



**Independent Expert Engineering Investigation and Review Panel**

# **Report on Mount Polley Tailings Storage Facility Breach**

**January 30, 2015**



# Report on Mount Polley Tailings Storage Facility Breach

## Independent Expert Engineering Investigation and Review Panel

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# EXECUTIVE SUMMARY

## THE EMBANKMENTS

Three contiguous embankments confine the Mount Polley tailings storage facility (TSF). Of these, the Perimeter Embankment, where the breach occurred, was the northern flank of the TSF.

The embankments are composed of a core with the function of acting as an impervious element. Downstream of the core, a filter zone restrains material in the core from outward migration. The core and filter are then supported by a rockfill zone. In the upstream direction, the core is supported by an upstream fill zone composed of rockfill and/or tailings.

## THE BREACH

The breach occurred within the Perimeter Embankment. At the time of the breach, the TSF was permitted under the Ministry of Energy and Mines, Permit M-200, with approval to raise the crest by 2.5 metres. The breach occurred early on August 4, 2014 at a crest elevation 1 metre short of its permitted elevation. Loss of containment was sudden, with no warning. The recorded pond elevation at 6:30 pm on August 3, 2014 was 2.3 metres below the crest.

## THE MANDATE

Following the breach of the tailings storage facility at the Mount Polley Mine, the Government of British Columbia, through the Ministry of Energy and Mines, together with the Williams Lake Indian Band and the Soda Creek Indian Band, established an independent expert investigation and review panel (the Panel) to investigate and report on that breach. The Panel was required to submit a final report to the Ministry of Energy and Mines and the Williams Lake Indian Band and the Soda Creek Indian Band on or before January 31, 2015.

The purpose of the investigation has been as follows:

- To investigate and report on the cause of the failure of the tailings storage facility that occurred on August 4, 2014 at the Mount Polley Mine (the Mine) in B.C.
- In addition, the Panel may make recommendations to government on actions that could be taken to ensure that a similar failure does not occur at other mine sites in B.C.
- The Panel is authorized, as part of its investigations and report, to comment on what actions could have been taken to prevent this failure and to identify practices or successes in other jurisdictions that could be considered for implementation in B.C.

# EXECUTIVE SUMMARY

Further, it was expected that the Panel would:

- Identify any mechanism(s) of failure of the tailings storage facility.
- Identify any technical, management or other practices that may have enabled or contributed to the mechanism(s) of failure. This may include an independent review of the design, construction, operation, maintenance, surveillance and regulation of the tailings storage facility.
- Identify any changes that could be considered to reduce the potential for future such occurrences.

## PANEL ACTIVITIES

The Panel began its inquiry with multiple hypotheses for failure:

- Human intervention
- Overtopping
- Piping and cracking
- Foundation failure

The Panel found no evidence of failure due to either human intervention or failure due to overtopping, notwithstanding the fact that an episode of overtopping over portions of the Perimeter Embankment occurred in May 2014. The question of piping and cracking, which is a common cause of failure of earth dams, received corresponding attention. Although factors of concern were identified by the Panel, it did not find evidence that piping and/or cracking caused the breach.

This reduced the focus of the Panel to failure in the foundation of the embankment. Visual evidence of bodily outward displacement and rotation of the embankment remnants were consistent with foundation failure. A foundation can be weak and fail in a number of ways. One is the presence of a weak layer that had been undetected during design. Another is the presence of a brittle stratum that loses strength as it comes under load and becomes too weak to support the load applied by the embankment and TSF contents, so that failure ensues. Yet another possibility is the presence of a layer that is compressible under the applied load and, when stressed, develops high pore pressure that results in weakening of an otherwise much stronger material. This is termed undrained failure. It was the object of the site studies undertaken by the Panel to determine which of these foundation failure mechanisms prevailed.

# EXECUTIVE SUMMARY

The Panel undertook comprehensive Surface Investigations that provided detailed, observable information on the sliding mechanism that had occurred. A challenging and complex Subsurface Investigation was also undertaken, partly in collaboration with the site investigation program initiated by the both the Mines Inspector and Mount Polley Mining Corporation (MPMC), and in addition, by the Panel alone.

The Subsurface Investigation was particularly valuable in defining the controlling stratigraphy in the breach area and identifying that the failure occurred in a glaciolacustrine layer, called Upper GLU. No indication of pre-shearing or the presence of markedly strain-weakening materials was detected, leaving undrained failure in the Upper GLU as the only viable hypothesis. The type and extent of pre-failure site investigations were not sufficient to detect this stratum or to identify its critical nature. The Panel's Subsurface Investigation was structured to obtain undisrupted samples of the Upper GLU and subsequently determine its properties.

The Upper GLU was found to be preconsolidated prior to embankment construction, but became normally consolidated under the loads applied by construction of the Perimeter Embankment. That is, it had experienced prior consolidation and strengthening under loads in its geological past, but not under the loads associated with the Perimeter Embankment, which created the normally consolidated state. Under these conditions, the Upper GLU was compressible and susceptible to undrained failure. This condition had not been recognized in the design of the TSF.

Laboratory tests were performed to determine the undrained strength of the Upper GLU and these parameters were utilized in computer analyses to calculate whether failure should have occurred under the applied load. The results were confirmatory.

# EXECUTIVE SUMMARY

## CONCLUSIONS

The Panel concluded that the dominant contribution to the failure resides in the design. The design did not take into account the complexity of the sub-glacial and pre-glacial geological environment associated with the Perimeter Embankment foundation. As a result, foundation investigations and associated site characterization failed to identify a continuous GLU layer in the vicinity of the breach and to recognize that it was susceptible to undrained failure when subject to the stresses associated with the embankment.

The specifics of the failure were triggered by the construction of the downstream rockfill zone at a steep slope of 1.3 horizontal to 1.0 vertical. Had the downstream slope in recent years been flattened to 2.0 horizontal to 1.0 vertical, as proposed in the original design, failure would have been avoided. The slope was on the way to being flattened to meet its ultimate design criteria at the time of the incident.

## REGULATORY OVERSIGHT

The Panel reviewed the roles and responsibilities of the B.C. Ministry of Energy and Mines (the Regulator) and its interactions related to the MPMC TSF. The Panel found that inspections of the TSF would not have prevented failure and that the regulatory staff are well qualified to perform their responsibilities. The Panel found that the performance of the Regulator was as expected.

## THE FUTURE

The Panel has examined the historical risk profile of the current portfolio of tailings dams in B.C. and concluded that the future requires not only an improved adoption of best applicable practices (BAP), but also a migration to best available technology (BAT). Examples of BAT are filtered, unsaturated, compacted tailings and reduction in the use of water covers in a closure setting. Examples of BAP bear on improvements in corporate design responsibilities, and adoption of Independent Tailings Review Boards. Specific recommendations are made in the body of the report.



# 1 | Introduction

Following the breach of the tailings storage facility at the Mount Polley Mine on August 4, 2014, the Government of British Columbia, through the Minister of Energy and Mines, together with the Williams Lake Indian Band and the Soda Creek Indian Band, established an independent expert engineering investigation and review panel (the Panel) to investigate and report on that breach.

## 1.1 PURPOSE OF THE PANEL

The purpose of the Panel is as follows:

- To investigate into and report on the cause of the failure of the tailings storage facility (TSF) that occurred on August 4, 2014 at the Mount Polley Mine (the Mine) in B.C.
- In addition, the Panel may make recommendations to government on actions that could be taken to ensure that a similar failure does not occur at other mine sites in B.C.
- The Panel is authorized, as part of its investigation and report, to comment on what actions could have been taken to prevent this failure and to identify practices or successes in other jurisdictions that could be considered for implementation in B.C.

Under its Terms of Reference, it is expected that the Panel will:

- Identify any mechanism(s) of failure of the TSF.
- Identify any technical, management or other practices that may have enabled or contributed to the mechanism(s) of failure. This may include an independent review of the design, construction, operation, maintenance, surveillance and regulation of the TSF.
- Identify any changes that could be considered to reduce the potential for future such occurrences.

## 1.2 PANEL MEMBERS

The members of the Panel are:

- Dr. Norbert R. Morgenstern (Chair), CM, AOE, FRSC, FCAE, Ph.D., P.Eng.
- Mr. Steven G. Vick, M.Sc., P.E.
- Dr. Dirk Van Zyl, Ph.D., P.E., P.Eng.

The detailed Terms of Reference are included as Appendix A.

# 2 | What Did the Panel Do?

## 2.1 PANEL ACTIVITIES

In furtherance of its mandate, the Panel undertook the following:

- It retained Thurber Engineering Ltd. (Thurber) to conduct field investigations, data compilation, laboratory testing, and analyses. All of this work proceeded under the direction of the Panel.
- It assembled and inspected related documents in the files of the Mine, its consultants who have acted as the Engineer of Record (EOR), and the Ministry of Energy and Mines (MEM).
- It solicited and collected relevant information from the public at large.
- It conducted a number of personal interviews to clarify information recorded in documents.
- It convened regular formal meetings with recorded minutes.
- It interpreted all of the above to arrive at conclusions and recommendations.

## 2.2 SUPPORTING INFORMATION

As directed by the Terms of Reference, the Panel has provided this final report, along with the appendices, to the Minister of Energy and Mines, the Williams Lake Indian Band and the Soda Creek Indian Band. The background reports and information used by the Panel for the preparation of this report were also made available to these parties through an online data room. The Panel considers the supporting information and substantiating documentation to be an integral part of its report that is necessary for a proper understanding of its findings.

The B.C. Ministry of Energy and Mines and Ministry of Environment, Conservation Officer Services, have directed the Panel to withhold some of the documents, and redact portions of other documents, so that the Panel's inquiry does not compromise any other investigations and to ensure it is in compliance with the privacy protection provisions of *Freedom of Information and Protection of Privacy Act* (FIPPA) (see Appendix A). The redaction of personal information was completed by Shared Services BC. As a result, these documents, which may have been cited in this report are not available at this time.

The background information was provided to the Panel by many different sources. Appendix B contains further details on background reports and information, and how these were organized and provided to the Minister of Energy and Mines, the Williams Lake Indian Band and the Soda Creek Indian Band.

Within the text of the report and appendices, specific documents are referenced by an endnote, which contains the document number as it relates to where it can be found in the data room. See Appendix B for more details.

Additional technical references are also cited by endnote directly within the body of this report. Endnotes can be found at end of each section of the report. The collected technical references can be found at the end of the report.

The observations of the Panel are supported by referenced documents where possible. The findings of the Panel are outlined in the sections of the report that follow. Conclusions and recommendations are presented in detail at the end of the report.

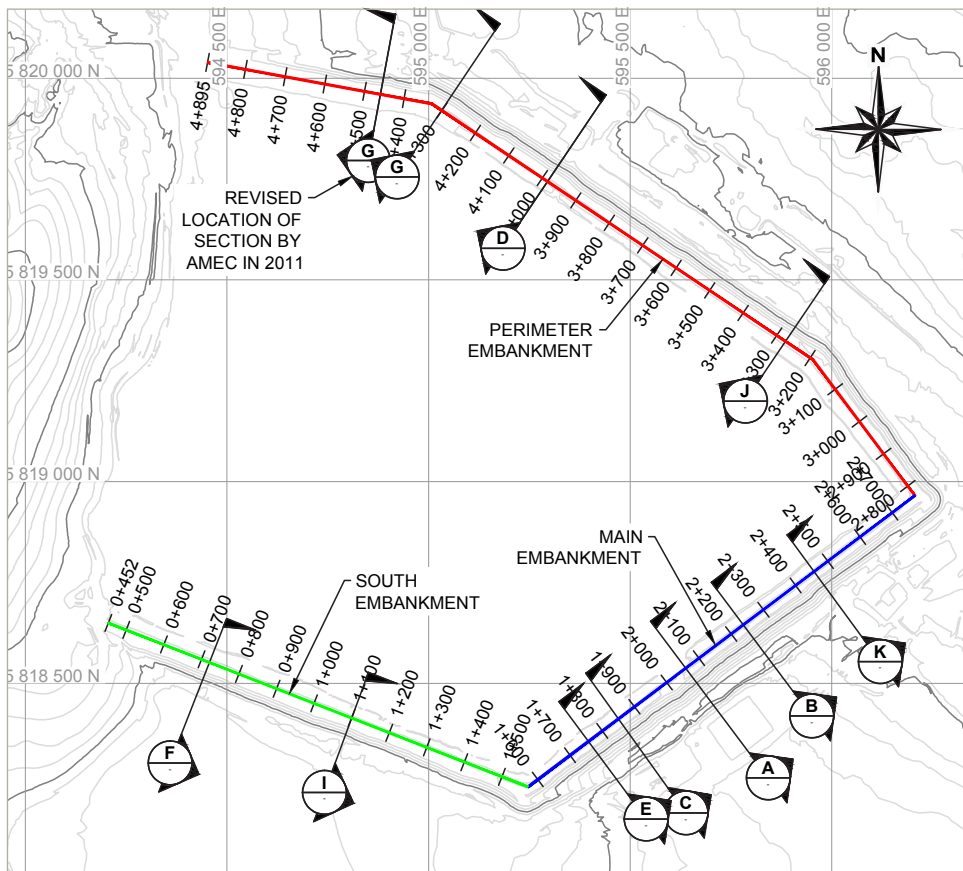
# 3 | Tailings Storage Facility (TSF)

## 3.1 DESCRIPTION OF TSF

Mount Polley Mining Corporation (MPMC) operates the Mine, and the British Columbia Ministry of Energy and Mines (MEM) is the Regulator. From the first approved and constructed portion of the TSF, in 1995, to early 2011, Knight Piésold (KP) was the Engineer of Record (EOR). Subsequently, AMEC assumed the responsibility as EOR and had that role at the time of the breach. BGC were to assume that responsibility after the 2014 construction season.

**Figure 3.1.1** is a plan of the TSF adapted from the last As-Built Construction Report.<sup>1</sup> It indicates that the TSF was composed of three embankments: the Main Embankment, the Perimeter Embankment, and the South Embankment. The TSF is closed to the west by rising natural ground. The figure also indicates the location of instrumented control sections utilized by the succession of EORs. The breach occurred in the Perimeter Embankment near Section G.

**FIGURE 3.1.1: TAILINGS STORAGE FACILITY PLAN**

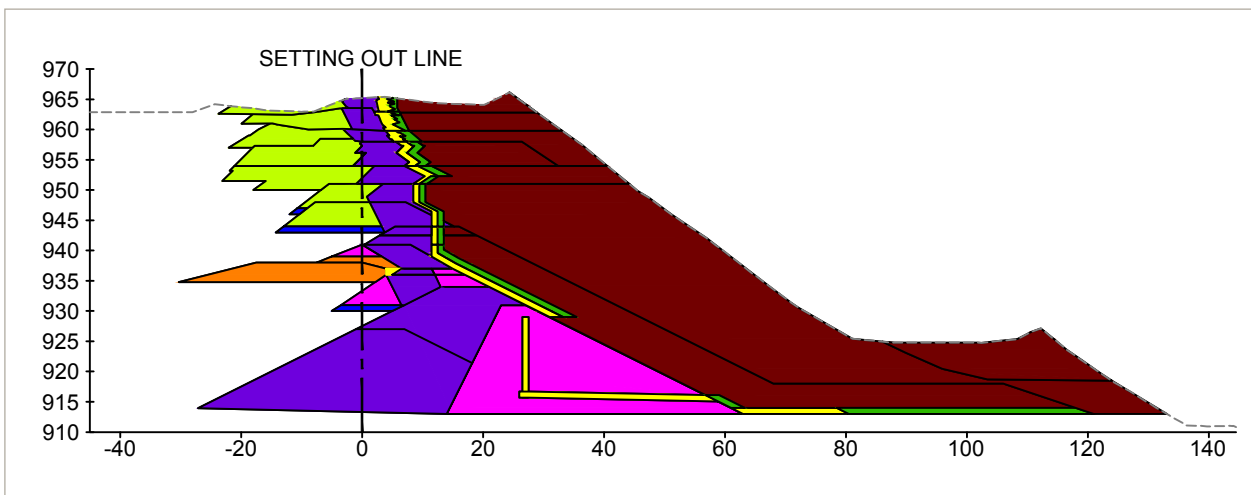


# 3 | Tailings Storage Facility (TSF)

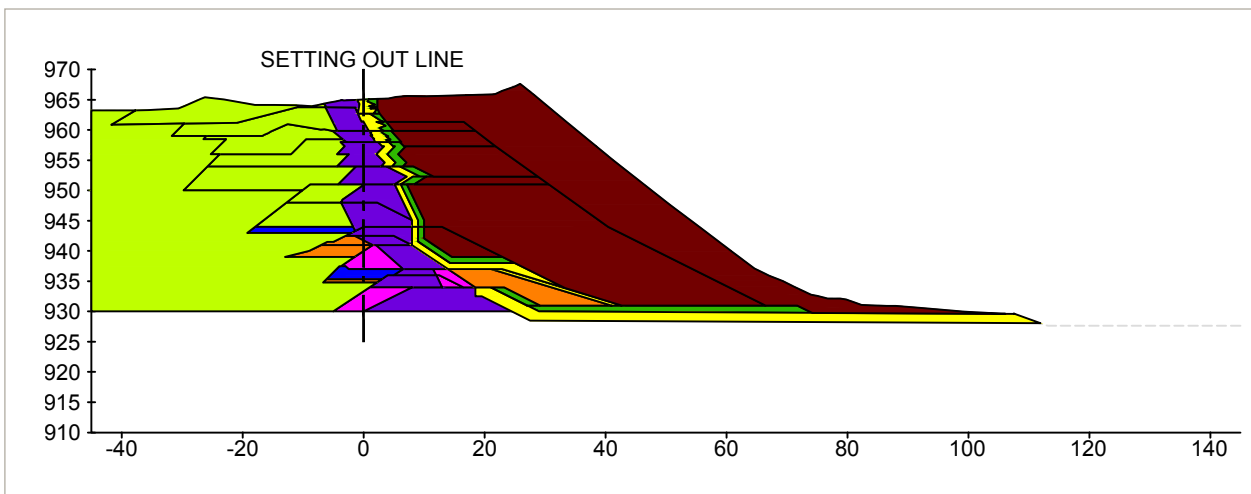
## 3.2 CONSTRUCTION OF TSF AND POND ELEVATION

The Main Embankment and the Perimeter Embankment were the first to go into construction, with the Starter Dam completed in 1996. The South Embankment followed in later years. **Figures 3.2.1** and **3.2.2** present the sections of the Main and Perimeter Embankments at the end of the 2013 construction season. **Figure 3.2.1** is for the Main Embankment at Section A, approximately the highest section. **Figure 3.2.2** is for the Perimeter Embankment and represents the closest instrumented section (Section D) to the breach zone at the time of failure.

**FIGURE 3.2.1: MAIN EMBANKMENT AT SECTION A**



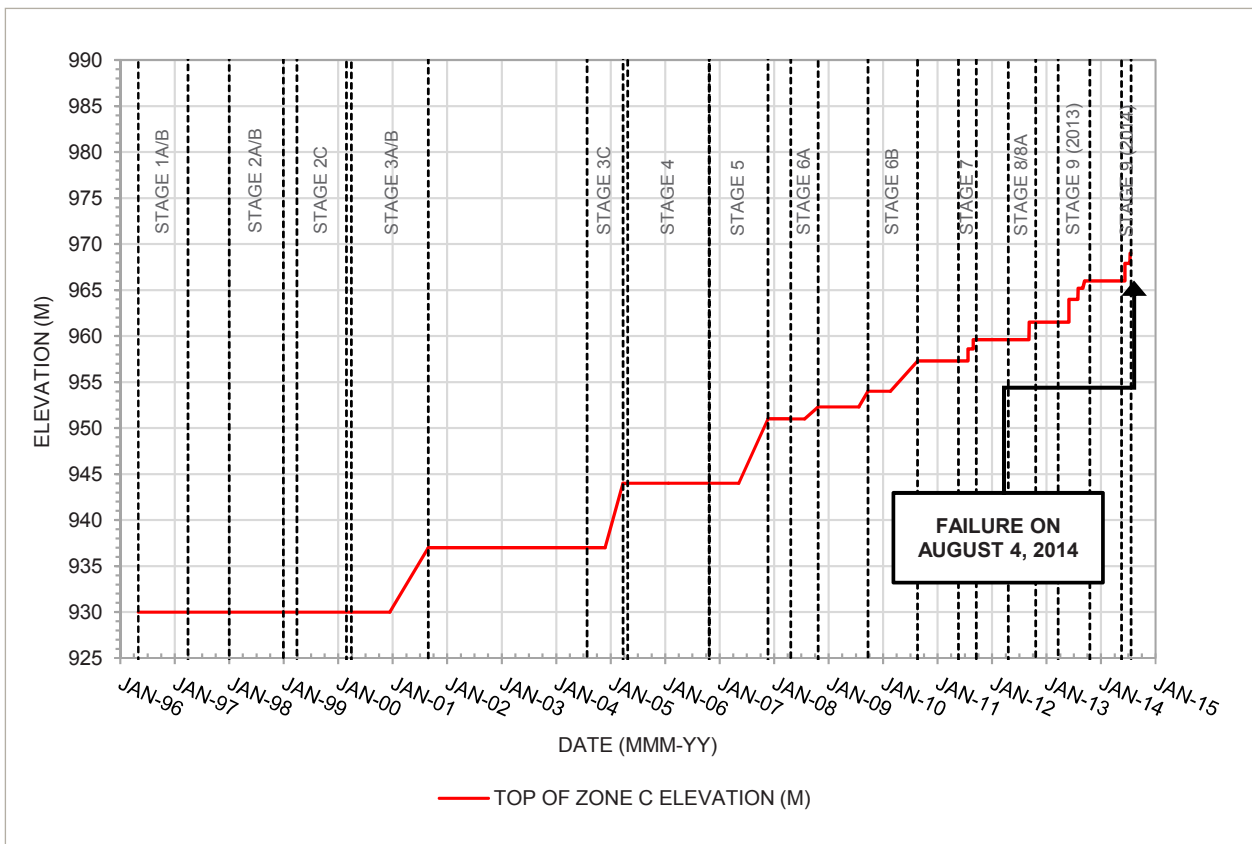
**FIGURE 3.2.2: PERIMETER EMBANKMENT AT SECTION D**



# 3 | Tailings Storage Facility (TSF)

The history of the construction of the embankments is summarized in **Figure 3.2.3**, which indicates each stage of dam raising up to the occurrence of the breach. The Starter Dam for the embankment was constructed in 1996 to a crest elevation of 927.0 metres (m). The embankments were subsequently raised together in stages as shown. Construction of the Stage 9 raise from approximately elevation (El.) 967.5 m to El. 970.0 m was started at the end of April 2014. Following completion of Stage 9, Stage 10 was planned to raise the crest to El. 972.5 m, which would have provided adequate storage to the end of September 2015. Stage 10 was under review for approval at the time of the breach.

**FIGURE 3.2.3: MOUNT POLLEY TSF AND ZONE C (SHELL) TOP ELEVATIONS VERSUS TIME**



## 3 | Tailings Storage Facility (TSF)

### 3.3 POND ELEVATION AT TIME OF BREACH

At the time of the breach, the TSF was permitted under MEM Permit M-200 with approval to raise the crest to El. 970 m. The breach occurred early on August 4, 2014 at a core elevation of El. 969.1 m. Loss of containment was sudden, with no identified precursors. The recorded pond elevation at 6:30 pm on August 3, 2014 was El. 966.83 m.

