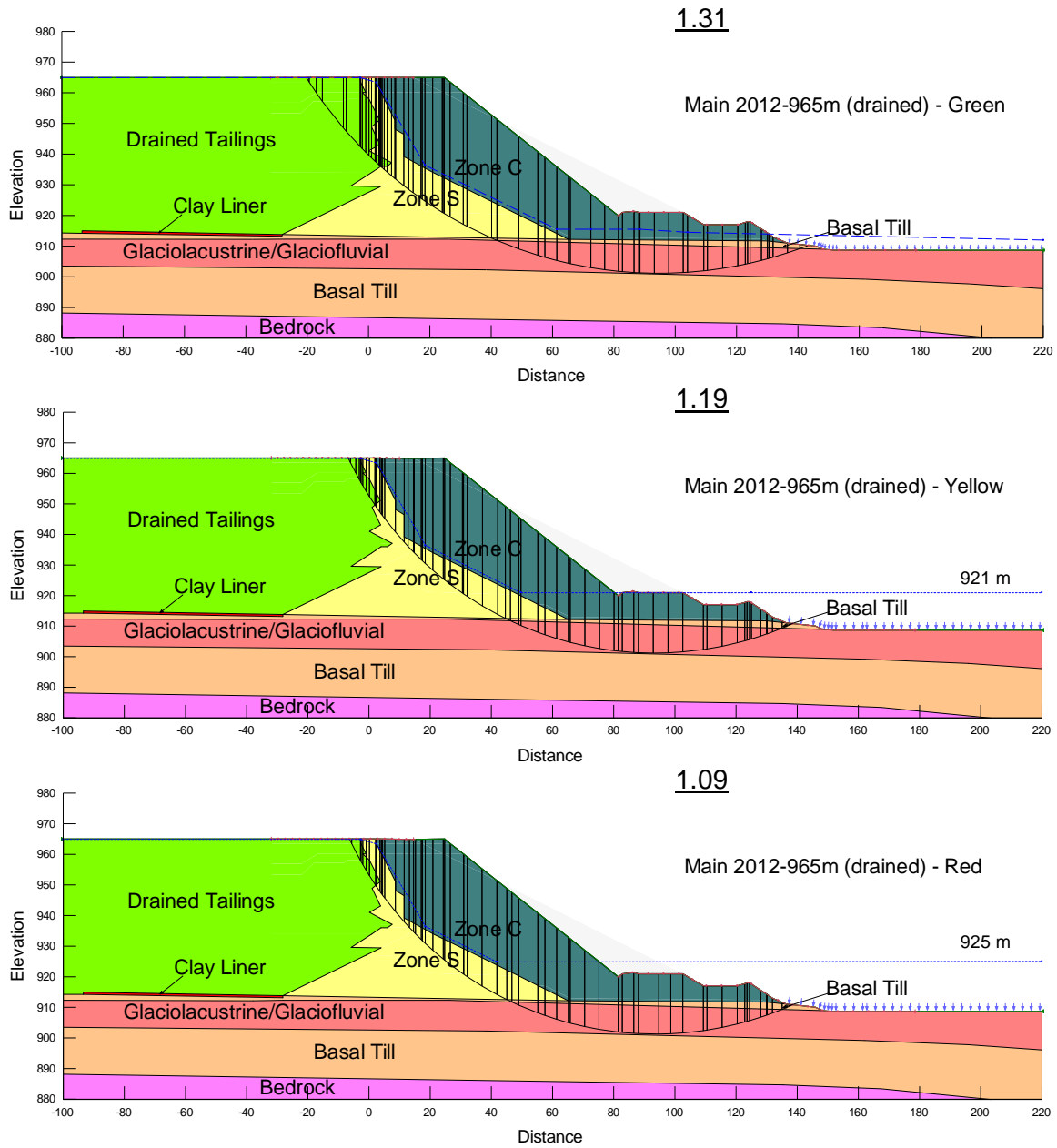


Please note that phreatic surface indicated is applied for the tailings, the till core, and the foundation soils only. Rockfill shell is assumed fully drained

**Figure 2.4 Sensitivity Analysis of Glaciolacustrine friction angle (24 through 33)  
Main Embankment**



**Figure 2.5 Pore Pressure Trigger Levels Stability Analysis**



Please note that phreatic surface indicated is applied for the tailings, the till core, and the foundation soils only. Rockfill shell is assumed fully drained

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