**Loss of containment  
was sudden, with no identified  
precursors.**

### ENDNOTE

1) MP00044

# 4 | How Did This Dam Fail?

## 4.1 CLASSIFICATION OF DAM FAILURE

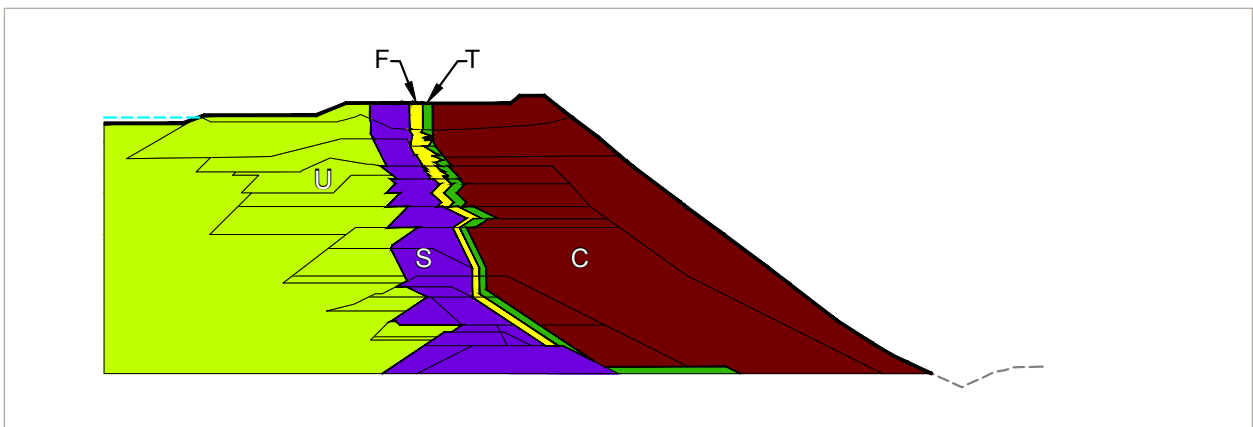
The following directional conventions are adopted throughout, with the dam as the frame of reference:

- Upstream — toward the impoundment interior
- Downstream — away from the impoundment interior
- Right — to the right, looking downstream
- Left — to the left, looking downstream

**Figure 4.1.1** shows a simplified cross-section of the dam. It is constructed of both earth and rockfill and would be classified as a zoned earth and rockfill dam. The specific zones are:

- U Zone Upstream fill
- C Zone Rockfill
- S Zone Till core
- F Zone Filter
- T Zone Transition

**FIGURE 4.1.1: SIMPLIFIED CROSS-SECTION OF THE MOUNT POLLEY DAM**



## 4 | How Did This Dam Fail?

The impervious element of the dam is the till core (Zone S), composed of glacial deposits (till) excavated from selected borrow areas. The duty of the rockfill zone (Zone C) is to support the core, without which the core would not be stable. When seepage flows through the core or if it became cracked, its relatively fine-grained material might erode into the rockfill. The duty of the filter (Zone F) and transition (Zone T) is to collect any seepage coming through the core and to prevent fines from migrating out of it. Zone T is a transition to Zone C and it is intended to stop migration of filter material into Zone C. While Zone C is somewhat compacted to improve its density, this is not sufficient to preclude migration of Zone F into Zone C. Hence, a transition Zone T is needed.

Zone U is composed either of tailings or rockfill if tailings beach material cannot be delivered in time. It should be noted that the core of the dam is inclined slightly in an upstream direction. This configuration is known as modified centreline construction. Zone U provides support on the upstream side of the core. It also has other functions such as keeping clear water away from the core. Zone U tailings would tend to migrate into and fill any cracks that might develop in the core, preserving its function.

### 4.2 POTENTIAL FAILURE MODES

Assessing potential failure modes should be consistent with the characteristics of the breach; it was relatively sudden and with no apparent warning. In addition, by outward comparison, the Perimeter Embankment appears less vulnerable than the Main Embankment design section (**Figure 3.2.1**). The Main Embankment is higher, is designed on the same principles as the Perimeter Embankment, and Zone U has a less developed beach. Moreover, by the time of the breach, small movements had previously been detected in the Main Embankment foundation, and they were being managed by design and construction changes in response to observations. The Panel concluded that, in order to account for such an abrupt event in the Perimeter Embankment, local features likely prevailed.

**The Panel concluded that, in order to account for such an abrupt event in the Perimeter Embankment, local features likely prevailed.**



# 4 | How Did This Dam Fail?

## 4.3 FOUR CLASSES OF FAILURE MECHANISMS CONSIDERED

Based on the experience of the Panel with both water and tailings dams, the Panel determined that the following four classes of failure mechanisms required consideration:

- Human intervention
- Overtopping
- Piping and cracking
- Foundation failure

Before considering each in turn, it is necessary to understand the timeline of activities at the site prior to the failure.

The timeline constructed from construction and personal reports is presented in **Table 4.3.1**. The breach section extends approximately from survey station (Sta.) 4+200 to 4+300. (refer to **Figure 3.1.1**). Key observations are:

- Last construction ending at 6:30 pm, August 3, 2014
- Site observation indicating no issues at 10:30 pm, August 3, 2014
- Operations at perimeter seepage pond, no issues at 11:45 pm, August 3, 2014
- Perimeter seepage pond water fluctuation beginning at 12:45 am, August 4, 2014
- Power lost (likely due to breach) at 1:15 am, August 4, 2014
- Breach identified at 2:05 am, August 4, 2014

**TABLE 4.3.1: TIMELINE OF EVENTS AND ACTIVITIES AT BREACH SECTION AND ADJACENT AREAS**

DATE	ACTIVITY	POND EI.	SOURCE
7/10	Zone F (filter) trenching from 4+300 to 4+750, El. 967.0	966.55	Construction Daily Report (MPMC)
7/14	Zone S (till) placement from 4+305 to 4+925, El. 968.5	966.55	Construction Daily Report <sup>1</sup>
7/15	Zone S (till) placement from 3+980 to 4+305, El. 968.5 Zone S (till) placement from 4+420 to 4+770, El. 968.8	966.55	Construction Daily Report <sup>1*</sup>
7/16	Zone S (till) placement from 3+990 to 4+768 @ El. 968.5 (completed to PE pipe) <sup>i</sup> Zone S (till) placement from 4+395 to 4+757 @ El. 968.8	966.53	Construction Daily Report <sup>1*</sup>
7/17	Zone S (till) placement from 3+995 to 4+395 @ El. 969.1 (completed to PE pipe)	966.60	Construction Daily Report <sup>1*</sup>
7/24	Zone C (rock) placement from 4+525 to 4+650, El. 968.8	966.68	Construction Daily Report <sup>1*</sup>
	Extreme rainfall, Perimeter overflowing <sup>ii</sup>		TSF Leadhand Report
7/25	Zone C (rock) placement from 4+335 to 4+525, El. 968.8	966.73	Construction Daily Report <sup>1*</sup>
	Perimeter still in "Red" with all pumps running, only 1.7 metres left in perimeter overflow		TSF Leadhand Report <sup>2</sup>

## 4 | How Did This Dam Fail?

**TABLE 4.3.1: TIMELINE OF EVENTS AND ACTIVITIES AT BREACH SECTION AND ADJACENT AREAS** *continued*

DATE	ACTIVITY	POND EL.	SOURCE
7/26	Perimeter held @ "6" on scale all day but still overflowing Till pit level went up 20 cm overnight Retrieved piezo below corner 1		TSF Leadhand Report <sup>2</sup>
7/28	Zone C (rock) placement from 4+180 to 4+335, El. 968.8	966.70	Construction Daily Report <sup>1*</sup>
8/01	Zone C (rock) placement (grading down near Corner 1), El. 969.0 Raising of PE pipe in the C zone	966.80	Construction Daily Report <sup>1*</sup>
	Last placement of Zone C (rock) in breach area with four 733s (60T) and one D-8R		Panel Interview 10/22/14
8/02	Placing Zone C on the PE pipe after raising it	966.82	Construction Daily Report <sup>1*</sup>
8/03	6:30 am to 6:30 pm	Placing C zone on the PE pipe after raising it (completed) Grading C Zone (rock) from corner 5 to the PE pipe that has been recently placed by Peterson	966.83 Construction Daily Report <sup>1*</sup>
	10:30 pm	Good berm, good slope, no visible cracks	Shifter Dump Logbook (Mine Ops), B-crew night shift <sup>3</sup>
	11:00 pm	East Perimeter Pond going to alarm in high level within the hour	Dam Breach Report <sup>4</sup>
	11:30 pm	Second pump (perimeter pond) started, nothing unusual noticed	Dam Breach Report <sup>4</sup>
	11:45 pm	Drove from perimeter pond across dam crest to PE pipe and back to Corner 5 (across breach area), nothing unusual noticed	Panel Interview 10/22/14
	12:00 midnight	Second pump drawing down perimeter pond water level (recollection of control instrumentation)	Panel Interview 10/22/14
8/04	12:15 am	Pond water starts to level out (recollection of control instrumentation)	Panel Interview 10/22/14
	12:45 am	Pond water level starts to slightly increase (recollection of control instrumentation)	Panel Interview 10/22/14
	1:00 am	Pond water level rising sharply (recollection of control instrumentation)	Panel Interview 10/22/14
	1:15 am	Lights went out in electrical shop, mill shut down, pond water level spikes sharply (recollection of control instrumentation)	Dam Breach Report <sup>4</sup> Panel Interview 10/22/14
	2:05 am	Dewatering operator discovers that tailings dam had breached	Dam Breach Report <sup>4</sup>

\* PHOTO AVAILABLE IN CONSTRUCTION DAILY REPORT.

<sup>1</sup> "PE PIPE" IS THE RETURN-WATER HDPE LINE FROM THE SEEPAGE RECYCLE PUMP THAT CROSSES THE DAM CREST AT THE LOCATION OF SECTION D, APPROX STA. 3+960 (SEE PHOTO IN CONSTRUCTION DAILY REPORT OF 8/02/14).

<sup>2</sup> "PERIMETER" REFERS TO PERIMETER SEEPAGE POND. "OVERFLOW" REFERS TO OVERFLOW FROM PERIMETER SEEPAGE POND INTO TILL BORROW PIT (PANEL INTERVIEW, 10/22/14).

## 4 | How Did This Dam Fail?

### 4.3.1 HUMAN INTERVENTION

Human intervention may be accidental, such as discharge from a tailings line eroding the structure in an uncontrolled manner, or wilful destruction. A tailings pipeline on the Perimeter Embankment crest was not in service at the time of the breach, and the Panel has found no other evidence of failure due to human intervention.

### 4.3.2 OVERTOPPING

Although water management had been challenging in later years, and an episode of overtopping over portions of the Perimeter Embankment had occurred in May 2014, freeboard was being carefully monitored around the time of the breach as a result of prior insistence on the part of the Ministry of Energy and Mines (MEM). The freeboard with respect to the core at the time of the failure was 2.3 metres (m).<sup>5</sup>

**The Panel has found no evidence of failure due to overtopping prior to breach development.**

The Panel has found no evidence of failure due to overtopping prior to breach development.

### 4.3.3 PIPING AND CRACKING

Piping and cracking of the core of an earth-rockfill dam can lead to internal erosion and ultimately loss of containment. This is one of the most common causes of failure of earth dams and has been much studied. The failure of the Omai Tailings Dam<sup>6</sup> provides an example of a failure by piping and internal erosion.

The following factors were of concern to the Panel:

- 1) Modified centreline tailings dams, while within precedent, are disposed to longitudinal cracking.
- 2) Following Stage 5, the core width was reduced to 5 m, which is thin for the planned hydraulic head; again, this has precedent but requires careful filter and transition design and construction.
- 3) The filter and transition were particularly thin and required meticulous care to be constructed as intended.
- 4) Details of filter and transition construction in as-built drawings indicated departure from intended design.<sup>7</sup>
- 5) Much of the as-placed filter material failed to meet applicable filter criteria and requirements for internal stability of its grading.
- 6) The core had been overtopped in one location for a brief period in 2014, resulting in softening and enhanced deformability.
- 7) The core was not contained by the steep rockfill shell in as stiff a manner as might have been possible.
- 8) A cavity was detected in the core remnant of the left abutment of the breach that was the result of internal erosion, see Appendix C.
- 9) Observed flow to the seepage collection system exhibited a transient spike on April 22, 2013, of the kind

## 4 | How Did This Dam Fail?

sometimes characteristic of internal erosion (see Appendix F for details).

Notwithstanding these concerns, the Panel notes:

- 1) No abnormal seepage observations were detected except for the spike on April 22, 2013 (see Appendix F).
- 2) Sonic drillholes were located as close to the abutments of the breach as safely possible. They did not detect any suspicious piping pathways.
- 3) Excavation of the right abutment of the breach did not find any piping pathways through the core.
- 4) Grading of samples of filter material recovered from the breach area indicate that internal erosion did not produce large flows or overall loss of core integrity (see Appendix C).

Accordingly, and despite the concerns identified by the Panel, it did not find evidence that the breach was caused by piping and/or cracking resulting in uncontrolled internal erosion.

### 4.3.4 FOUNDATION FAILURES

Observations from the Surface Investigations (see section 5.1 and Appendix C for details) provided clear evidence for shearing, bodily lateral displacement, and rotation of the embankment that resulted in the breach. The Panel concluded that the primary cause of the breach was dislocation of the embankment due to foundation failure. This resulted in loss of containment of both the clear water contained in the tailings storage facility (TSF), and tailings, which flowed out of the breach. Clearly, the foundation has behaved in a weaker manner than anticipated in the design. A major focus of this investigation was therefore to determine the foundation characteristics that account for the observed failure mode and to compare the outcome with the design basis.

**Observations from the Surface Investigations provided clear evidence for shearing, bodily lateral displacement, and rotation of the embankment that resulted in the breach.**

A number of circumstances can contribute to such weak behaviour, and all require careful assessment.

It is well-known that glaciated terrain can be exposed to glacial drag forces and leave in the underlying sediments and bedrock, if relatively soft, continuous weak surfaces at the residual strength of the material. The residual strength is the weakest resistance that the material can offer and arises from preferred orientation of platy clay particles. Valley rebound folding and expansion in soft bedrock can also result in weak residual strength materials, but these processes have not acted at the TSF. Examples of large tailings facilities on glacially sheared material at

## 4 | How Did This Dam Fail?

low strengths are the Mildred Lake Settling Basin at the Syncrude Canada Ltd. site<sup>8</sup> and the large TSF at the Zelazny Most Copper Mine.<sup>9</sup> In both cases, movements are slow and are managed by adaptive response to observations.

Another source of unanticipated behaviour can be a deposit within the foundation of a dam that exhibits pronounced strain-weakening behaviour; that is, it loses considerable resistance once its peak resistance is attained. This type of behaviour was discovered during the forensic investigation into the Aznalcollar (Los Frailes) Tailings Dam failure in Spain.<sup>10</sup> The movements in that case were sudden, without any observable precursors. The instrumentation at the time was minimal.

A further source of unanticipated behaviour arises when a structure is being built in stages on a soft substrate that is contractant; that is, it tends to contract, or densify. When such a soil is subjected to shearing due to loading that occurs slowly, the resulting volume changes strengthen the soil as it densifies. This is known as drained loading. However, if the contractant soil were to be loaded too quickly for water to be expelled and permit volume change, pore pressures develop that weaken the soil. This is known as undrained loading, and the resistance is less than its drained equivalent.

Undrained response can also be initiated if the soil displays a rapid reduction in resistance as yielding is initiated, even under drained conditions. This has sometimes been called spontaneous liquefaction when flowslides develop. While the Mount Polley failure was not sufficiently mobile to be regarded as a flow, the concept of spontaneous undrained response cannot be disregarded in a broadly based inquiry such as this. The implications for stability of the undrained response of soft, contractant soils to staged construction were presented at length in a classic paper by Ladd<sup>11</sup> and are discussed in a widely accepted graduate-level text by Duncan and Wright.<sup>12</sup> The Kingston fly ash slurry spill is an example of a TSF that failed by this undrained mechanism (<http://www.tva.gov/kingston/rca/>).

All of the above hypotheses regarding the characteristics of the ground conditions in the breach zone were considered in the Technical Commentary that follows. The Technical Commentary presents the findings arising from both surface and subsurface investigations, laboratory studies, computational analyses, and design, construction, and monitoring reviews, in support of the explanation of the cause of failure.

## 4 | How Did This Dam Fail?

### ENDNOTES

- 1) MP10000
- 2) MP10021
- 3) MP10022
- 4) MP10013
- 5) MP00188
- 6) Vick, S.G., 1996. The failure of the Omai Tailings Dam. *Geotechnical News*, September, pp. 34-39.
- 7) MP00044
- 8) Alencar, J., Morgenstern, N.R. and Chan, D.H., 1994. Analysis of foundation deformations beneath the Syncrude tailings dyke. *Canadian Geotechnical Journal*, Vol. 31, pp. 868-884.
- 9) Jamiolkowski, M., 2014. Soil mechanisms and the Observational Method: Challenges at the Zelazny Most copper tailings disposal facility. *Geotechnique*, Vol. 64, pp. 590-618.
- 10) Gens, A. and Alonso, E.E., 2006. Aznalcollar dam failure – Part 2: Stability conditions and failure mechanism. *Geotechnique*, Vol. 56, pp. 185-201.
- 11) Ladd, C.C., 1991. Stability evaluations during staged construction. *Journal of Geotechnical Engineering*, American Society of Civil Engineers, Vol. 117, pp. 540-615.
- 12) Duncan, J.M. and Wright, S.G., 2005. *Soil Strength and Slope Stability*. John Wiley and Sons, Inc., New Jersey, 298 p.

# 5 | Panel Observations

## 5.1 SURFACE INVESTIGATIONS

Surface investigations of the breach and adjacent areas were conducted to gather evidence about the cause of the failure, to document this evidence, and to provide necessary context for related Panel activities. Appendix C provides a comprehensive account of the surface investigations from which this summary has been compiled. The electronic version of Appendix C also contains a virtual three-dimension (3-D) flyover to help orient the reader to features and interrelationships described here.

### 5.1.1 DATA COLLECTION

The surface investigations made use of imagery from a variety of sources, field mapping on the ground, and exploratory excavation of key features. Data sources and collection activities included the following:

- Review of pre-failure satellite imagery
- Review of a helicopter video made by the Cariboo Regional District during failure
- Review of post-failure helicopter photos by the Panel and airphoto stereopairs
- Review and preprocessing of Panel ground photos
- Preparation of topographical base maps and cross-sections
- Field mapping of ground features and exposures
- Excavation and logging of exploratory works in a remnant section of the dam core

### 5.1.2 KEY FEATURES

An oblique view of the breach area looking upstream is provided in **Figure 5.1.1**. This and the following image were obtained from the Cariboo Regional District's video. The video was taken on the morning of August 4, 2014, about 8 hours after breach initiation with breach outflow still in progress. It provides a unique opportunity to observe how many of the key post-breach features were formed.

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**FIGURE 5.1.1: VIEW LOOKING UPSTREAM THROUGH THE BREACH (ARROW SHOWS DIRECTION OF OUTFLOW)**



The labelled features in **Figure 5.1.1** can be interpreted with reference to the internal zoning of the dam previously provided as **Figure 4.1.1**. On the upstream side of the breach, remnant projections of the dam core (S) can be seen on the left and right abutments. The projection on the right abutment acts like a jetty in directing flow toward the left abutment.

Zone C rockfill (C) is exposed on the left abutment, where it has been eroded by these redirected breach outflows. On the right abutment, the surface of the displaced rockfill (D) was subject to erosional overflow during earlier stages of breach development. It was subsequently protected from erosional undercutting in the main channel by the projecting core remnant and eddies that developed downstream.

The whaleback feature (W) is a linear, uplifted ridge of foundation till that extends across the entire width of the breach. Highly erosion-resistant in both native and compacted forms, the upthrust till here acts as the control section for breach outflow.



## 5 | Panel Observations

A reverse-angle perspective looking downstream through the breach is provided in **Figure 5.1.2**. It shows the damage to the upstream side of the dam and the processes that caused it.

**FIGURE 5.1.2: VIEW LOOKING DOWNSTREAM SHOWING UPSTREAM SIDE OF DAM AND REMAINING TAILINGS**



At this point, two major flow channels have developed within the impoundment that converge at the upstream entry to the breach. In the distance (centre left), one of these can be seen flowing along the left side of the dam. As it did so, it eroded away the supporting tailings. This caused the upstream side of the dam to collapse, leaving the prominent near-vertical face. These structural effects are again best appreciated with reference to **Figure 4.1.1**.

Conditions adjacent to the right side of the dam illustrate how the combined action of fluvial erosion and tailings flowsliding produced similar effects. The active flowslide (B) has left a semicircular headscarp that is progressing back and undermining the Zone U tailings supporting the upstream side of the dam core. Arcuate headscarps of earlier flowslides (A) have captured and concentrated overland flows from surface water remaining in the impoundment. The resulting cascades readily transport flowslide debris, while at the same time causing backward erosion of the headscarps by scour within their terraced plunge pools.

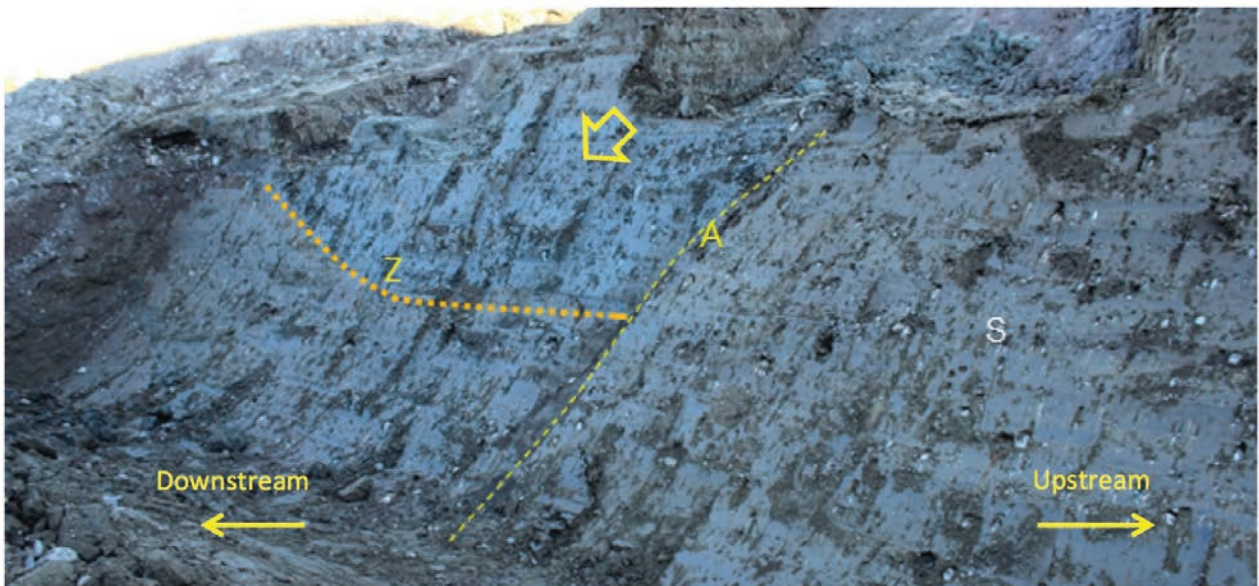
## 5 | Panel Observations

### 5.1.3 FOUNDATION SLIDING

The surface investigations produced direct evidence for foundation sliding as the initiating mechanism for the breach. This is most clearly demonstrated by a shear surface observed within the remnant core projection on the right abutment at the location shown previously in **Figure 5.1.1**.

**Figure 5.1.3** shows an excavated exposure of the shear surface (A) through the Zone S core material (S). The marker bed (Z) was not present on the upstream footwall side (right in photo), indicating at least 3.3 metres (m) of downthrow on the downstream hanging wall. Appendix C, section 3.7, contains more detail on the orientation and configuration of the shear surface.

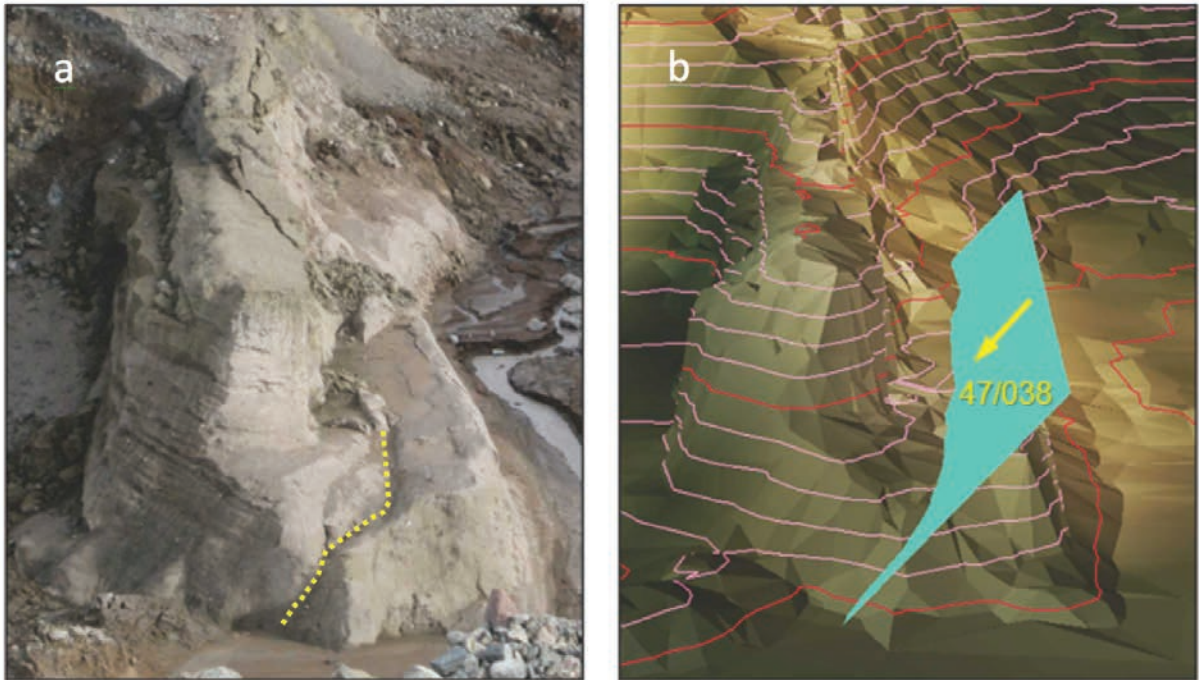
**FIGURE 5.1.3: SHEAR SURFACE THROUGH REMNANT DAM CORE (ARROW INDICATES DIRECTION OF DOWNDROP)**



The surface trace and 3-D orientation of the shear are shown on the core remnant in Figure 5.1.4, with a dip angle and direction as indicated.

# 5 | Panel Observations

**FIGURE 5.1.4: PHOTO (a) AND SURFACE MODEL (b) SHOWING SHEAR SURFACE ORIENTATION**





## 5 | Panel Observations

Other artifacts of foundation sliding are evident elsewhere. As indicated in **Figure 5.1.5**, lift lines in the Zone C rockfill on the left abutment (C) are tilted at an inclination of 7° to 10°, with corollary inclinations on the right abutment of 5° to 14°. Also shown in **Figure 5.1.5** are the Zone S core (S), with a containment dike (K) under construction in the background. Scarps higher on the left abutment were produced by post-sliding downdrop of large slump blocks into the breach due to erosional undercutting at foundation level.

**FIGURE 5.1.5: APPARENT BEDDING ROTATION ON LEFT ABUTMENT OF BREACH (SEPT. 4, 2014 PHOTO)**



## 5 | Panel Observations

**Figure 5.1.6** shows the right abutment, with the left abutment in the background. Noteworthy features include open cracks (O) and headscarps (H) at higher elevations. At the downstream toe, upthrust of foundation till (L) ranging from 2.8 m to 3.5 m has occurred along an alignment collinear with the whaleback (W).

The mass of rockfill (D) from the upper headscarps to the lower upthrusted till was rotated and displaced by foundation sliding. It was preserved when surface sheet flow was terminated by breach downcutting on the left side. From displacement of surface lineations, lateral (downstream) translation of 11 m occurred in the vicinity of the arrows in the figure.

**FIGURE 5.1.6: SLIDING-RELATED FEATURES AT RIGHT ABUTMENT (SEPT. 4, 2014 PHOTO)**

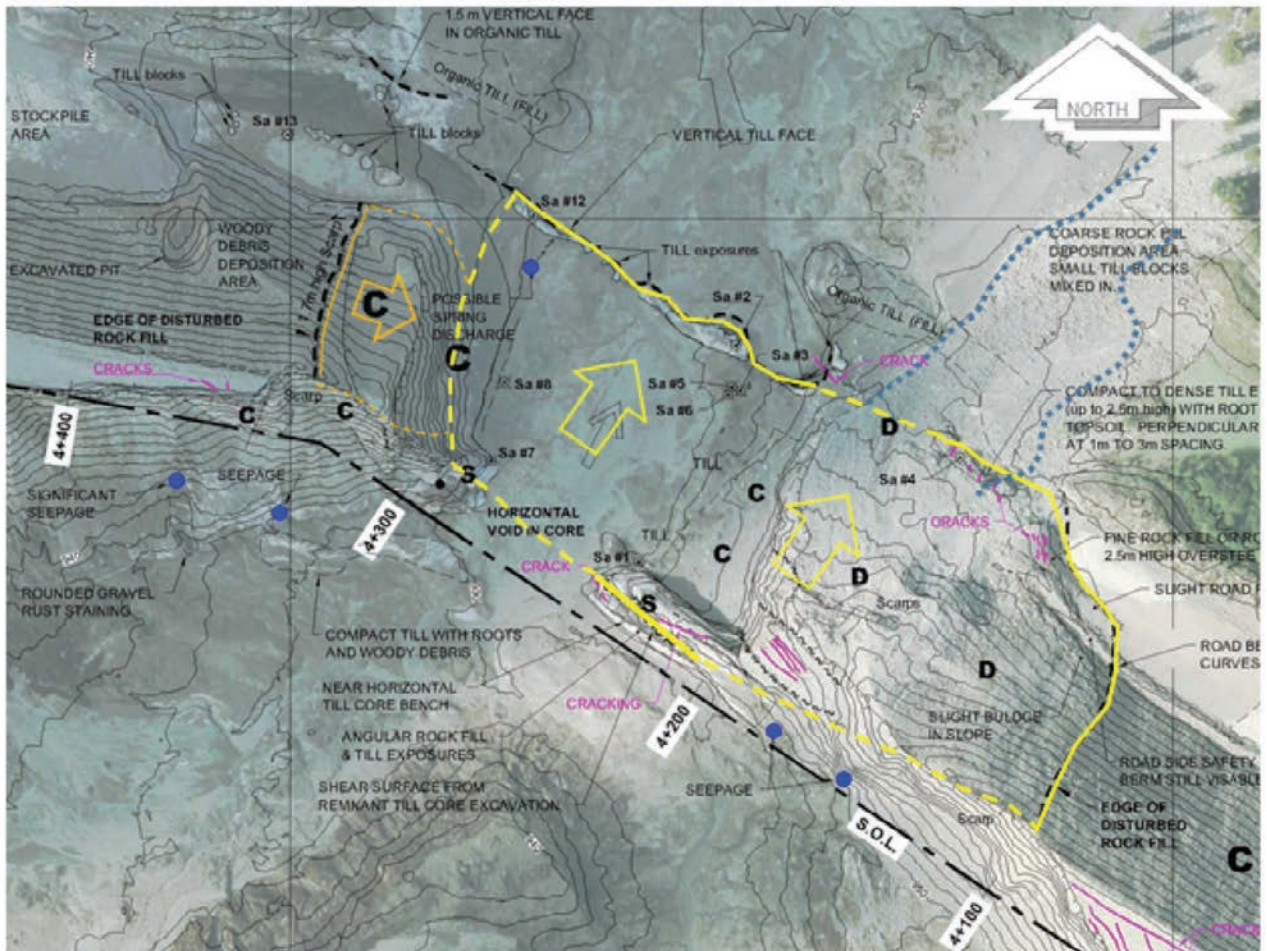




## 5 | Panel Observations

Surface investigations delineate the limits of foundation sliding given in **Figure 5.1.7**, with the solid and dashed yellow lines indicating observed and inferred boundaries, respectively. These are superimposed on the post-failure orthophoto and contours to show the extent of mass movement in relation to the breach. Arrows indicate directions of movement from surface observations.

**FIGURE 5.1.7: PLAN SHOWING DIRECTION AND EXTENT OF MASS MOVEMENTS**



## 5 | Panel Observations

Movements are summarized in **Table 5.1.1** below.

**TABLE 5.1.1: MEASURED AND INFERRED SLIDE MOVEMENTS**

LOCATION	DISPLACEMENTS AND ORIENTATIONS
DOWNSTREAM TOE	Vertical: 2.8 to 3.5 m upward Horizontal: 11 m downstream
UPSTREAM SHEAR SURFACE	Vertical: >3.3 m downward Dip: 47 degrees
RIGHT ABUTMENT	Rotation: 5 to 14 degrees
LEFT ABUTMENT	Rotation: 7 to 10 degrees

## 5 | Panel Observations

### 5.1.4 INTERNAL EROSION

A void shown in **Figure 5.1.8** was observed on the upstream side of the left abutment, measuring 0.7 m by 0.3 m and extending back 1.1 m into the Zone S core. The angular corners and abrupt transitions at the opening are distinct from the smoother, more rounded surfaces produced by surface erosion at other locations.

**FIGURE 5.1.8: VOID ON UPSTREAM SIDE OF LEFT ABUTMENT**





## 5 | Panel Observations

Additionally, Zone F filter material immediately downstream from the core was sampled in the right abutment excavation. Gradation data show that none of these samples met filter criteria that would have enabled them to prevent transport of fines from the till. Together, these factors suggest internal erosion as the likely cause of the left abutment void.

This filter material also has an internally unstable gradation, such that its finer fraction is free to pass through the voids in the coarser fraction under sufficient flow velocity. However, the fact that this finer fraction is still present means that high discharge through the filter, and therefore through the core, did not occur. Moreover, painstaking excavation and thorough logging found no evidence of other such voids in the excavated core. Nor were continuous cracks or softened zones indicative of hydraulic fracturing discovered. These factors suggest that internal erosion was not pervasive over the breach area or sufficiently severe to have compromised core integrity overall.

**These factors suggest that internal erosion was not pervasive over the breach area or sufficiently severe to have compromised core integrity overall.**

# 5 | Panel Observations

## 5.2 SUBSURFACE INVESTIGATIONS

### 5.2.1 INTRODUCTION

Following the breach of the Mount Polley Perimeter Embankment, site investigation programs were initiated by the Mines Inspector team with Klohn Crippen Berger (KCB) as the geotechnical lead, Mount Polley Mining Corporation (MPMC) with Golder Associates (Golder) as the geotechnical lead, and the Panel, with the support of Thurber Engineering Limited (Thurber). This section summarizes the Panel's program, its outcome, and key findings that affect other aspects of the Panel's activities. Appendix D provides further details of Thurber's work, results and interpretations. Results of the KCB work are available in Appendix B.<sup>1</sup> The Panel relied on factual data collected by both Thurber and KCB, but made its own interpretations of these data.

### 5.2.2 JOINT SITE INVESTIGATION

In early September 2014, MPMC invited the Panel to participate in a coordination meeting with KCB and Golder to review proposed joint site investigation plans consisting of:

- Geophysics — Direct current resistivity, induced polarity and seismic refraction surveys.
- Drilling and coring — Sonic drilling to allow initial foundation characterization at the dam breach and adjacent areas, and mud rotary drilling and sampling with focus on clays and silts designated the Upper Glaciolacustrine Unit (Upper GLU).
- In situ testing and instrumentation — cone penetration test (CPT) and piezometer and inclinometer installation at selected locations.

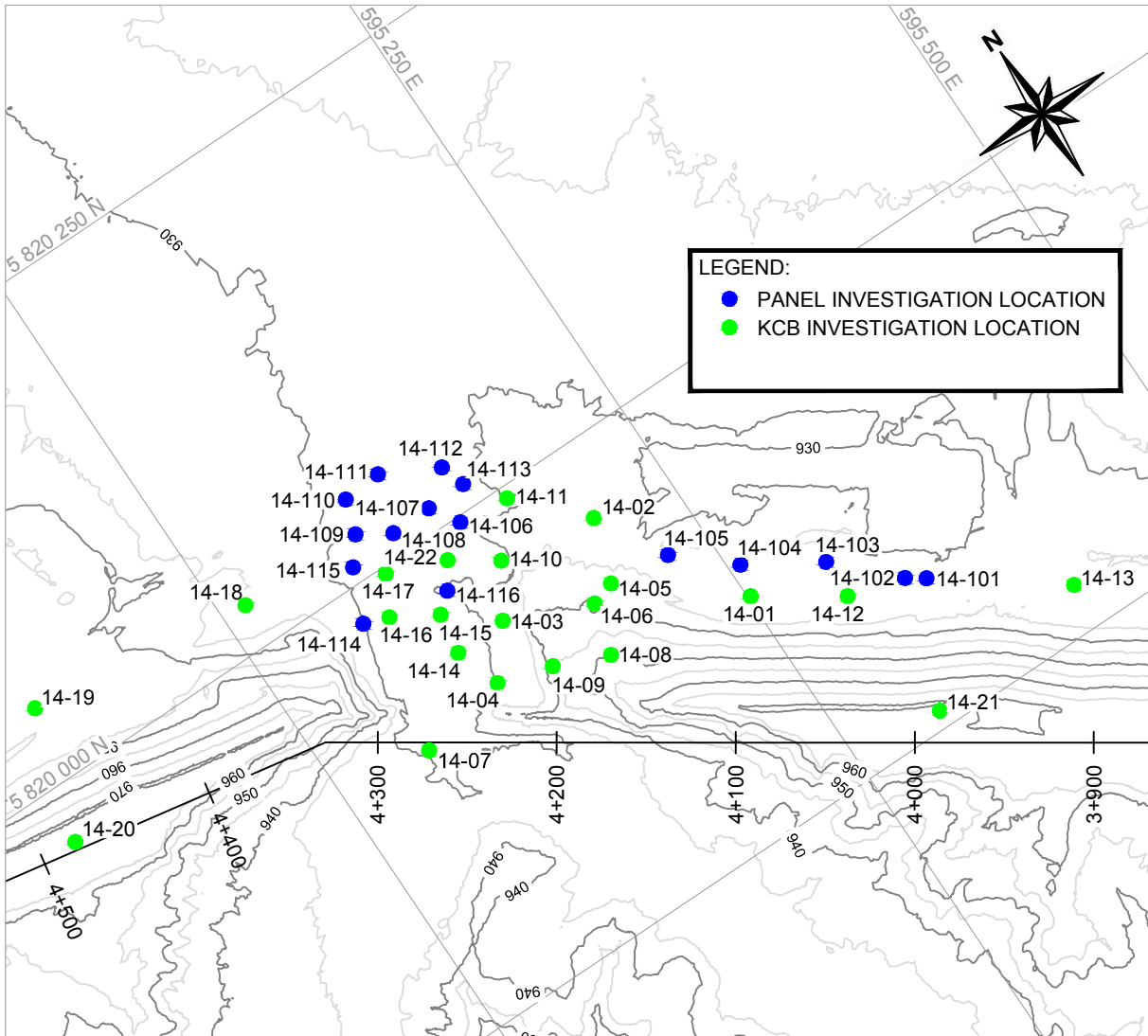
Safe work plans had to be implemented to establish access limits with respect to the remaining breach abutments. These limits influenced the locations of the final geophysics lines and the drillhole locations.

KCB field engineers took large numbers of samples from the sonic cores for routine or index testing in the KCB laboratory. Index test results were shared with the Panel. Thurber also collected samples for index testing from a number of locations during the site mapping and other activities for testing in the Thurber laboratories. All index test results are presented in Appendix D, Attachment 7.

Daily reports describing fieldwork progress were shared with all parties. Weekly conference calls, in which the Panel participated, served to further coordinate the fieldwork and provide updates. Adjustments were made to the detailed locations of the geophysics lines as well as drillholes as the program was implemented. **Figure 5.2.1** shows the final locations of the joint site investigation holes in and around the breach area.

# 5 | Panel Observations

FIGURE 5.2.1: JOINT AND PANEL SITE INVESTIGATION DRILLHOLE LOCATIONS



Thurber's field engineer observed the sonic drilling by KCB and logged all the sonic holes in parallel with the KCB personnel. The Thurber logs of the KCB sonic holes are provided in Appendix D, Attachment 1. Related seismic and resistivity surveys are also described in Appendix D, Attachment 1.

# 5 | Panel Observations

## 5.2.3 PANEL SITE INVESTIGATION

About half of the locations for the KCB investigation were located in the failed section footprint as well as through the remaining embankment into the underlying foundation. The Panel developed a separate field program that allowed in situ testing and sampling at a larger number of locations where foundation materials had not been preloaded by the embankment. The Panel site investigation consisted of:

- CPT and vane testing to characterize the foundation stratigraphy and to identify sampling locations for advanced shear and consolidation testing (refer to section 5.3 for further details of the laboratory testing).
- Mud rotary drilling to obtain disturbed and undisturbed samples of the Upper GLU and other units for laboratory testing.
- Pressuremeter testing in the till to obtain shear strength and shear modulus values.
- Drilling and sampling using Large Penetration Testing (LPT) in selected areas of foundation till.

Details of drilling methods, in situ testing and related information are included in Appendix D. Excavation, sampling and related laboratory testing are described in Appendix C.

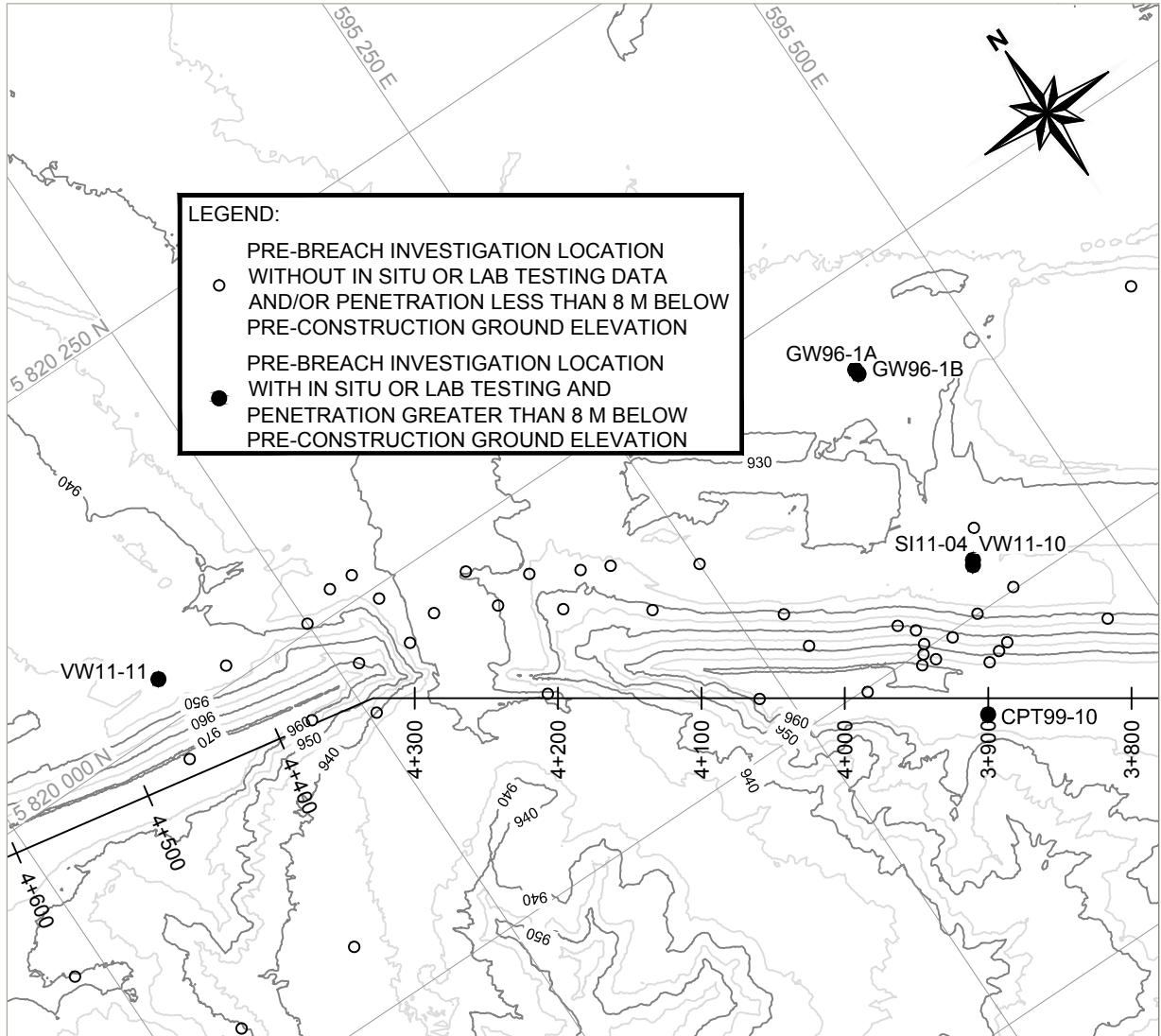
## 5.2.4 PRE-FAILURE SITE INVESTIGATIONS IN BREACH AREA

Appendix D provides a summary of pre-failure site investigations for the Mount Polley tailings storage facility (TSF). Knight Piésold (KP) performed site investigations in the early to mid-1990s for the design of the facility. Additional site investigations and laboratory testing were done during operations, notably a sonic drilling and instrumentation program implemented by AMEC in 2011. **Figure 5.2.2** shows all the geotechnical drillhole locations for pre-failure investigations in the breach area.

While a large number of locations are shown, many were condemnation holes or shallow test pits of limited usefulness for embankment design purposes. As subsequently discussed, the Panel found the critical soils in the breach area at depths of about 8 m. There are only four locations where the holes were deeper than 8 m and where in situ or laboratory testing was done. None of these locations were in the area where the breach occurred.

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**FIGURE 5.2.2 PRE-FAILURE SITE INVESTIGATION DRILLHOLE LOCATIONS IN BREACH AREA**



Based on the subsurface investigations and related laboratory data, the Panel derived several key findings that informed its larger efforts. These are discussed in the following sections.

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## 5.2.5 CONTROLLING STRATIGRAPHY

Soils in the breach area are of three main types, all glacially deposited. These include:

- Glaciolacustrine soils (designated GLU) deposited in standing water.
- Glaciofluvial, or streamchannel, deposits.
- Glacial tills produced by glacial transport and reworking.

Appendix D describes the interpreted depositional environment that results in the generalized sequence shown below.

**FIGURE 5.2.3: GENERALIZED SOIL STRATIGRAPHY IN BREACH AREA**

MAJOR STRATIGRAPHIC UNIT	STRATIGRAPHIC SUB-UNIT
UPPER TILL	
	UPPER GLACIOLACUSTRINE (UPPER GLU)
LOWER TILLS	LOWER BASAL TILL
	LOWER GLACIOLACUSTRINE (LOWER GLU)
	GLACIOFLUVIAL
	LOWER BASAL TILL
WEAK BEDROCK	

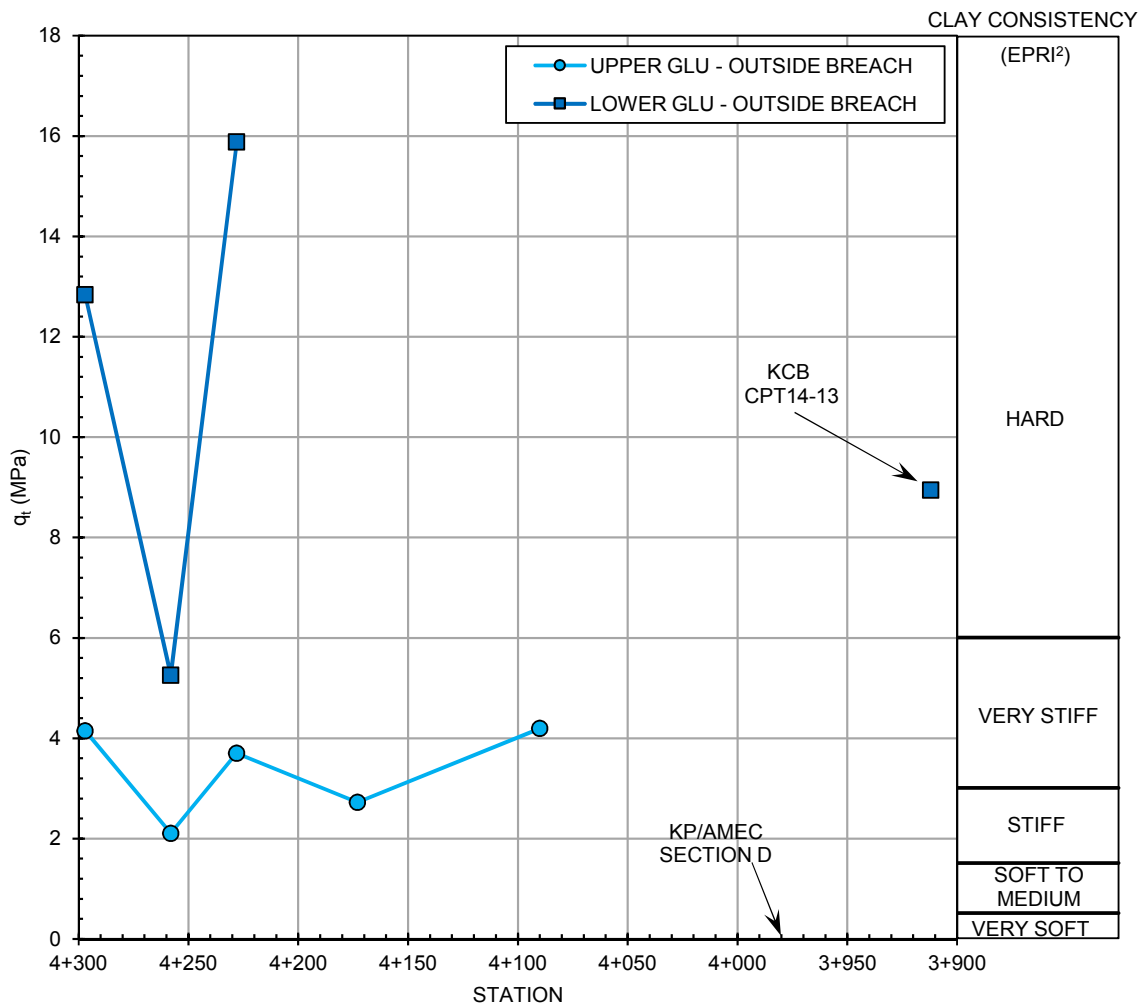
Of special significance are the two glaciolacustrine units designated Upper and Lower GLU, shown in **Figure 5.2.3**, in turquoise and blue, respectively. Both consist of thinly laminated, or varved, silts and clays, and both classify predominantly as low- to high-plasticity clay (CL to CH). They can be distinguished by differences in their pre-failure water content, CPT tip resistance, and overconsolidation ratio (OCR). Establishing these differences requires looking to areas outside the embankment footprint, or those covered by slide debris, in order to eliminate preloading effects. The resulting comparisons therefore reflect initial pre-construction conditions.

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Firstly, the difference in water content is substantial. The average of mean values from individual borings for the Upper GLU is 32%, compared to 24% for the Lower GLU.

Secondly, CPT tip resistance in the two units is distinctly different. **Figure 5.2.4** shows that tip resistance  $q_t$  for the Upper GLU is less than one-half of that for the Lower GLU across the breach area. Using  $q_t$  as a measure of clay consistency, the Upper GLU classifies as stiff to very stiff, while the Lower GLU classifies as very stiff to hard.

**FIGURE 5.2.4: LONGITUDINAL VARIATION IN CPT TIP RESISTANCE**



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During their depositional history, glaciolacustrine deposits can experience episodes of drying, freezing, glacial overriding, or other factors that consolidate them to varying degrees. The result is to induce an effective preconsolidation pressure, designated  $\sigma'_p$ , that has a substantial influence on undrained strength properties. This effect can also be expressed as the ratio of the preconsolidation pressure to the effective stress in the ground, termed OCR. In general, higher OCR and higher  $\sigma'_p$  correlate with higher undrained strength.

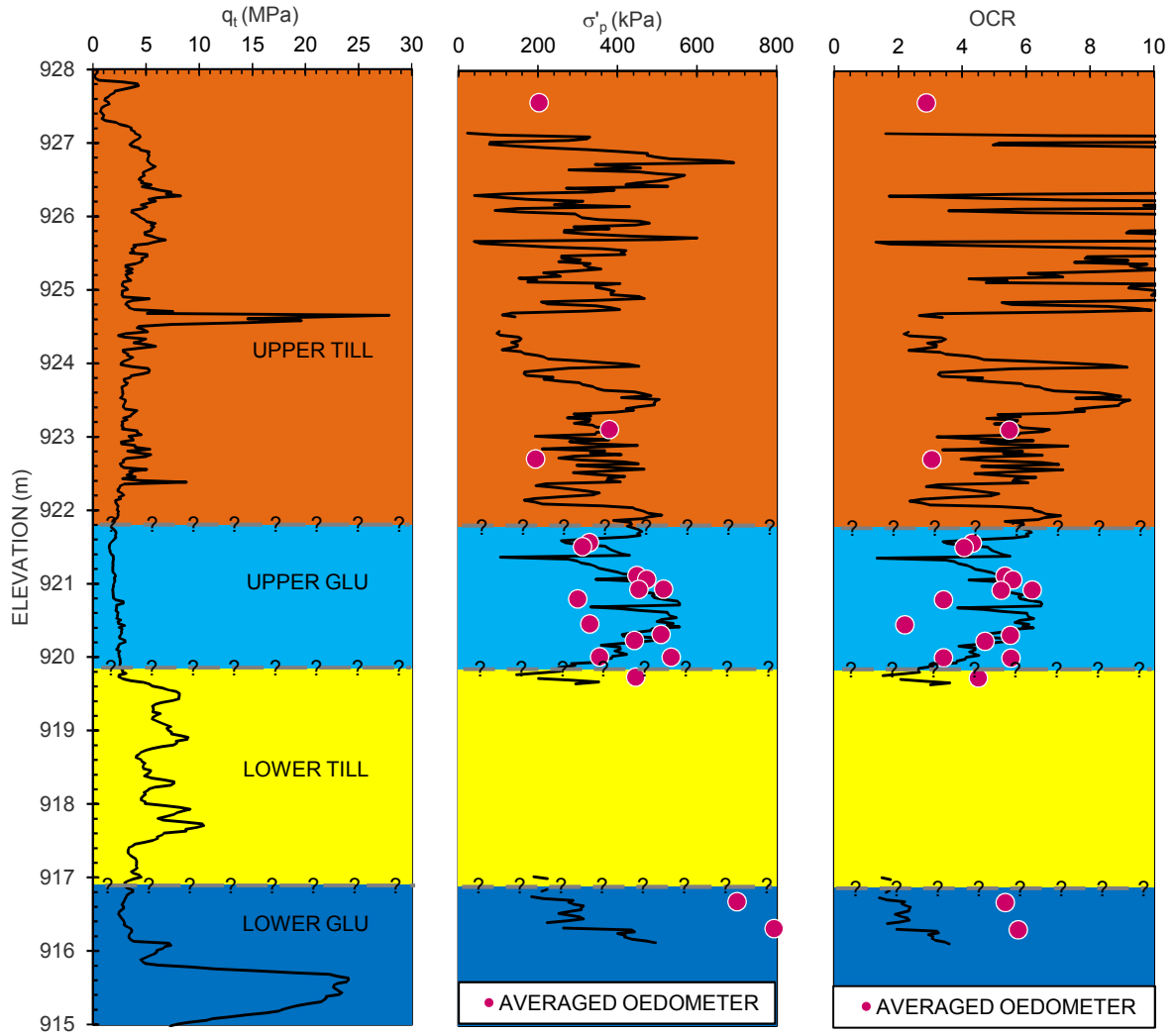
But when the applied stress increases, for example, due to placement of overlying dam fill, OCR decreases. If the higher stress reaches or exceeds  $\sigma'_p$ , the beneficial effects of preconsolidation no longer pertain, and the clay is said to be normally consolidated with  $\text{OCR} = 1.0$ . Together, the preconsolidation of a clay, the stresses it experiences, and the changes in these stresses are called its stress history, which has a major influence on its undrained strength.

These factors are reflected in the Upper and Lower GLU, where the initial pre-construction  $\sigma'_p$  and OCR are compiled on a composite plot in **Figure 5.2.5** using CPT data from RCPT14-107 and all available oedometer data.



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**FIGURE 5.2.5: INITIAL (PRE-CONSTRUCTION) STRESS HISTORY**



The continuous plots of  $\sigma'_p$  and OCR adopt published CPT correlations,<sup>2</sup> while the laboratory-derived data points are taken from Appendix E. From laboratory data, the average  $\sigma'_p$  for the Upper and Lower GLU units is 433 kilopascals (kPa) and 748 kPa respectively, with corresponding OCRs of 6.0 and 6.9.

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The differences in properties are summarized in **Table 5.2.1**. Drawing D18 in Appendix D provides further details of the properties.

**TABLE 5.2.1 PRE-CONSTRUCTION PROPERTIES OF UPPER AND LOWER GLU IN BREACH AREA**

STRATIGRAPHIC UNIT	WATER CONTENT, AVG. AND RANGE	CPT TIP RESISTANCE, $q_t$ AVG., RANGE, AND CONSISTENCY	PRECONSOLIDATION PRESSURE, $\sigma'_p$ AVG. AND RANGE	OVERCONSOLIDATION RATIO, OCR AVG. AND RANGE
Upper GLU	32% (19 — 53)	3.4 MPa (2.1 — 4.2) (stiff to v. stiff)	433 kPa (312 — 535)	6.0 (4.1 — 7.7)
Lower GLU	24% (19 — 29)	11.4 MPa (5.6 — 16) (v. stiff to hard)	748 kPa (701 — 794)	6.9 (6.7 — 7.2)

Taken together, these properties show that the Upper GLU is the weaker of the two units.

## 5.2.6 EXTENT AND CONTINUITY OF UPPER GLU

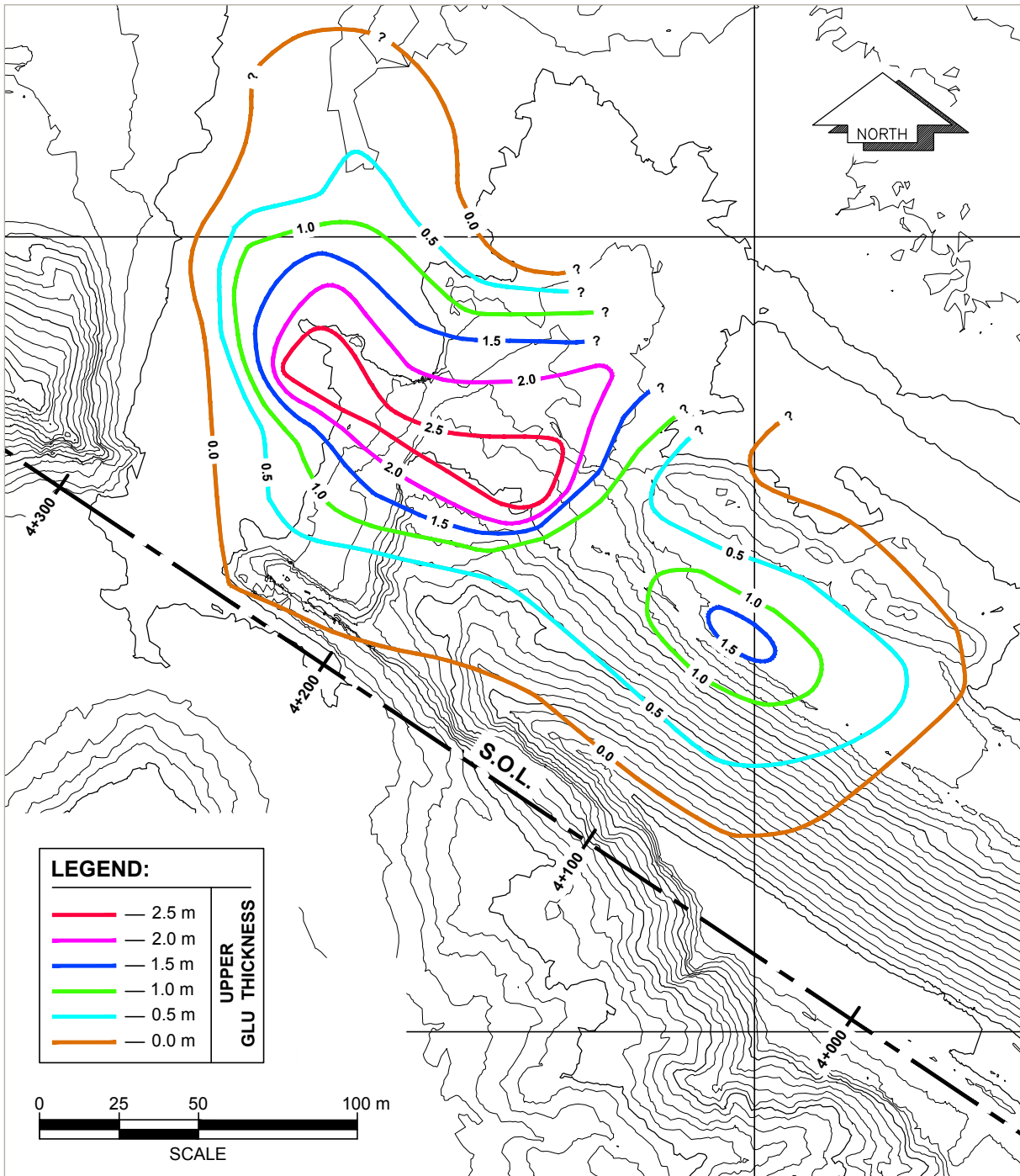
Having targeted the Upper GLU as the controlling stratum, it is of further interest to determine its extent. These results are also highly significant. **Figure 5.2.6** demonstrates that the Upper GLU is not pervasive throughout this entire section of the Perimeter Embankment. But is present in the area beneath the footprint between Sta. 4+050 and Sta. 4+300. Moreover, the greatest thickness directly underlies the remaining slide debris on the right side of the breach, thinning toward the left but still extending across the entire width. The maximum thickness also directly underlies the location of the downstream toe of the embankment at the time of failure.

**Having targeted the Upper GLU as the controlling stratum, it is of further interest to determine its extent. These results are also highly significant.**

There are smaller-scale variations even within this area. **Figure 5.2.6** shows a localized thickening to the east, just at the limit of the slide from **Figure 5.1.7**.

# 5 | Panel Observations

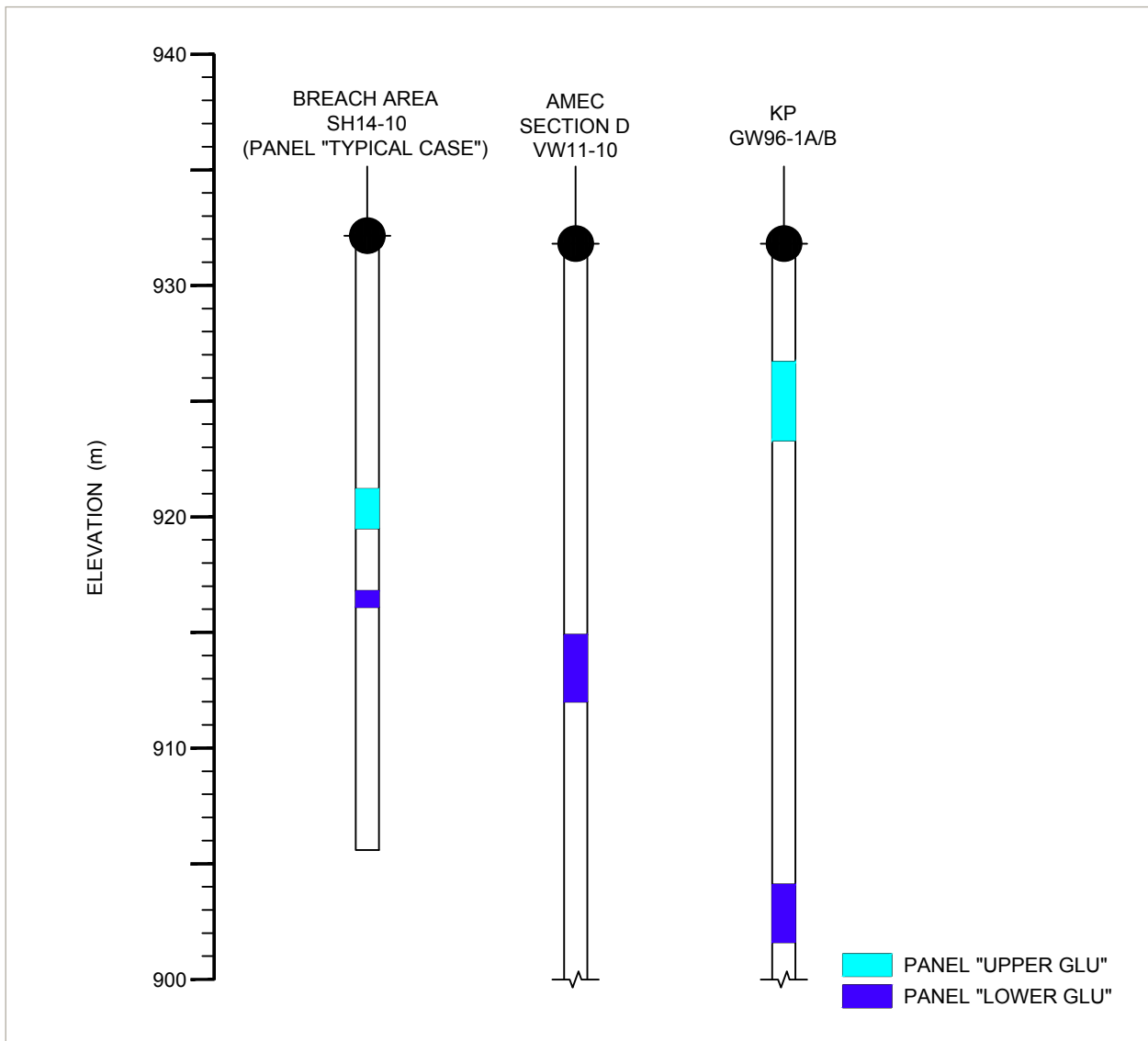
FIGURE 5.2.6: CONTOURS OF UPPER GLU THICKNESS IN BREACH AREA



## 5 | Panel Observations

Stratigraphic variations on a larger scale are also apparent from **Figure 5.2.7**, which relates the Upper and Lower GLU units at the breach to glaciolacustrine soils elsewhere at boring locations presented previously in **Figure 5.2.2**. The Upper GLU at the breach shows apparent similarities to glaciolacustrine soils at similar elevation in GW96-1A that would be characterized as soft to medium-stiff according to their standard penetration test (SPT) blow count of 6. On the other hand, the uppermost GLU layer encountered in VW11-10 has an average water content of 23%, which corresponds closely to that of the Lower GLU shown in **Table 5.2.1**. Details are provided in Appendix D.

**FIGURE 5.2.7: COMPARISON OF GLU UNITS IN BREACH TO OTHER AREAS**



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These illustrations of both small-scale and large-scale variation in stratigraphy and properties of the GLU materials serve to highlight the complexity that their depositional environment produced. This degree of geologic complexity discourages attempts at broader generalization beyond the immediate areas where subsurface data have been obtained.

**Both small-scale and large-scale variation in stratigraphy and properties of the GLU materials serve to highlight the complexity that their depositional environment produced.**

# 5 | Panel Observations

## 5.2.7 LOCATION AND CHARACTERISTICS OF FAILURE SURFACE

Section 5.1 previously identified the entry of the failure surface through the surviving core remnant and into the upper foundation till. The subsurface investigations described here reveal the nature and location of the failure surface at depth.

As will be chronicled in section 5.4, the presence of a glacially pre-sheared surface in the dam foundation posed significant uncertainty throughout the design process. This type of pre-shearing, with the residual strength it produces, was also hypothesized as a potential failure mechanism by the Panel in section 4.3.4. Commensurate effort was devoted to detecting the presence of pre-shearing in foundation soils within the breach.

Pre-shearing in stiff, clayey soils manifests as a thin (a few millimetres to a few centimetres) zone with slickensides—shiny surfaces polished by shearing—on both sides of the zone or within it. These surfaces are continuous and traceable between borings, often along bedding. While detailed logging did show some small, discontinuous slickensided surfaces at random orientations, an expected condition in stiff clays, no continuous surfaces common to multiple borings were found. In this respect, the Panel's investigation at the breach corroborated the more general conclusion of the 2011 site investigation.<sup>3</sup>

Even so, the Upper GLU exhibited other signs of shearing inside but not outside the breach. For example, **Figure 5.2.8** compares the Upper GLU for these two locations.

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**FIGURE 5.2.8 UPPER GLU (a) INSIDE THE BREACH, (b) OUTSIDE THE BREACH**



The thinly laminated, planar varving outside the breach contrasts sharply with the contorted and folded laminations within it. This is consistent with shearing of the Upper GLU having occurred within the breach.

Additionally, CPT tip resistance  $q_t$  in the Upper GLU varies systematically. Drawings D19 and D20 in Appendix D show that average  $q_t$  inside the breach is only about one-third to two-thirds of that outside of it (inferred sensitivity of 1.0 to 3.0), reflecting the effects of remoulding attributable to shearing. Hence, both visual inspection and CPT data indicate that the failure produced shearing in the Upper GLU. This, together with its less favourable properties summarized in **Table 5.2.1**, identifies the Upper GLU as the location of the failure surface in the analyses to be presented in section 6.



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## 5.2.8 COMMENTARY

The key findings of the subsurface investigations with regard to the failure mechanism can be summarized as follows:

- The Upper GLU can be distinguished as a distinct foundation unit based on its water content and other properties.
- The failure occurred within varved silts and clays of the Upper GLU.
- There is no indication of pre-shearing in these or other foundation soils.
- Stratigraphic variability reflects a complex geologic environment and depositional history.

Beyond these immediate findings lie other insights that concern characterization of the GLU during the design process. These are summarized below:

- The discontinuous Upper GLU stratum, the seat of the failure, was infelicitously situated at the worst possible place in the dam foundation.
- The type and extent of pre-failure site investigations were not sufficient to detect this stratum or identify its critical nature.
- The strength behaviour of the GLU was misinterpreted.

The first two of these points are evident from the material presented above. The third requires explanation.

From the outset, the stiffness of GLU materials in the Main Embankment foundation was recognized and attributed to overconsolidation. In response to a review comment by the Ministry of Energy and Mines (MEM), KP obtained samples of GLU materials at the Main Embankment in 1995.<sup>4</sup> Commenting on the characteristics of these soils, KP made the following observations:

*Two additional Shelby samples were recently collected (May 16, 1995) during the soil investigation survey. These samples were obtained from the glaciolacustrine sediments and have confirmed that the foundation materials consist of dense, overconsolidated materials. In fact, it was extremely difficult to insert the Shelby tubes in the field and it was not possible to extract the undisturbed samples from the tubes in the laboratory... It is unlikely that any significant pore pressure development will occur in these materials during construction of the embankment.*

KP concluded that no significant pore pressures would develop, but did not directly relate this to the effective-stress strength properties its stability analyses adopted. This connection was made explicit much later in AMEC's 2011 Geotechnical Site Investigation.<sup>5</sup>

## 5 | Panel Observations

Among other things, the AMEC report compiled all available data for Liquidity Index (LI) at the Main Embankment, a laboratory parameter that can be correlated to preconsolidation pressure  $\sigma'_p$ . The report noted that a number of GLU samples had low LI values near zero, some of them even negative, pointing again to the overconsolidated condition of the GLU. Elaborating on the strength interpretation this supported, the report went on to say:

*Moreover, for heavily overconsolidated soils with high fines contents (such as the GLU) that will shear in an undrained manner due to low hydraulic conductivity, the undrained shear strength will typically exceed the drained shear strength, owing to negative shear-induced pore pressure.*

Thus, undrained strength could be disregarded for the GLU, with drained (effective-stress) strength applicable instead.

Review of the AMEC data calls into question the premise of this conclusion. While most of the LI values were indeed low, fully one-third of them were equal to or greater than 0.5. This means that significant portions of the GLU beneath the Main Embankment were not so heavily overconsolidated. From published correlations,<sup>6</sup> the Panel estimates that  $\sigma'_p$  for these higher LI values ranged from about 250 to 575 kPa, quite similar to the range for the Upper GLU at the breach from **Table 5.2.1**. These  $\sigma'_p$  values correspond to an average OCR of only about 3, given the loading conditions of the catalogued samples, insufficiently high to warrant neglecting undrained strength.

But more than this, the assessment did not account for stress history—how these loading conditions varied at different locations beneath the dam or how they would change over time. Stage 7 of the Main Embankment had just been completed at the time of 2011 site investigation. The Panel estimates that normally consolidated conditions (OCR=1) had already been reached beneath the crest of Stage 2 years before and would continue to propagate outward beneath the slope as the dam grew higher. The key factor that went unrecognized was that undrained strength behaviour would unequivocally control for these normally consolidated conditions.

**The key factor that went unrecognized was that undrained strength behaviour would unequivocally control for these normally consolidated conditions.**

The same effect, equally unrecognized, would occur at the Perimeter Embankment breach section. Normally consolidated conditions, and the governing undrained strength accompanying them, would first develop beneath the crest during Stage 5 and continue to spread thereafter. This would set the stage for much that followed.

# 5 | Panel Observations

## 5.3 ADVANCED LABORATORY STUDIES

### 5.3.1 INTRODUCTION

A distinction can be made between routine and advanced laboratory studies. Routine laboratory studies are performed as part of the description and classification of materials encountered in a site characterization study. Routine laboratory studies undertaken in this investigation have been reported as part of the description of materials identified in the site characterization investigation (see Appendix D). Advanced laboratory studies are undertaken to aid in the explanation of the physical response of a soil to loading. Important responses are the reduction in volume of a soil when loaded, reflected by consolidation testing, and the ultimate resistance of a soil specimen, as measured by a variety of shear strength tests.

### 5.3.2 SAMPLING

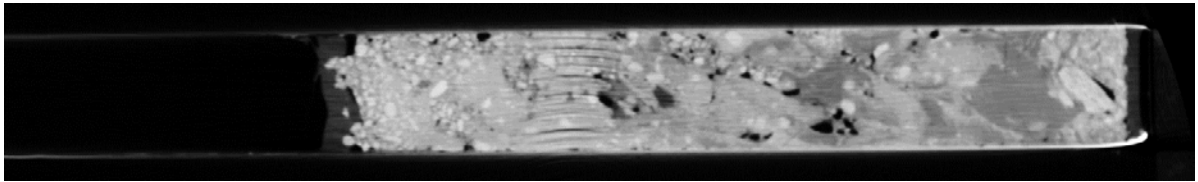
The joint site investigation, summarized in section 5.2, concentrated on the ground conditions adjacent to the breach that would have been affected by the ground movements. The Panel-directed investigation concentrated on the ground conditions adjacent to the disturbed zone in order to provide the opportunity to inspect soil conditions that would not have been affected by the ground movements. Obtaining undisturbed samples for both inspection and advanced laboratory studies was an integral objective of this investigation.

Obtaining undisturbed samples requires pushing a thin-walled sampler into the ground. Given the conditions encountered, this was not a straightforward exercise. The till contains numerous rocks, and even the fine-grained glaciolacustrine (GLU) deposits contain gravel-sized pieces, most likely deposited during melt of ice rafts.

The inventory of samples that were potentially useful for undisturbed sample testing is tabulated in Appendix E. All samples were subject to scanning at FP Innovations at the University of British Columbia (UBC). This facility can undertake both X-ray and CT scanning on large items. Both digital radiography and CT scans were completed on all sample tubes. Observation of internal disturbance, voids or natural structure aided in the quality control. The horizontal CT scans ultimately proved best to determine complex interlayering and to detect voids. **Figure 5.3.1** displays a sample of till (MR14-104-SA8) that exhibits significant sample disturbance, together with a sample of the GLU (MR14-106E-SA3) that shows internal structure with minimal disturbance. Scans performed on the inventory of samples obtained are included in Appendix E.

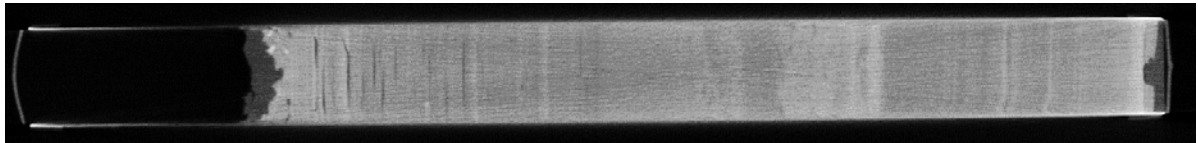
# 5 | Panel Observations

**FIGURE 5.3.1: CT SCAN/TILL/GLU**



TILL SAMPLE (MR14-104-SA8)

DEPTH: 11.4 TO 12.0 M / EL. 920.3 TO 919.7 M



UPPER GLU SAMPLE (MR14-106E-SA3)

DEPTH: 8.2 TO 8.8 M / EL. 920.5 TO 919.9 M

## 5 | Panel Observations

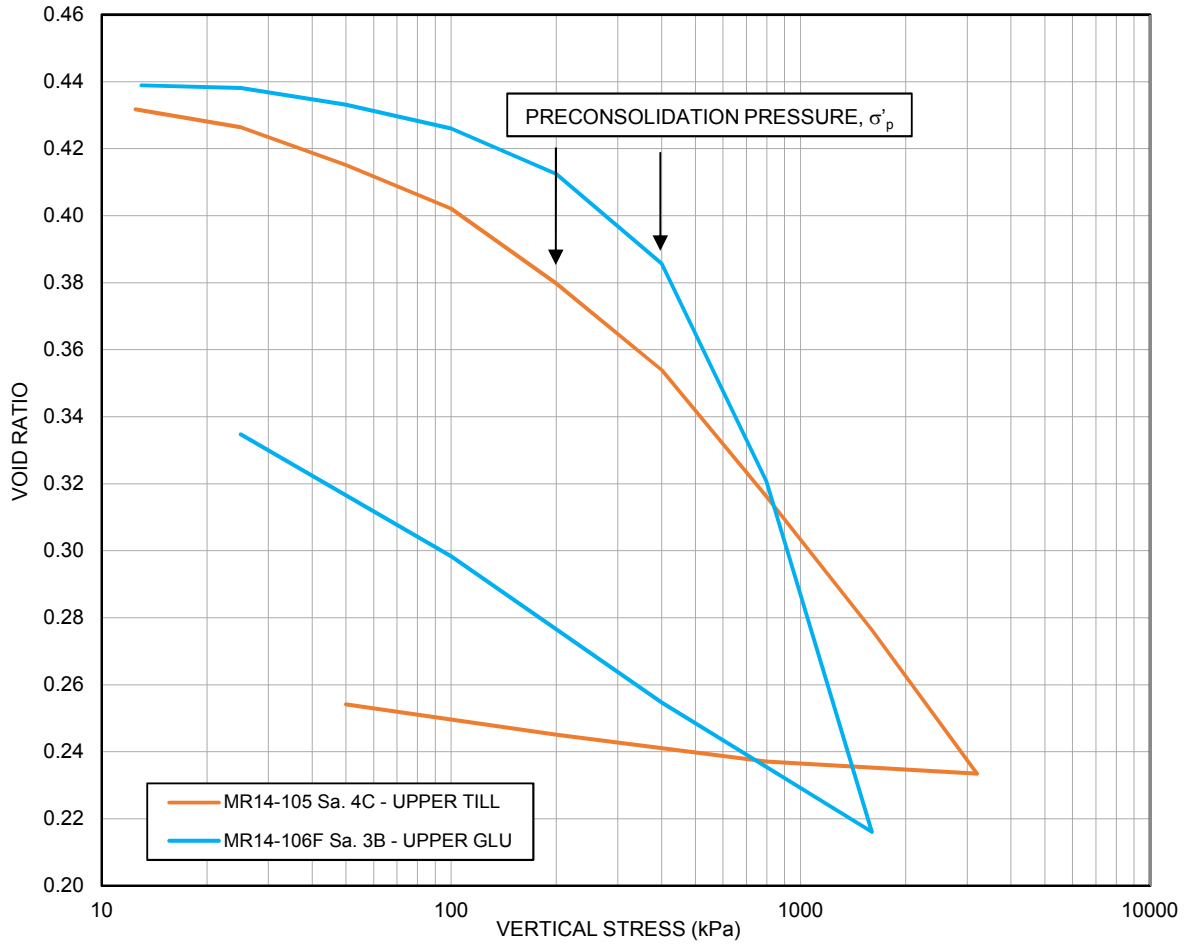
### 5.3.3 OEDOMETER TESTS

Oedometer tests are used to study the reduction in void ratio (porosity), with applied load simulating the construction of the embankment in stages. The change in curvature of the settlement response provides a base for estimating the preconsolidation pressure of the deposit, which is the maximum pressure experienced by the deposit in its geological past. The technique is illustrated in **Figure 5.3.2** for both a till specimen and a GLU specimen. Till is fundamentally less compressible than the GLU, and the technique to estimate preconsolidation stress has greater uncertainty. The data reveal that these deposits are not highly overconsolidated and that the pressure to be applied by the embankment will exceed the preconsolidation pressure, creating normally consolidated conditions. Normally consolidated conditions are conducive for the soil to behave in a contractive manner when subjected to both vertical pressure and shear.

**The data reveal that these deposits are not highly overconsolidated and that the pressure to be applied by the embankment will exceed the preconsolidation pressure, creating normally consolidated conditions.**

# 5 | Panel Observations

FIGURE 5.3.2: PRECONSOLIDATION STRESS EFFECT FOR BOTH TILL AND GLU

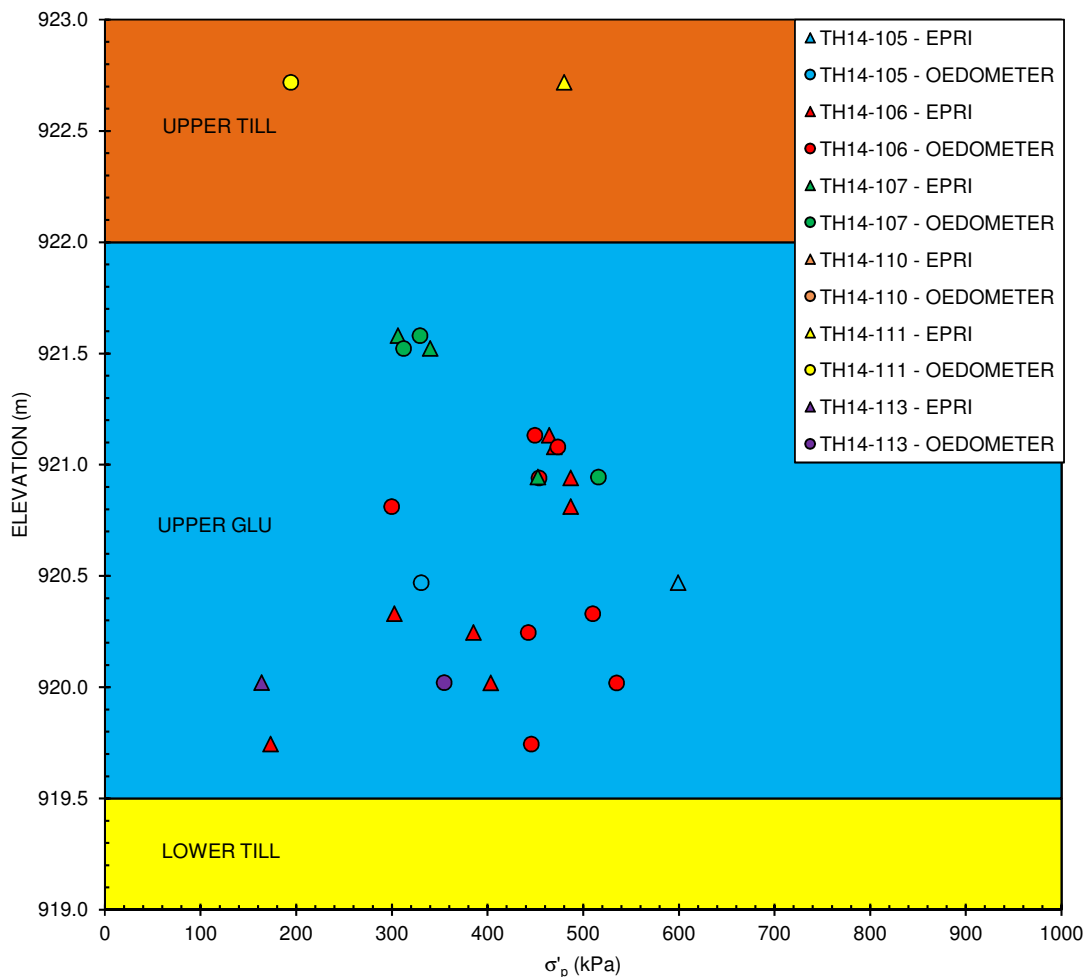


# 5 | Panel Observations

Preconsolidation pressure can also be inferred from the CPT testing conducted as part of the Panel's site investigation. Again, only modest preconsolidation stresses have been determined. A comparison between the results obtained from the field tests with those obtained from the oedometer tests is shown in **Figure 5.3.3**, and the agreement is acceptable. As the embankment was raised to a stress level beyond the preconsolidation stresses, the underlying GLU reverted to normally consolidated behaviour.

**As the embankment was raised to a stress level beyond the preconsolidation stresses, the underlying GLU reverted to normally consolidated behaviour.**

**FIGURE 5.3.3: PRECONSOLIDATION PRESSURE WITH ELEVATION**

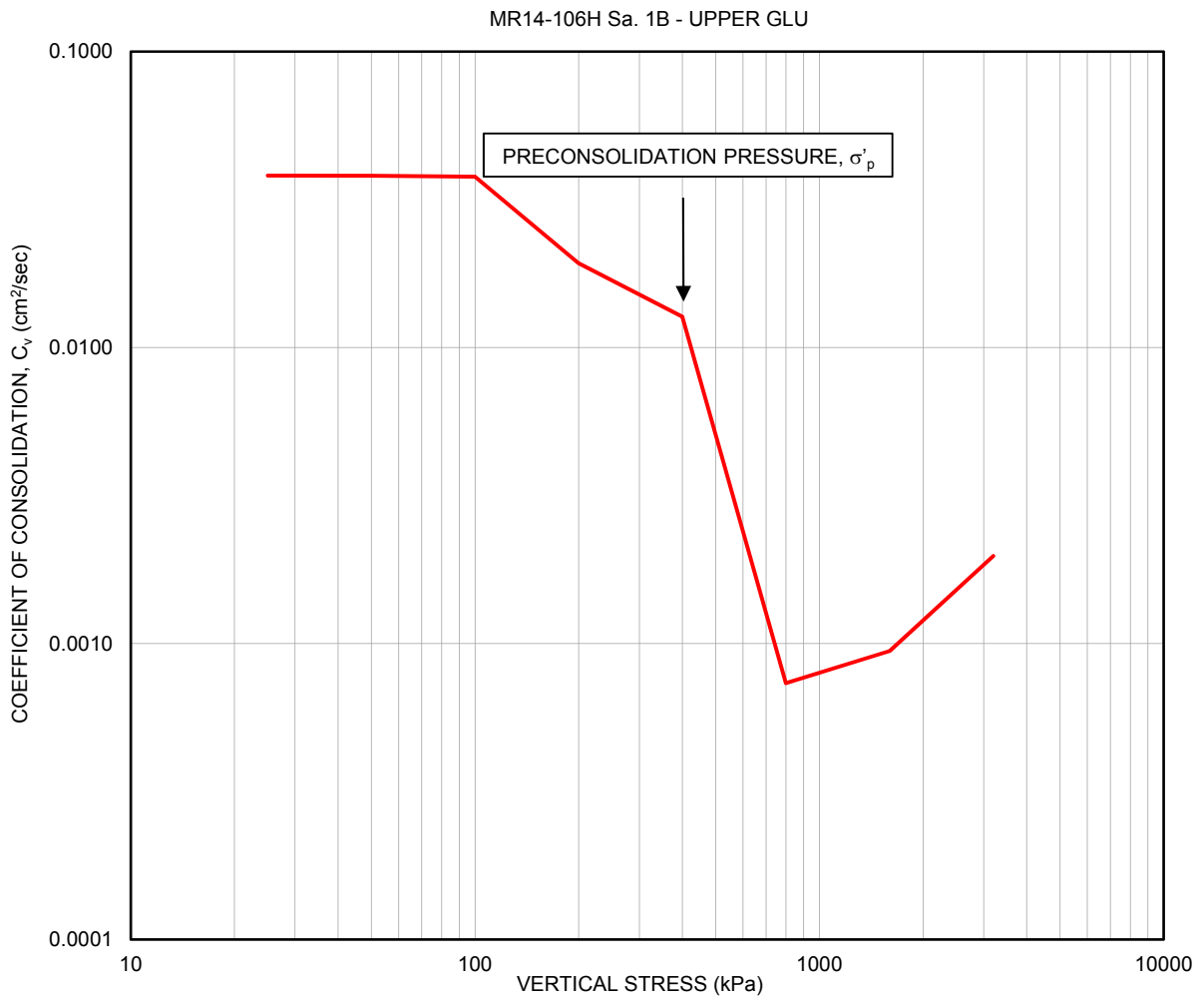




# 5 | Panel Observations

The data from oedometer tests are presented in Appendix E, including information on the coefficient of consolidation that reflects the rate of pore pressure dissipation on loading. **Figure 5.3.4** provides an example of this response. The significant reduction in this value in the GLU at pressures in excess of the preconsolidation stress is noteworthy.

**FIGURE 5.3.4: VARIATION OF COEFFICIENT OF CONSOLIDATION WITH APPLIED VERTICAL STRESS**



## 5 | Panel Observations

### 5.3.4 DIRECT SIMPLE SHEAR (DSS) TESTS

The subsurface characterization has inferred a sub-horizontal shear zone at about El. 920 m. The strength along this zone is best evaluated by DSS tests, which provide the ratio of undrained strength  $S_u$  to effective vertical consolidation stress  $\sigma_v'$ , or simply the undrained strength ratio. Tests on specimens from the GLU unit that reflect the shear zone at about El. 920–921 m are particularly relevant to the stability analyses that are discussed in section 6. Accordingly, the test program has been extensive, varying initial confining stress and initial shear stress. Testing with an initial shear stress (i.e., stress bias) is intended to explore the influence of a stage-constructed embankment that induces shear stresses in the ground prior to failure.

Another important feature exhibited by this test program on GLU specimens is a decline in resistance following its peak. This is called strain weakening. An example of a test exhibiting strain weakening is shown in **Figure 5.3.5**. As will be discussed, the presence of strain weakening contributes to understanding of the sudden nature of the breach mechanism. **Table 5.3.1** summarizes test characteristics and results from the DSS test program. Complete test results, with a brief description of test methodology, are presented in Appendix E.

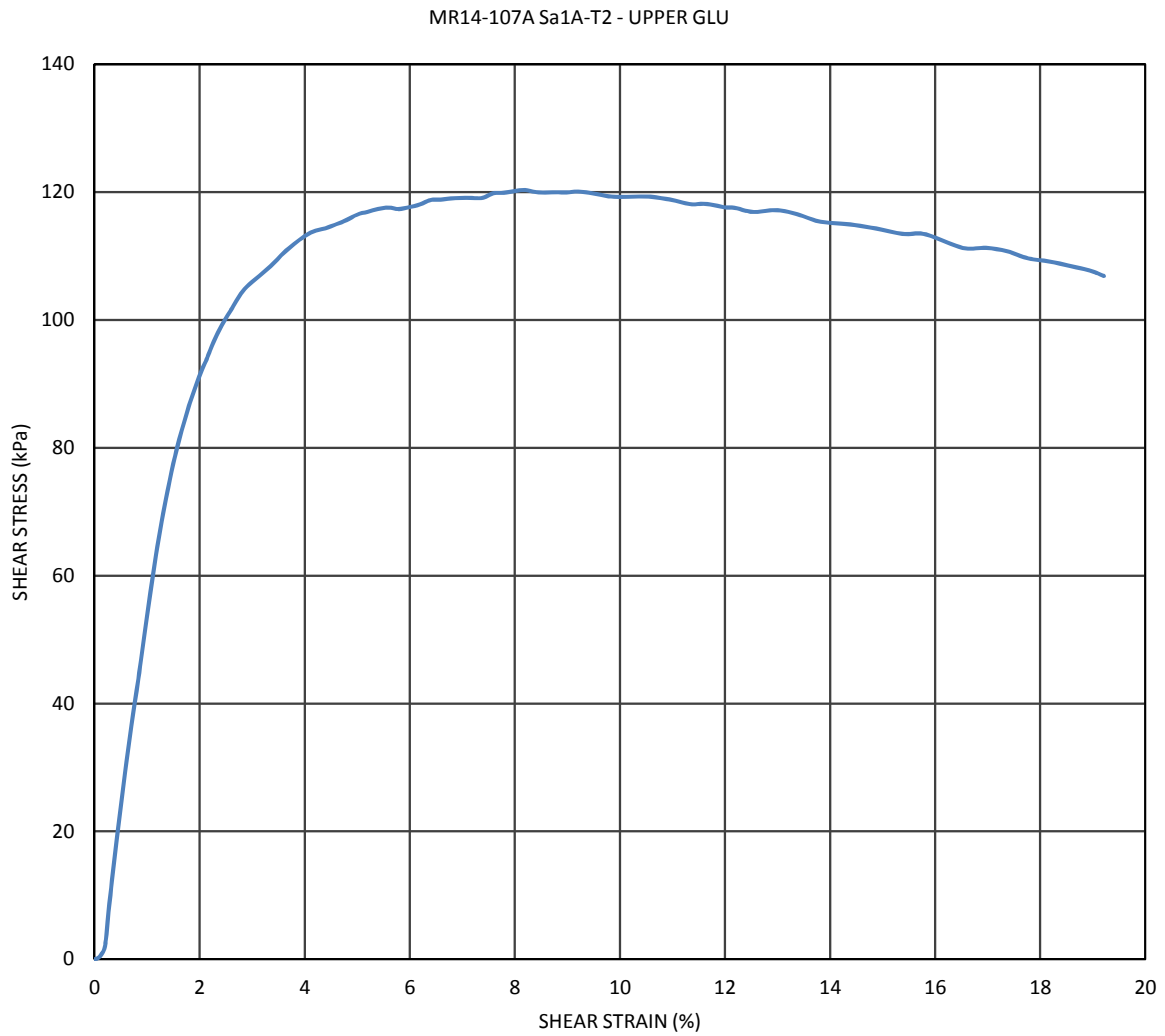
## 5 | Panel Observations

**TABLE 5.3.1: SUMMARY OF DIRECT SIMPLE SHEAR (DSS) TEST RESULTS**

SAMPLE	ELEVATION (m)	INFERRED PANEL SOIL UNIT	WATER CONTENT (%)	PLASTICITY INDEX	VERTICAL STRESS (kPa)	SHEAR BIAS	PEAK UNDRAINED STRENGTH RATIO
<b>14-106A Sa1C-T1</b>		UPPER GLU	43	31	600	0%	0.22
<b>14-106A Sa1C-T2</b>	921.1	UPPER GLU	37	31	600	10%	0.23
<b>14-106A Sa1C-T3</b>	921.1	UPPER GLU	38	31	600	20%	0.26
<b>14-106A Sa1C-T4</b>	921.1	UPPER GLU	33	31	300	20%	0.28
<b>14-106C Sa1B-T1</b>	921.2	UPPER GLU	44	33	600	30%	N/A
<b>14-106C Sa1B-T2</b>	921.2	UPPER GLU	43	33	600	25%	0.27
<b>14-106C Sa1B-T3</b>	921.2	UPPER GLU	39	33	600	10%	0.21
<b>14-106G SA2B-T1</b>	920.9	UPPER GLU	44	21	600	10%	0.21
<b>14-106G SaB-T2</b>	920.6	UPPER GLU	38	21	600	20%	0.26
<b>14-107 Sa6C-T1</b>	921.5	UPPER GLU	44	23	300	10%	0.27
<b>14-107 Sa6C-T2</b>	921.5	UPPER GLU	43	23	300	10%	0.25
<b>14-107A Sa1A-T1</b>	920.9	UPPER GLU	43	34	600	0%	0.21
<b>14-107A Sa1A-T2</b>	921.0	UPPER GLU	42	34	600	0%	0.20
<b>14-107A Sa7</b>	916.3	LOWER GLU	26	15	600	0%	0.30
<b>14-109 Sa6B</b>	916.6	LOWER GLU	22	13	300	10%	0.42
<b>14-110 Sa6C</b>	916.3	LOWER GLU	21	15	300	10%	0.27
<b>14-113 Sa4B</b>	922.9	UPPER TILL	13	7	300	10%	0.43
<b>Average Peak Undrained Strength Ratio in Upper GLU</b>					(no shear bias)	0.21	
					(10% shear bias)	0.23	
					( $\geq$ 20% shear bias)	0.27	
					overall	0.24	

# 5 | Panel Observations

FIGURE 5.3.5: DSS TEST: GLU WITH STRAIN WEAKENING



# 5 | Panel Observations

## 5.3.5 TRIAXIAL COMPRESSION TESTS

Triaxial compression tests with pore pressure measurements have also been conducted, in part for the record, and in part for use in stability analysis. The triaxial test data on till, together with the results from in situ pressuremeter tests, were used to inform the judgment of the Panel on an appropriate value to be used in the stability analyses.

Details of all triaxial tests are tabulated and presented in Appendix E.

## 5.3.6 DIRECT SHEAR TEST

While not used directly on any of its analyses, the Panel undertook a direct shear test on a pre-cut specimen of the GLU. This was primarily for the record, but afforded an opportunity for comparison with magnitudes adopted in some phases of the design. The Panel's measured residual strength of 16 degrees is at the lower end of the range used by others. The data are found in Appendix E.

## 5.3.7 DESIGN BASIS TESTING

The design of the Perimeter Embankment did not rely on any deep sampling of its foundation. Hence, no undisturbed samples were obtained, and no advanced laboratory tests were performed to provide data for purposes of comparison.

## 5.3.8 JOINT INVESTIGATION

Advanced laboratory studies were also performed on samples procured during the joint site investigation. The tests were not performed under the direction of the Panel, but are also included in a separate identifiable section within Appendix E.

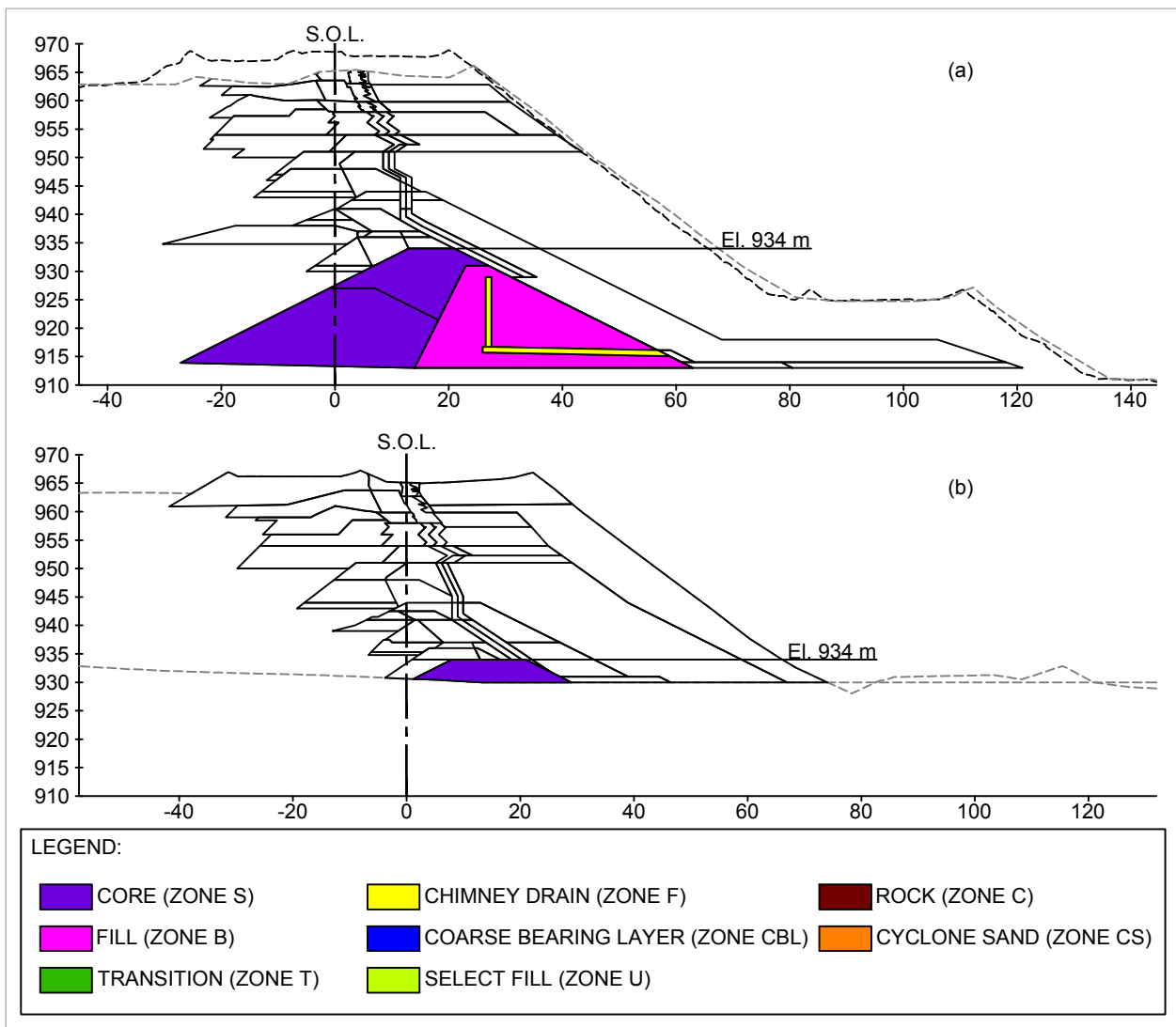
# 5 | Panel Observations

## 5.4 DESIGN, CONSTRUCTION AND OPERATION

This section describes the historical and sequential development of the Mount Polley Tailings Dam. The Main Embankment is included here along with the Perimeter Embankment to explain salient features and milestones related to design, construction and operation. The dam was developed in stages designated 1 through 9 that are treated in turn in the following discussion. At each stage, as-built cross-sections for the Main Embankment and for the Perimeter Embankment at the breach location are used to portray the dam's progressive expansion.

### 5.4.1 STAGE 1: 1997 — 1998

**FIGURE 5.4.1: STAGE 1 (a) MAIN EMBANKMENT (b) PERIMETER EMBANKMENT**



## 5 | Panel Observations

Design of the Main Embankment in May 1995<sup>7</sup> by KP established the direction for subsequent events. The overall plan incorporated dam raises to El. 960, of which Stage 1 would be the first, as illustrated in **Figure 5.4.1**. With planned raising by the “modified centreline” method,<sup>8</sup> the ultimate dam would rely on deposited tailings to provide structural support for the core. Fill would consist primarily of glacial till borrow soils, with sand tailings obtained by cycloning placed upstream of the core. The setting out line (S.O.L) provided the reference for dimensioning the dam’s fill zones and for stationing along its length.

Seismic criteria were based on a “low” consequence classification as defined by the Canadian Dam Association (CDA). The minimum factor of safety (FS) for the downstream dam slope was taken as 1.3 during impoundment operation and 1.5 at closure, design criteria that remained in effect for all subsequent raises. For the 2H:IV Stage 1 downstream slope, an effective-stress analysis (ESA) showed  $FS = 1.43$ , thereby satisfying the operational requirement.

Glaciolacustrine (GLU) fine sands, silts and clays were recognized from the outset to be present in the Main Embankment foundation. They were described as “typically dense to very dense and have been heavily overconsolidated by glaciers,” with two samples confirming that they consisted of “stiff, overconsolidated materials.”<sup>9</sup> In a crucial interpretation of their behaviour that would be relied upon throughout, a Ministry of Energy and Mines (MEM) query prompted KP to respond that “it is unlikely that any significant pore pressure development will occur in these materials during construction of the embankment.”<sup>10</sup>

Refinement of the Stage 1 design and its component Stages 1A and 1B continued as construction approached. With encouragement from MEM’s regulatory precursor, the Ministry of Employment and Investment, a narrow (1 m wide) chimney drain was added,<sup>11</sup> and four relief wells were installed in the foundation of the Main Embankment to reduce uplift pressures acting on GLU layers.<sup>12</sup>

In addition, the detailed Stage 1 design included a small dam only a few metres high to close off a topographic depression west of the Main Embankment. Designated the Perimeter Embankment and shown in **Figure 5.4.1(b)**, it would grow with subsequent stages to itself become a substantial structure contiguous with the Main Embankment. It would also host the site of the breach.

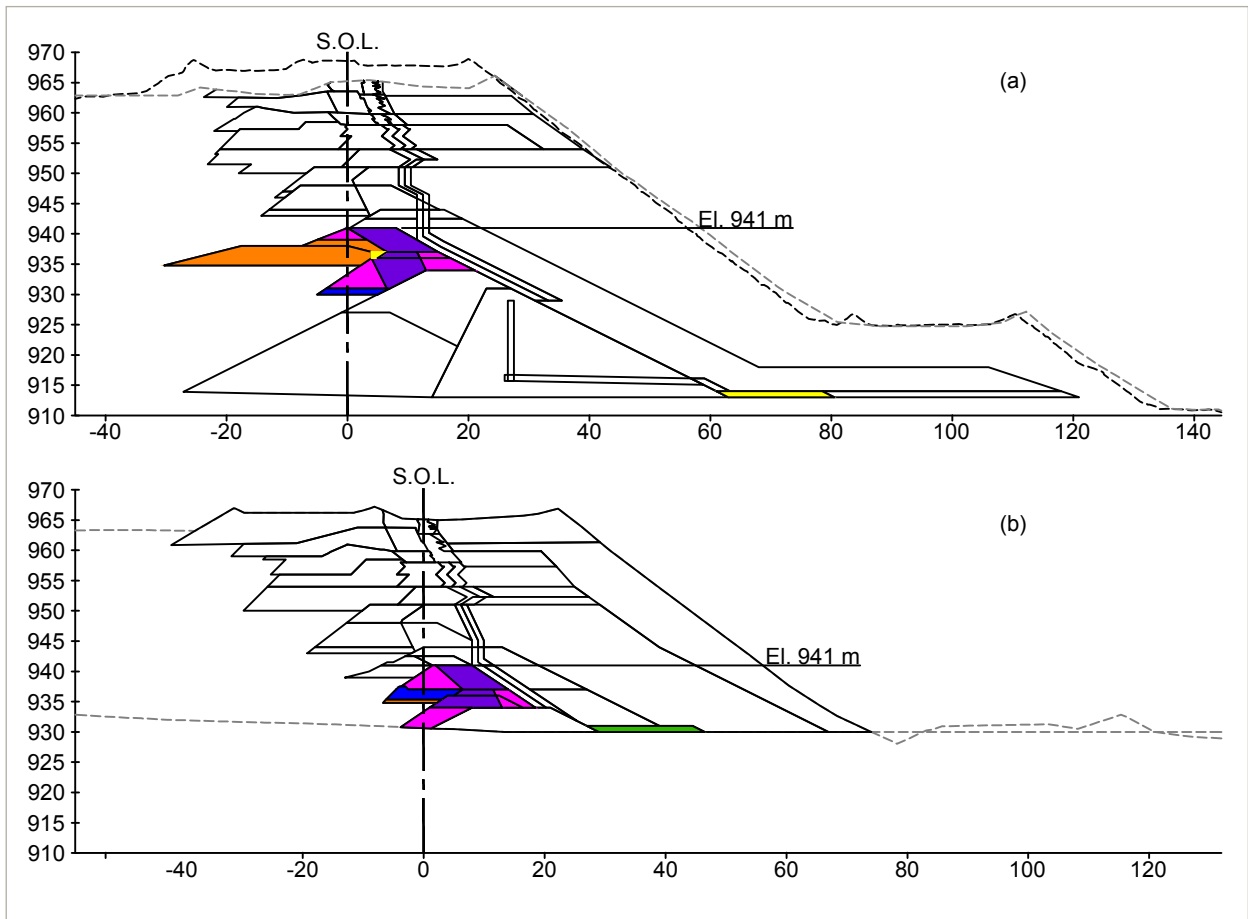
Construction of Stage 1 was completed in March 1997. Glacial till (Zone S and Zone B) was sourced from borrow excavations within the impoundment interior. The chimney drain materials (Zone F) were obtained by crushing, as would remain the case for subsequent raises. Both materials were subject to Construction Quality Assurance (CQA) testing. Vibrating wire piezometers were installed in both the embankment fill and foundation, with four of the six foundation instruments indicating elevated pressures. In response, a new operational stability criterion was established—an allowable ESA factor of safety of 1.1 at a trigger (action) level of 6 m of measured pressure head above the ground surface.<sup>13,14</sup>



# 5 | Panel Observations

## 5.4.2 STAGE 2: 1998 — 2000

**FIGURE 5.4.2: STAGE 2 (a) MAIN EMBANKMENT (b) PERIMETER EMBANKMENT**



## 5 | Panel Observations

Stage 2 was designed to be the first “modified centreline” raise. In the design, the core (Zone S) and chimney drain (Zone F) were extended upward, while adding a new zone of what was intended to be mine waste rock (Zone C) on the downstream slope and outward as a berm along the Main Embankment.<sup>15</sup> Another new feature for the Main Embankment was a longitudinal drain, designated the “upstream toe drain,” on the upstream side of the core near the crest of the raise. Its purpose was to allow drainage of the deposited tailings and reduce the embankment phreatic surface. An additional seven relief wells<sup>16</sup> and a relief trench were also included to reduce elevated foundation pore pressures. For the design configuration of the Main Embankment,  $FS = 1.67$  was computed for the downstream slope, exceeding the minimum required value of 1.3.<sup>17</sup>

The as-built configuration of the Stage 2 Main Embankment shown in **Figure 5.4.2(a)** differed from the design in several important respects. The intended Zone C mine waste fill was not added to the downstream slope, and the berm along the toe was not constructed. Rather than adhering to a “centreline” configuration, raise 2 utilized entirely “upstream” construction.<sup>18</sup> The same conditions prevailed for the Perimeter Embankment shown in **Figure 5.4.2(b)**. These as-built conditions were never reconciled with the Stage 2 stability analyses, which had been predicated on the original design configuration.

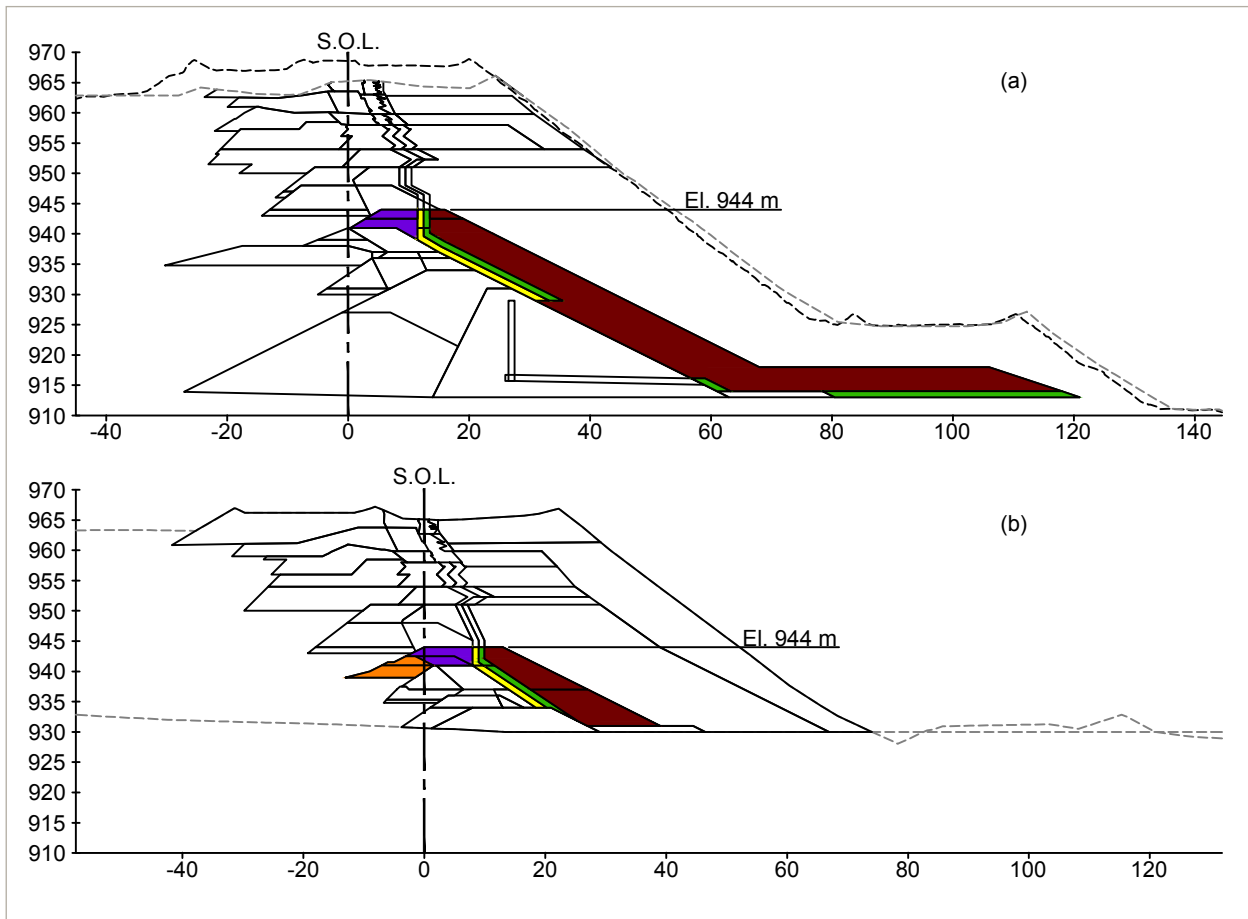
Operational trials and test fills established the feasibility of using cyclone sand underflow upstream of the core (Zone CS), and Stage 2 was the first to do so. A limited trial zone of cyclone sand would remain in the downstream shell of the Perimeter Embankment at design Section D, but cycloning would later be abandoned for both operational and economic reasons.<sup>19,20</sup>

In other operational matters, problems with the tailings pipeline system produced difficulties in maintaining the required tailings beach, with water directly contacting the embankments in some places.<sup>21</sup>

# 5 | Panel Observations

## 5.4.3 STAGE 3: 2000 — 2001

**FIGURE 5.4.3: STAGE 3 (a) MAIN EMBANKMENT (b) PERIMETER EMBANKMENT**



## 5 | Panel Observations

As Stage 2 was being constructed, efforts were underway to select materials for the upcoming Stage 3 and the remainder of the dam. A series of design studies in 1999 and 2000 developed a variety of configurations and options using cyclone sand<sup>22, 23</sup> as well as a rockfill alternative.<sup>24</sup> Various combinations were considered for the Main Embankment, the Perimeter Embankment, and the newly added South Embankment that would confine the third side of the impoundment beginning with Stage 3.

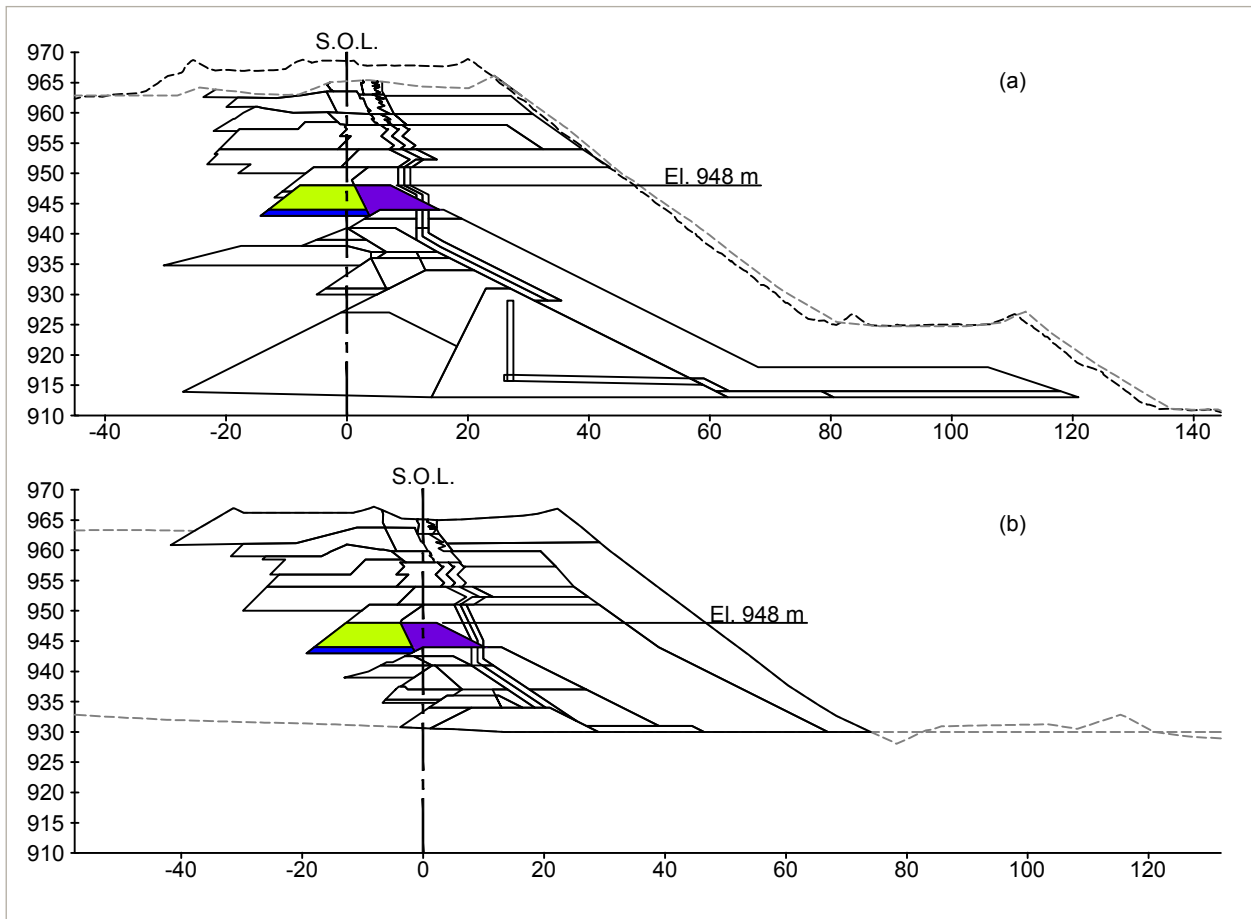
In May 2000, MPMC requested approval from MEM for a Stage 3 design using only cyclone sand for the Perimeter Embankment, with the Main Embankment raised using rockfill and the South Embankment with glacial till.<sup>25</sup> This was changed, however, in April 2001, when MPMC requested MEM approval for yet a different Stage 3 design using rockfill for the downstream Zone C in all three embankments. As shown by the as-built configuration in Figure 5.4.3, this plan was ultimately adopted for Stage 3 using rockfill sourced from a quarry.<sup>26</sup>

Despite the convoluted nature of the Stage 3 design process, an important milestone was that the Observational Method was formally invoked as the basis for design.<sup>27</sup> Thenceforward, each incremental raise was to be continually re-evaluated during operations, based on measured data from the piezometers and two inclinometers installed in July 2001. Putting this into effect, however, would have to wait. Not long thereafter, Mine operations were suspended for economic reasons on October 13, 2001.<sup>28</sup>

# 5 | Panel Observations

## 5.4.4 STAGE 4: 2005 — 2006

**FIGURE 5.4.4: STAGE 4 (a) MAIN EMBANKMENT (b) PERIMETER EMBANKMENT**



## 5 | Panel Observations

After a hiatus of over 3 years, Mine operations resumed in February 2005. Design of the Stage 4 raise called for a small cap on the Stage 3 crest extending over the tailings in an “upstream” configuration, together with Zone C rockfill on the downstream slope. Also included in the design was a rockfill buttress on the downstream slope of the Main Embankment to increase the factor of safety to 1.5 in anticipation of closure requirements.<sup>29</sup>

As illustrated in **Figure 5.4.4**, only the cap was constructed in Stage 4 without any additional rockfill on the downstream slope, resulting in another “upstream”-type raise.<sup>30</sup> In constructing this raise, trial programs pioneered the use of hydraulic-cell deposition of tailings for the upstream Zone U, a practice that continued throughout construction.

Separately, operational problems in maintaining the required tailings beach continued, with water directly against the embankment in several areas.<sup>31</sup>

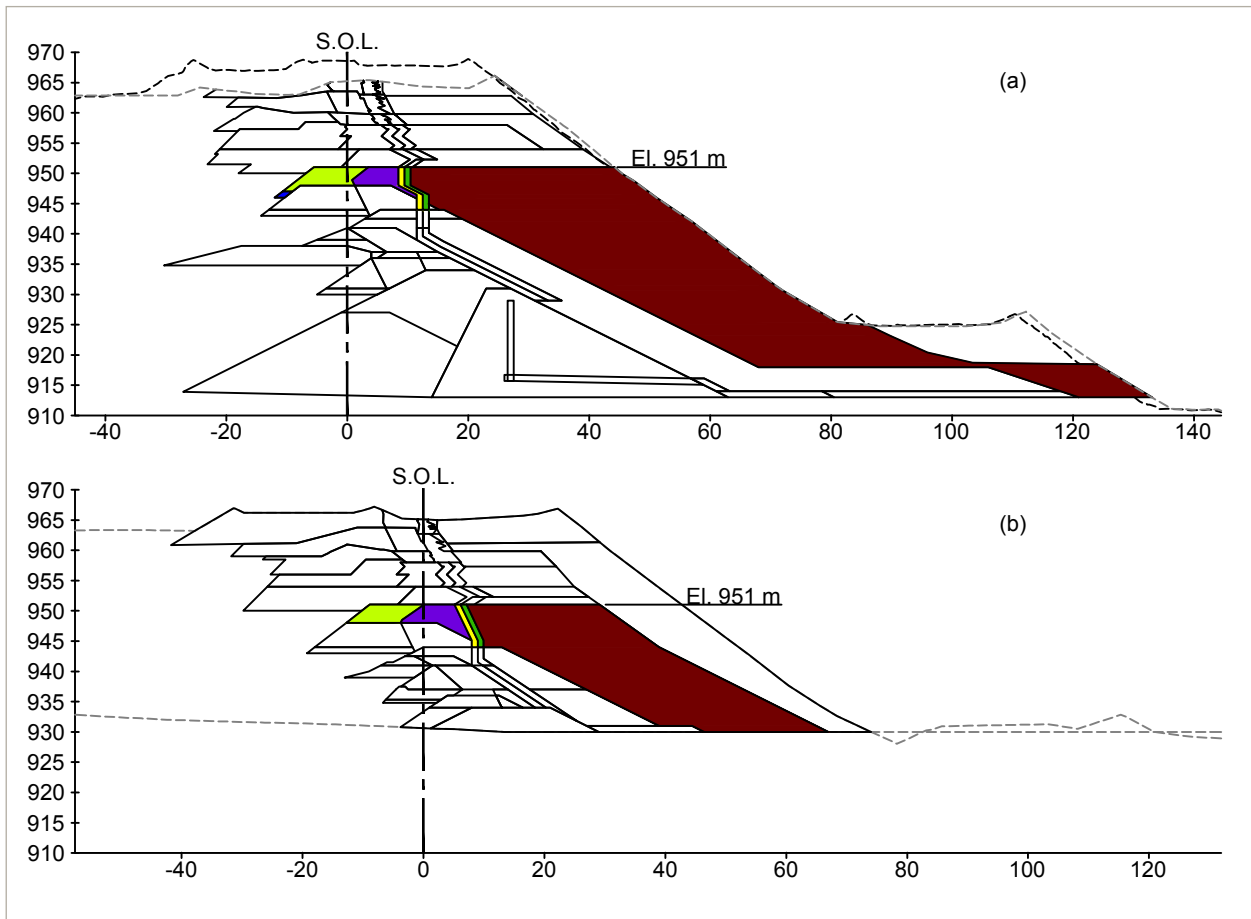
Renewed operation brought renewed queries from MEM concerning the glaciolacustrine foundation materials. One concerned the characteristics and effects on dam stability of softer GLU deposits at groundwater well GW96-1A downstream from the Perimeter Embankment.<sup>32</sup> In response, KP cited borrow area test pits and auger borings as confirming that “the glaciolacustrine deposit encountered in GW96-1A is a discontinuous unit and will not adversely affect the dam stability.”<sup>33</sup>

The breach subsequently occurred 300 m due west of GW96-1A.

# 5 | Panel Observations

## 5.4.5 STAGE 5: 2006 — 2007

**FIGURE 5.4.5: STAGE 5 (a) MAIN EMBANKMENT (b) PERIMETER EMBANKMENT**



## 5 | Panel Observations

The Stage 5 design once again incorporated the downstream Zone C rockfill that had been deferred in previous stages, and this time it was built. But since the material would now be sourced from mine waste rather than quarried, mine production and delivery had to be accommodated.<sup>34</sup> Due to related restrictions, it was planned to place the Zone C outslope to an “interim” 1.4H:1V inclination—rather than the design basis 2.0H:1V—as a temporary expedient until mine waste delivery could catch up with construction. The steeper slope would be expanded and flattened to 2.0H:1V “once the embankments have reached the Stage 5 design elevation.”<sup>35</sup> An ESA factor of safety of 1.5 was reported for the steeper interim slopes of the Main Embankment and 1.9 for the Perimeter Embankment.

**It was planned to place the Zone C outslope to an “interim” 1.4H:1V inclination—rather than the design basis 2.0H:1V—as a temporary expedient until mine waste delivery could catch up with construction.**

Stage 5 construction proceeded from Stage 4 in a continuous, uninterrupted campaign and was completed in November 2007. But instead of rectifying the interim steep slopes at this time as had been intended, such measures were left to future stages of embankment raising.<sup>36</sup>

Stage 5 saw the first substantial enlargement of the Perimeter Embankment, with widening of the crest and expansion of the downstream Zone C rockfill as shown in **Figure 5.4.5(b)**. At the same time, an upstream toe drain was added to complement the companion drain already installed at the Main Embankment.

Operationally, chronic problems with maintaining the tailings beach continued, with procurement of enough tailings pipe to traverse the entire embankment perimeter now the anticipated solution.<sup>37</sup>

The year 2006 marked the 10-year interval for the mandatory third-party Dam Safety Review (DSR), which was prepared by AMEC.<sup>38</sup> The most salient aspects of this report concern its assessment of foundation strength and related dam stability. Shear failure of the dam slope, including failure through the foundation, was first on a list of potential failure modes applicable to the Mount Polley dam in relation to “excessive loading at or near the crest or a weakness in the foundation.” Noting the apparent overconsolidation of the glaciolacustrine materials, the report identified two conditions of particular interest: the possible presence of pre-sheared planes of weakness, and the potential for “brittle” response involving strength loss at small strains. The DSR contained no mention of the behaviour of foundation materials in undrained shear.

**The Dam Safety Review contained no mention of the behaviour of foundation materials in undrained shear.**



## 5 | Panel Observations

The DSR also remarked on the lack of a tailings placement strategy that had impeded systematic development of a tailings beach for so long, calling lack of such a beach a “deficiency” and noting that the dam had not been designed as a water dam.

Shortly after the DSR was submitted, at MPMC’s request, AMEC produced a follow-up report that reviewed several possible optimization measures for the TSF.<sup>39</sup> One measure was to reduce the width of the core to as little as 3 m to 4 m. Another was to eliminate the uppermost 1 m of the dam core, since this part of the crest “only provides freeboard.”

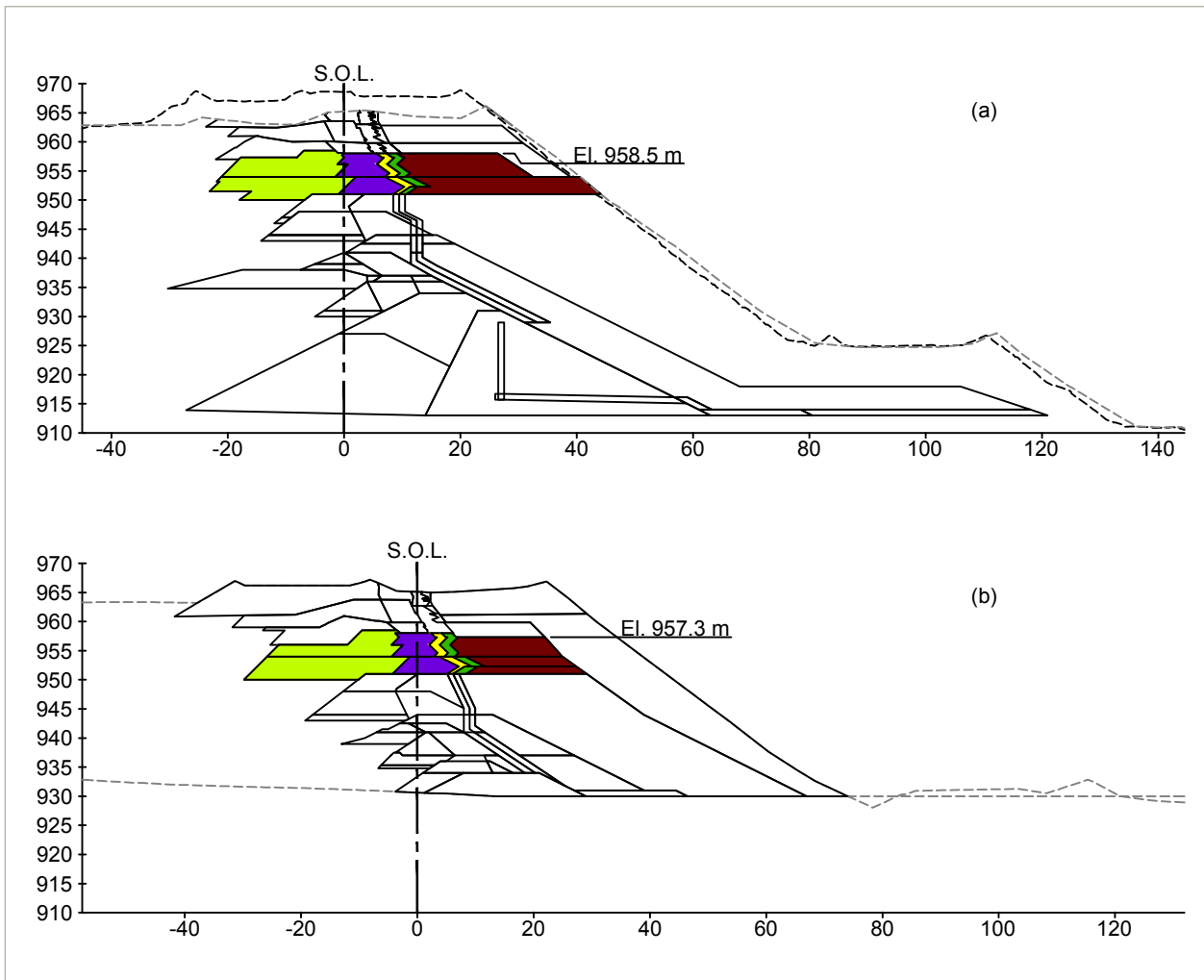
The optimization report also questioned the need for the Main Embankment buttress first proposed for Stage 4 and partially constructed for Stage 5. It concluded that foundation strengths used previously would result in adequate stability without a buttress. The only proviso was the potential for pre-sheared planes of weakness in the foundation, a question that remained outstanding from the DSR.

With the water balance “fine tuned to an accuracy that is in the range of centimeters” in terms of impoundment water elevation, the report proposed that the wave runup allowance, and therefore freeboard requirements, could be reduced. Remarking on beach development, it further stated that unless water was deep enough to affect stability of the Zone U tailings, there was “no rush” in developing a beach along the Main Embankment to correct the deficiency identified in the DSR.

# 5 | Panel Observations

## 5.4.6 STAGE 6: 2007 — 2011

**FIGURE 5.4.6: STAGE 6 (a) MAIN EMBANKMENT (b) PERIMETER EMBANKMENT**



## 5 | Panel Observations

The Stage 6 design for the Main Embankment incorporated two components: an additional 7 m of fill on the crest, and a Zone C rockfill buttress at the downstream toe. The Zone S core was reduced from its former 8 m width to 5 m on the basis of the effectiveness of the upstream toe drains in lowering the phreatic surface and gradients within the core.<sup>40</sup> Stage 6 also introduced the practice of raising the Zone S core, the thin Zone F filter, and the equally thin Zone T transition in an intricate zigzag configuration.

The Main Embankment buttress, first included in the Stage 4 design but never fully constructed, was an outgrowth of two factors. First was the effect on stability of the “interim” 1.4H:1V slopes that had persisted since Stage 5. Second were the foundation strength interpretations put forward in the DSR. Stage 6 stability analyses adopted an estimated residual strength of 24° for the GLU foundation materials at the Main Embankment to account for the possible presence of pre-shearing. The resulting buttress produced an ESA factor of safety of 1.4, satisfying the FS = 1.3 design requirement for operation.

The Stage 6 design sought to accommodate the limited mine waste delivery rates experienced in Stage 5—and the consequent slope oversteepening—by extending construction over a 2-year period. Even so, the calculated FS = 1.4 for the Stage 6 Main Embankment indicated that the buttress would need to continue being raised in future dam stages, requiring more material. To make matters worse, KP noted that only non-reactive mine waste could be used, further constraining available quantities and confirming buttress construction as a continuing proposition. But once again, the Stage 6 buttress was not constructed as designed, turning out to be about 5 m below its design height and short of its design extent.<sup>41</sup>

**But once again, the Stage 6 buttress was not constructed as designed, turning out to be about 5 m below its design height and short of its design extent.**

None of these buttressing considerations pertained to the Perimeter Embankment. Residual strength parameters were not applied to its foundation, and the resulting factor of safety of 1.7 required no enhancement according to the FS = 1.3 criterion.

Elsewhere, beach deposition from the extended tailings discharge line had not been successful in preventing water accumulation against the Main Embankment. This increased flows in the same upstream toe drain whose effectiveness had been cited as justification for reducing the width of the Stage 6 core.<sup>42</sup>

## 5 | Panel Observations

Meanwhile, follow-up related to the DSR continued. MEM requested that KP provide the results of its recommended direct shear testing, which essentially confirmed the Stage 6 design ESA factor of safety.<sup>43, 44</sup> But beyond this was another item in KP's response that marked a milestone in two fundamental respects. For the first and only time during the design process, an undrained strength analysis (USA) was performed. This was also the only instance that the foundation clay behaviour would be taken as other than that of stiff and highly overconsolidated material. Using a typical  $S_u/\sigma'_v$  of 0.25 for soft, normally consolidated clays, KP found a USA factor of safety of 1.1 for the Stage 6 configuration. Not recognizing that this strength might indeed be the operational strength under static loading conditions, KP concluded that "there is also sufficient undrained strength in the lacustrine unit for the embankment to remain stable." This conclusion would henceforth never be called into question.

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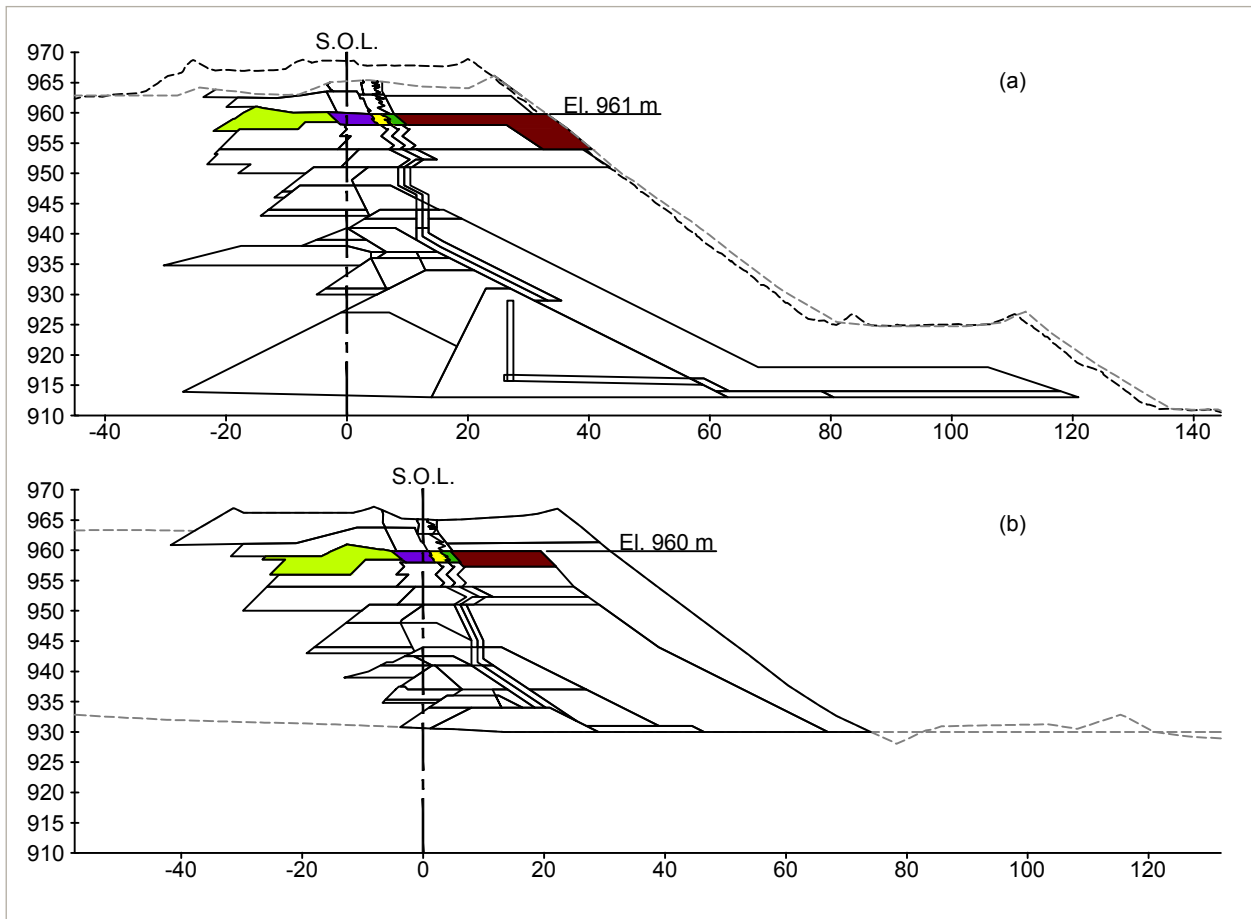
Operation of Stage 6 throughout 2009 and 2010 highlighted other matters. In 2009, movements in the GLU recorded at Inclinometer SI01-02 resulted in expanding the Main Embankment buttress in the immediate area. This proved to be effective in arresting further displacements.<sup>45</sup> By 2010, the buttress had been extended along the west side of the Main Embankment, but still remained to be completed along its entire length.<sup>46</sup> In another development, a tension crack appeared at the downstream edge of Zone C at Sta. 3+400 of the Perimeter Embankment. Although interpreted to be an artifact of near-surface movement, a follow-up stability assessment was nonetheless recommended.

Inadequate tailings beach development along the Main and South Embankments was flagged yet again, this time in an MEM inspection. Noting that an above-water beach was a requirement of the design, the inspector considered its absence at the southeast corner of the Main Embankment to be a "Departure from Approval" and ordered that a beach be "re-established as soon as possible in this area to meet the design objectives."<sup>47</sup>

# 5 | Panel Observations

## 5.4.7 STAGE 7: 2011 — 2012

**FIGURE 5.4.7: STAGE 7 (a) MAIN EMBANKMENT (b) PERIMETER EMBANKMENT**



## 5 | Panel Observations

In 2011, the Engineer of Record (EOR) responsibilities were transferred from KP to AMEC, and with them the design of Stage 7 for a height increase of 2.5 m. No new Zone C fill would be added to flatten the downstream slope, and no buttress expansion would be conducted. Continuing the stability analysis protocols from Stage 6 and the 2006 DSR, AMEC found that the ESA factor of safety using residual strength for the foundation GLU was unchanged from the Stage 6 value of 1.4<sup>48</sup> for the Main Embankment. Similar conclusions applied to the factor of safety for the Perimeter Embankment.

The same year also saw the completion of the 2011 Geotechnical Site Investigation, the first major foundation exploration program since Stage 1. It consisted of 11 sonic drillholes, with piezometers installed in each, plus three new inclinometers.<sup>49</sup> Emphasis was on definitively evaluating the DSR hypothesis that the glaciolacustrine foundation soils might contain pre-sheared planes of weakness and the operative residual strengths that would accompany them.

Careful inspection of recovered core revealed no indications of slickenside features and no evidence of pre-shearing. Thus, residual strengths need no longer be considered. Neither, it was concluded, did these conditions indicate that the 2010 crack in the Perimeter Embankment was attributable to weak soil conditions in the area.

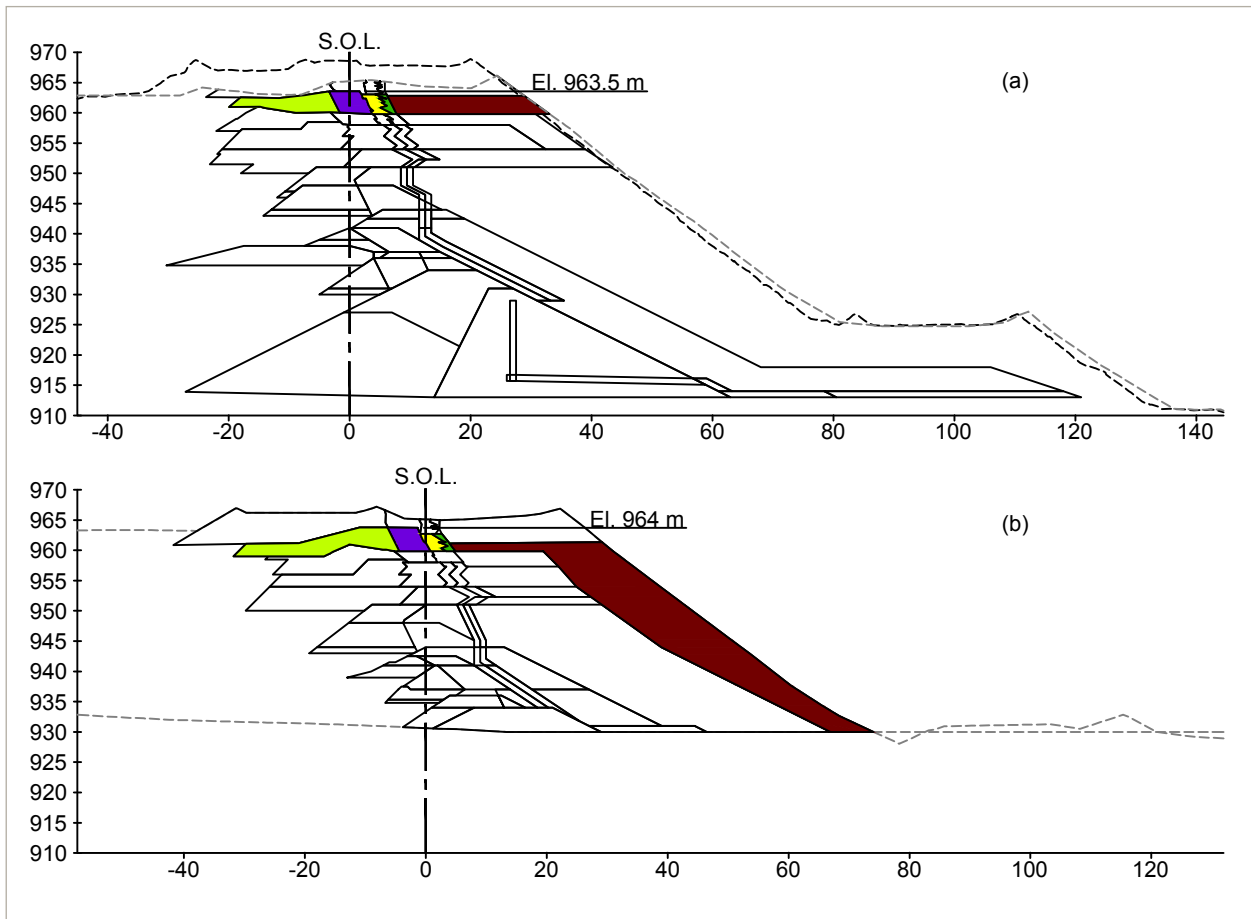
With respect to stress history, the 2011 report further concluded that the GLU was overconsolidated, consistent with previous interpretations. The softer conditions in monitor well GW96-1A adjacent to the Perimeter Embankment that MEM had questioned in 2005 were said to be “not of significant concern in this instance as the drillhole location is approximately 140 m further downstream from the current toe of the dam.” In fact, the report said, “based upon available information, foundation conditions along the Perimeter Embankment appear more favourable than those along the Main Embankment” in terms of the presence and extent of clay-rich zones within the GLU.

At the Main Embankment, piezometers were installed generally beneath the buttress where measured pore pressures would be reflective of additional fill. This was not the case at the Perimeter Embankment, where all of the new piezometers were located 15–20 m from the downstream toe. Similarly, Inclinometer SI11-04 installed during the program was 15 m away.

# 5 | Panel Observations

## 5.4.8 STAGE 8: 2012 — 2013

**FIGURE 5.4.8: STAGE 8 (a) MAIN EMBANKMENT (b) PERIMETER EMBANKMENT**



## 5 | Panel Observations

Stage 8 was initially designed as a 3.5 m raise, then increased to 5 m with an accelerated construction program. It would change to conventional “centreline” raising from the previous “modified centreline” that had progressively shifted the raises upstream. Stage 8 fill was added only to the crest of Stage 7 Main Embankment,<sup>50, 51</sup> while the Perimeter Embankment was widened as well. But in both cases, flattening of the Stage 5 “interim” oversteepened slope to 2H:1V was deferred yet again—not until completion of Stage 5 raise as first proposed, but this time until completion of the entire dam.

**Flattening of the Stage 5 “interim” oversteepened slope to 2H:1V was deferred yet again—not until completion of Stage 5 raise as first proposed, but this time until completion of the entire dam.**

In evaluating the stability of the steepened slope, AMEC returned to the peak-strength interpretation for the GLU materials based on the findings of its 2011 field program. For a peak effective-stress friction angle of 28°, the ESA factor of safety was found to be a barely adequate 1.31<sup>52</sup> for the Main Embankment.

The 2011 investigation showed the GLU materials at Section D of the Perimeter Embankment to be deeper than at the Main Embankment. In stability analyses at Section D near the breach, the critical failure surface did not reach the GLU and remained within the overlying foundation till, producing a much higher factor of safety of 1.77.

The larger issue of what minimum factor of safety should be required was addressed in a September 19, 2012 communication from MEM to MPMC that deserves to be quoted at length:

*The factor of safety for the main embankment is only marginally above the short-term design criteria of 1.3... AMEC has interpreted Table 6-2 from the 2007 Dam Safety Guidelines somewhat differently than I have seen in the past. This table recommends a minimum factor of safety of 1.3 at the end of construction and ‘before reservoir filling’ and a factor of safety of 1.5 at the ‘normal reservoir level.’ AMEC has interpreted the construction period as the entire pre-closure period, and this is open to debate. However, I consider that sufficient mitigation measures are in place (i.e., piezometer trigger thresholds) to support this more liberal interpretation in this instance.*<sup>53</sup>

Although questioning AMEC’s interpretation of the Dam Safety Guidelines, MEM was prepared to accept FS = 1.3, but only in conjunction with the Observational Method.

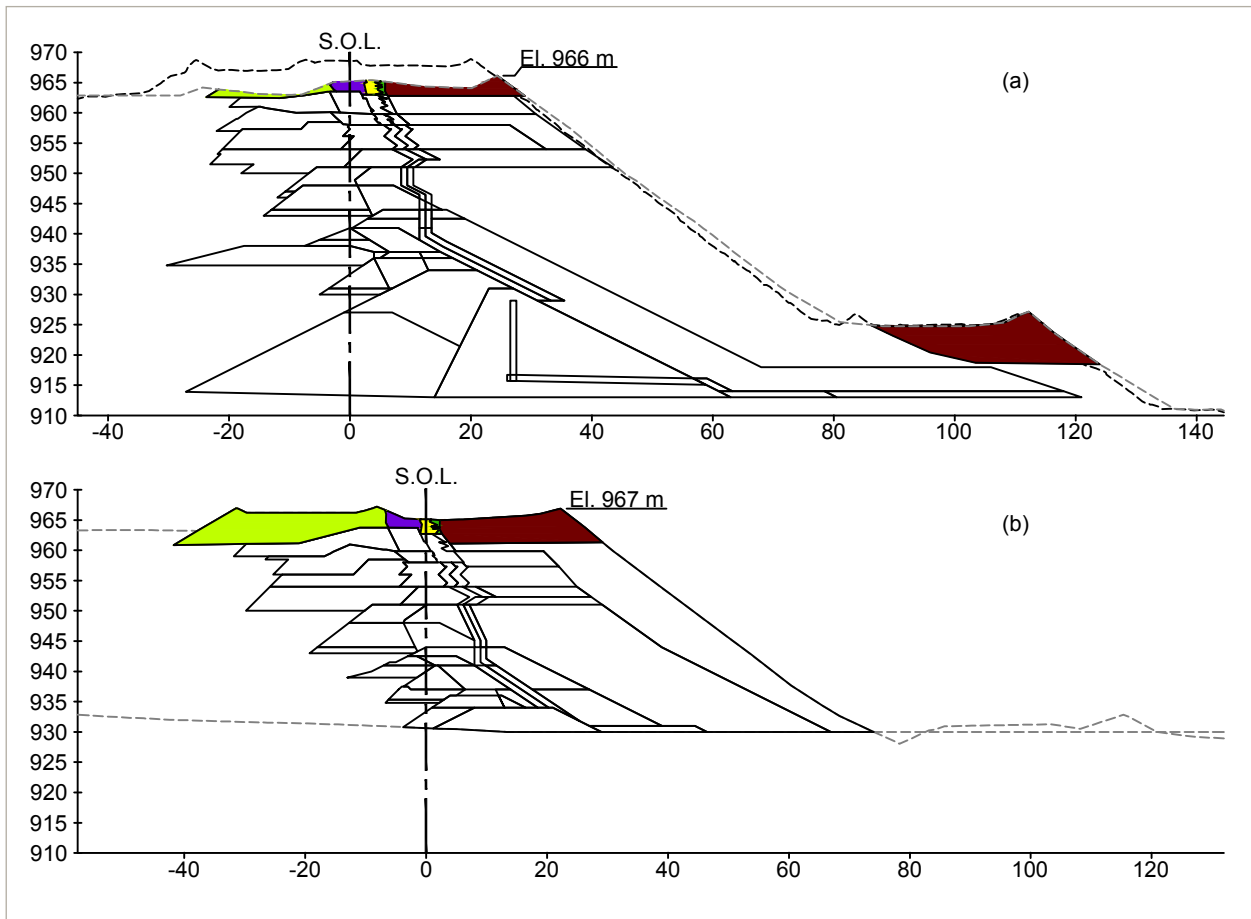
In other matters, the recurring problem of tailings beach development was not directly addressed in the 2012 inspection report, but an airphoto showed no tailings beach over approximately 40% of the impoundment perimeter.<sup>54</sup> The report also noted that seepage had been present at the toe of the Perimeter Embankment near the breach section and that it had moved from previous years. Based on interviews with MPMC personnel, the Panel believes that the likely source of the apparent seepage was actually a buried outlet of the upstream toe drain.<sup>55, 56</sup>



# 5 | Panel Observations

## 5.4.9 STAGE 9: 2013 — 2014

**FIGURE 5.4.9: STAGE 9 (a) MAIN EMBANKMENT (b) PERIMETER EMBANKMENT**



## 5 | Panel Observations

Stage 9, whose construction was being completed when the breach occurred, encompassed a period of intense activity with a number of seminal events in the months, weeks and days preceding the failure. While AMEC remained the EOR until the planned completion of Stage 9 to El. 970, BGC would officially become the EOR beginning with construction of the planned Raise 10. Consequently, 2013 to 2014 was also a period of transition, with overlap in activities, if not responsibilities.

AMEC's April 11, 2013 design for Stage 9 planned a substantial 6.5 m height increase by adding fill to the crest of Stage 8. Retaining the peak-strength interpretation for the GLU foundation materials, AMEC found that raising the Main Embankment buttress to El. 925 m would be needed to nominally achieve a minimum ESA factor of safety of 1.3.<sup>57</sup>

Commenting on the implications of this value, MEM's remarks on July 29, 2013, echoed its previous concerns:

*The stability analyses indicate that the FOS for the 'Main Embankment' only marginally achieves the short term CDA design criteria of 1.3. ... Previous correspondence from MEM has highlighted the difference in interpretation of the CDA Guidelines. AMEC has considered the construction period to be the entire 'pre-closure' period while CDA Guidelines, Table 6-2 recommends a minimum FOS of 1.3 'before reservoir filling,' and a FOS of 1.5 at the 'normal reservoir level.'*

*MEM requires a commitment from Mount Polley that they are moving toward increasing these FOS for the main embankment as part of subsequent dam raises in an effort to move toward achieving a long term FOS equal to 1.5. It is expected that Mount Polley will continue their transition to centerline construction and provide additional buttressing with time.*<sup>58</sup>

This marked a major change in direction. A factor of safety of 1.5, not 1.3, would become the governing criterion. Moreover, buttressing could no longer be deferred for either embankment. A factor of safety of 1.58 had been calculated for Section D of the Perimeter Embankment, once more unaffected by the GLU foundation materials. But even this value was approaching the new minimum of 1.5 that MEM was now aiming to enforce, and buttress preparation needed to begin.

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By the end of the 2013 construction season, pre-stripping for a buttress around the Perimeter Embankment had been completed, including the area of the breach section.<sup>59</sup> In a Panel interview, the contractor who performed the work stated that portions of this area remained open at the time of the breach,<sup>60</sup> an assessment confirmed by MPMC.<sup>61, 62</sup>

## 5 | Panel Observations

Meanwhile, attention was turning to longer-term prospects for continued dam raising, and the outlook was not good. BGC made explicit the connection between the structural limitations of the dam and the ever-growing volumes of surplus water it was being called upon to contain. In a June 18, 2013 memorandum, it stated:

*A continuous beach along the complete upstream length of the dam is the design requirement necessary for dam stability and needs to be achieved moving forward regardless of the final targeted crest elevation. The current water pond surplus does not allow for the development/maintenance of above-water beaches.*<sup>63</sup>

It elaborated on this topic a month later, on July 25, 2013:

*An above-water tailings beach separating the till core from the reclaim water pond constitutes a fundamental design element of the dam. Without a wide above-water beach, the MPMC tailings dam is effectively being operated as a water-retaining dam, with the water pond effectively in direct contact with the till core, separated by only a narrow zone of tailings or waste rock.*<sup>64</sup>

During the ensuing months, this chronic water-surplus problem would become acute. For years, dam raising had managed to stay one step ahead of the rising water. But on May 24, 2014, the water caught up. With Stage 9 nearing completion, what was described as “seepage flow” was observed over the dam core.<sup>65</sup> Intensive surveillance and construction activity over the following days and weeks succeeded in raising low areas around the embankment perimeter, restoring containment integrity, and saving the dam from overtopping failure.

**For years, dam raising had managed to stay one step ahead of the rising water. But on May 24, 2014, the water caught up.**

As the gravity of the water problem was becoming apparent, so was the consequent necessity of dam raising beyond Stage 9. MPMC required some estimate of future dam footprint so that prerequisite stripping of additional areas could commence immediately. BGC responded on October 22, 2013, with a memorandum that outlined an approach to dam raising that resurrected the residual-strength interpretation for GLU, while at the same time establishing new factor of safety criteria conforming to MEM’s 2013 directive.<sup>66</sup>

This approach was formalized in BGC’s design report for Stage 10 issued on July 25, 2014, just eight days before the breach.<sup>67</sup> The proposed raise would achieve a minimum  $FS = 1.5$  for the Main Embankment using peak effective-stress strength for the GLU and full dissipation of load-induced pore pressures. But this new design philosophy would go one step further.

## 5 | Panel Observations

Notwithstanding AMEC's 2011 subsurface investigation, a "more conservative" approach would be taken by allowing for the possibility of brittle behaviour or pre-shearing in the GLU. This would apply an additional criterion of  $FS = 1.1$  using residual strength in the GLU for what was characterized as a "reasonable worst-case scenario." So the residual-strength interpretation was now reintroduced after first being suggested in the 2006 DSR, adopted in design of Stages 6 and 7, then abandoned in design of Stage 8.

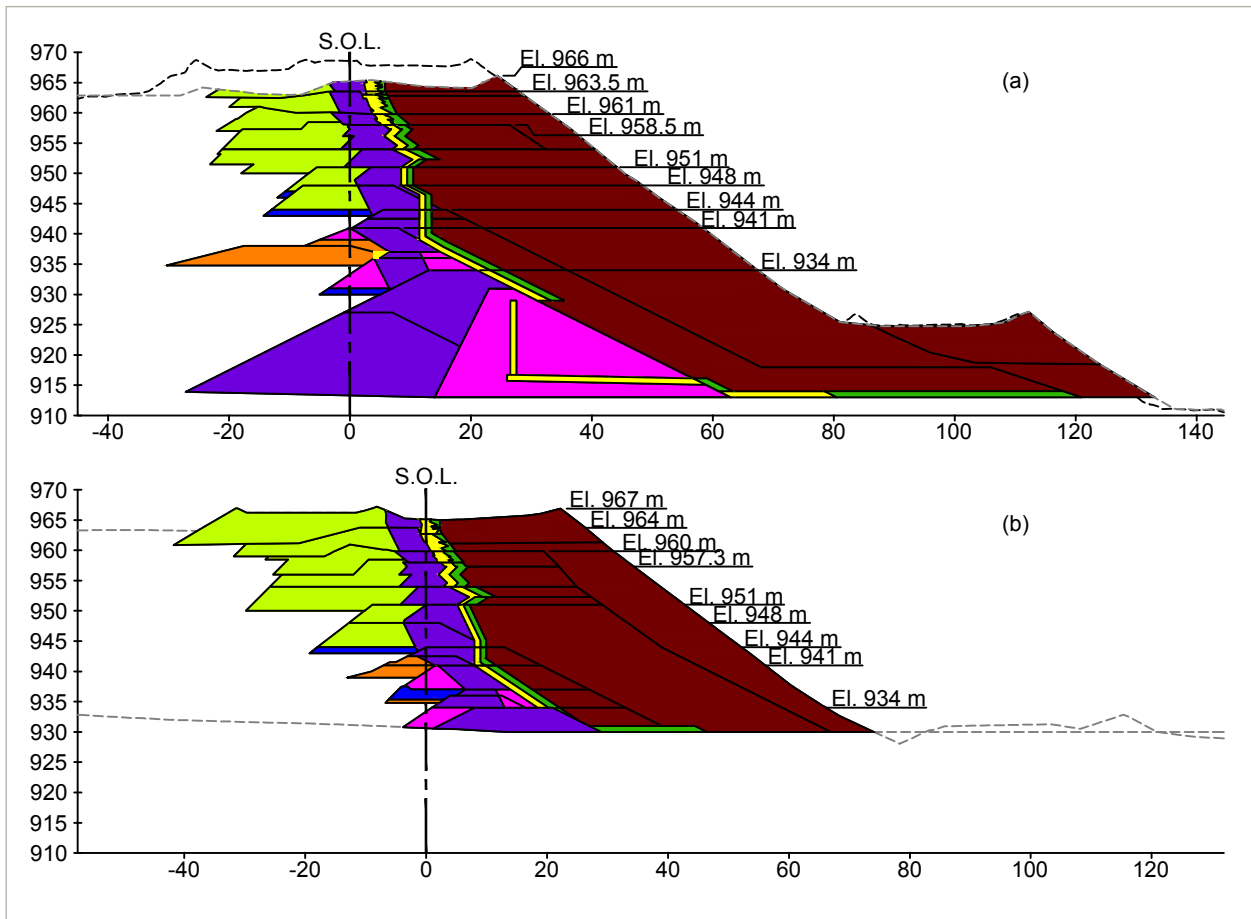
The BGC report also commented on the application of the Observational Method to these conditions. Citing its chief progenitor Ralph Peck, the report recognized that this design strategy requires preplanned actions to deal with "every unfavourable situation that might be disclosed by the observations."<sup>68</sup> But it also acknowledged that any brittle behaviour detected by the instrumentation would result in strength reduction too rapid to recognize and respond to. Hence the need, it said, for the minimum  $FS = 1.1$  and its associated residual strength interpretation as a contingency. As a result, the existing buttress on the Main Embankment would be raised, and a new buttress about 8 m high would be added to the Perimeter Embankment. This was to include what would become the area of the breach.

In a final irony, the Stage 10 buttress was scheduled for construction on the Perimeter Embankment in late 2014 or early 2015. Had it been in place on August 3, 2014, the dam would have survived.

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# 5 | Panel Observations

**FIGURE 5.4.10: DAM CONFIGURATION ON AUGUST 3, 2014. (a) MAIN EMBANKMENT (b) PERIMETER EMBANKMENT AT BREACH SECTION**



## 5 | Panel Observations

### 5.4.10 COMMENTARY

The preceding account is in many ways a story of too little, too late. From the beginning, dam raising proceeded incrementally, one year at a time, driven by impoundment storage requirements for only the next year ahead. More reactive than anticipatory, there was little in the way of long-term planning or execution. This was most clearly displayed by the absence of an adequate water balance or water treatment strategy, and the overtopping failure that nearly resulted. Moreover, the related absence of a well-developed tailings beach violated the fundamental premise of the design as a tailings dam, not a water-storage dam.

**The preceding account is in many ways a story of too little, too late. From the beginning, dam raising proceeded incrementally, one year at a time, driven by impoundment storage requirements for only the next year ahead.**

The same problem was apparent in production and scheduling for mine waste used in dam construction. The design was caught between the rising water and the Mine plan, between the imperative of raising the dam and the scarcity of materials for building it. Something had to give, and the result was oversteepened dam slopes, deferred buttressing, and the seemingly ad hoc nature of dam expansion that so often ended up constructing something different from what had originally been designed.

Ultimately, the tortuous, incremental nature of this process, and the constraints under which it was conducted, caused it to lose sight of basic precedent. With a slope steepness ordinarily reserved exclusively for rockfill dams on sound rock foundations, the Perimeter Embankment at the breach section was allowed to reach a height of almost 40 m with an unbuttressed downstream slope of 1.3H:1V.

Not just the design process but also the design itself had shortcomings. Even if not contributing directly to the failure, some design details were problematic. Already thin to begin with, reducing the core width from 8 m to 5 m made it even more vulnerable to differential settlement and cracking. Both the filter and transition zones were just 1 m wide, placing great demands on their performance. Yet in a sampling of as-placed Zone S filter gradations, the Panel found that 30% were too coarse to meet the  $D_{15} < 0.7$  mm filter criterion and 70% had internally unstable grading,<sup>69,70</sup> with only about 25% satisfying both filter and internal stability requirements.

## 5 | Panel Observations

There were ambiguities in the governing factor of safety, adapted from CDA Guidelines never intended for tailings dams. An  $FS = 1.3$  design criterion using peak effective-stress strength left little margin for error, and trigger-level factors of safety for critical piezometric conditions were even lower at 1.1. Such values may have made it easier to rationalize the departure from slope precedent, but harder to gauge just how closely dam raising was approaching the edge of the cliff.

There was an oversimplified conception of the complex stratigraphy of the glacial deposits described in section 5.2. An Upper GLU unit had been encountered in groundwater well GW96-1A and a lower unit in sonic borehole VW11-10. But only the lower unit was included in stability analysis of the Perimeter Embankment, and it had no influence on calculated factors of safety. The possibility that the upper unit might be present beneath the Perimeter Embankment was not accounted for in conceptualization of geologic conditions. More than this, its stress history was much less favourable.

Yet the overarching problem, and the one the Panel finds most troubling, is the failure throughout to adopt the appropriate undrained strength interpretation for the glaciolacustrine silts and clays in the foundation. These materials were assumed everywhere to be stiff, and therefore overconsolidated, although there was never any attempt to quantify their degree of overconsolidation or stress history. And even if they were overconsolidated to begin with, it was not recognized that the increasing loads imposed by the dam as it grew higher would eventually cause them to reach a normally consolidated state.

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There is a fundamental difference in pore pressure behaviour between these two conditions and the undrained strengths they produce. Overconsolidated clays are dilatant during undrained shearing. That is, they tend to increase in volume, producing no positive pore water pressures. By contrast, normally consolidated clays are contractive and do develop positive pore pressures. This difference in pore pressure response during shearing makes the undrained strength of a normally consolidated clay lower than the same material in an overconsolidated state. But the design did not account for the undrained strength that would pertain if the dam were to fail rapidly—which proved in the end to be the case.

## 5 | Panel Observations

Rather, the design was based exclusively on ESA in various forms using peak and residual strengths, all of which neglected pore pressures that would develop in normally consolidated GLU during rapid, undrained shearing. The design never incorporated an undrained strength analysis (USA), except in one instance. A USA performed for Stage 6 using an undrained strength typical of normally consolidated clays produced a factor of safety of only 1.1. But this was not seen to be the operative strength and was not considered further here or in subsequent stages. If undrained strength behaviour had been properly understood and applied throughout, the outcome could have been much different.

The Observational Method was invoked early on as the basis for design. This commonly accepted approach uses observed performance from instrumentation data for implementing preplanned design features or actions in response.

But there were a number of problems in applying this strategy to the Mount Polley dam that are treated in the following section. The first was simple geometry. The Observational Method relies on measuring the right things in the right places. While this was comparatively easy over the 1,000 m length of the Stage 1 dam, it became increasingly difficult as the length grew to 5 kilometres (km) by Stage 9. Nor could foundation instrumentation be installed beneath the dam crest and slopes where piezometric data mattered most. The slopes were too steep to be accessible, and few instruments installed on the crest could survive the near-constant construction there for very long. As a result, the few piezometers and inclinometers at the Perimeter Embankment were too far beyond the dam toe to produce critical data, and too far between to cover the area where the breach occurred.

Even more fundamentally, the piezometers as installed were only capable of measuring static (“water table”) pore pressures and, if properly located, those induced by applied loads. But piezometers cannot measure pore pressures induced by undrained shearing because the location of the failure surface on which to measure them cannot be known in advance.

The remaining problem is that the Observational Method is useless without a way to respond to the observations. Constructing buttresses and obtaining the necessary mine waste had been hard enough under ordinary circumstances. Were the instruments to warn somehow of a rapidly developing failure, there would be no way to respond in time to avert it. Hence, the Observational Method could not be relied on to determine the need for buttressing, so the buttress would be required regardless.

This fact was belatedly recognized in the Stage 10 design just days before the breach—the final fateful instance of too little, too late.



# 5 | Panel Observations

## 5.5 INSTRUMENTATION AND MONITORING

### 5.5.1 PRE-BREACH MONITORING OF TSF

Geotechnical instrumentation was installed beginning with Stage 1 of Main Embankment construction in 1996 and early 1997. During the initial phase, the focus was on vibrating wire piezometers, survey monuments, drain flow monitoring, and monitoring wells.<sup>71</sup> The first inclinometers on the Main Embankment were installed in July 2001. During the pre-breach period, instrumentation was installed at a total of 12 sections for the three embankments, Main, Perimeter and South (see Appendix F, Drawing F1). Further details of the inclinometers, piezometers, and drain flow monitoring during pre-breach monitoring are presented in Appendix F, Attachment 1.

A total of 10 inclinometers were installed after the start of operations. Of these, nine were still operating when the failure occurred: six at the toe of the Main Embankment and three along the toe of the Perimeter Embankment. One of the inclinometers along the Perimeter Embankment (SI11-04) was still being read, but was not reliable due to “a compression failure”<sup>72</sup> and had been replaced by Inclinometer SI12-04. Therefore, the Perimeter Embankment had two reliable inclinometers.

Vibrating wire piezometers were installed during ongoing construction activities at the 11 sections shown in Drawing F1. The last two sections (J and K) were added in 2011. As of August 2014, there were a total of 64 operating piezometers and 52 non-operating piezometers, of which 47 in the Main Embankment operated and 34 did not (see Appendix F, Attachment 1). Piezometers can fail not only due to instrumentation defects but also due to construction damage to piezometer cables. For example, during Stage 4 construction from May 2005 to October 2006, “22 piezometers were accidentally destroyed,” of which five were repaired.<sup>73</sup> In contrast, a number of the piezometers installed in 1996 and 1998 were still operating in 2014.

Piezometers were installed in the dam foundation, in various embankment components, such as the upstream fill, core, and downstream transition zone, in drains located in the embankment and foundation, and in the tailings upstream from the embankment.

## 5 | Panel Observations

The majority of the piezometers maintained steady pore pressures during 2014. Typical observations of piezometer pore pressure readings during construction were:

- Pore pressures in foundation piezometers typically increased due to fill placement and dissipated readily following construction.
- Pore pressures in piezometers located in embankment components (core and other downstream layers) and drains were stable.
- Pore pressures in tailings and upstream fill increased in response to the rising pool level.
- Piezometers located near the upstream toe drains experienced less pore pressure increases than those near the pond elevation.<sup>74</sup>

During the first phase of construction in 1996–1997, artesian pressures were observed in three of the six foundation piezometers in the Main Embankment. This prompted the development of trigger levels, or action levels, for many of the piezometers in the foundation and drains.<sup>75</sup>

As part of their annual construction manual in 2012, AMEC developed the instrumentation trigger framework shown in **Table F.1.1**,<sup>76</sup> Appendix F. This framework is for all the inclinometers and the Main Embankment foundation piezometers. The AMEC construction manual states that “embankment construction will be suspended if the inclinometers or piezometers fall under the yellow or red condition described in the Table, and/or if embankment foundation piezometer data indicates a significant increasing trend.” No corresponding trigger levels were established for the Perimeter Embankment piezometers because “factor of safety values...are sufficiently high that monitoring of piezometric trends, without defined trigger levels, is deemed sufficient.”<sup>77</sup>

Drain flow of the foundation drains and chimney drain was measured for the Main Embankment during the first phase of construction. Flow measurements were also initiated when similar drains were installed in the Perimeter and South Embankments. Upstream drains were installed in the tailings (also referred to as “upstream toe drains”<sup>78</sup>) at all the dams as they progressed in height, and these flows were also measured starting in 1996. Flows from these drains report to the seepage collection ponds constructed downstream of each dam. These flows were measured monthly (weather permitting) in a manifold for the Main Embankment and across ditch profiles close to the ends of the outlet pipe for the Perimeter and South Embankments.<sup>79</sup> In Appendix F, drain flow readings are shown in **Figure F.1.2**, and these results are further discussed.

Survey monuments were used from Stage 1 construction until about 2010 to measure surface movements of the embankments. These were installed after completing the raise construction.

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## 5.5.2 PRE-BREACH MONITORING IN BREACH AREA

Locations of the inclinometers and piezometers in the breach area are shown in Appendix F, Drawing F2. In this area, one reliable inclinometer was located about 300 m east of the breach. It was about 15 m to 44 m from the toe of the embankment at the time of the failure. There were nine operating and 13 non-operating piezometers along this section of the Perimeter Embankment. Locations of all the piezometers are shown in the sections in Appendix F, Drawings F3 and F4.

The upstream toe drain in the tailings shown in Drawings F3 and F4 was located at El. 946.3 m. Seepage collection elements for the upstream toe drain are shown in Drawing F4. Flows were conveyed along a drainage ditch to the Perimeter Embankment seepage collection pond at the time of the breach.

## 5.5.3 PANEL KEY OBSERVATIONS

Section 5.2 clearly demonstrates that foundation conditions in the area of the breach were complex and that the Upper GLU layer was not continuous along the full length of the Perimeter Embankment. The foundation conditions assumed for the initial and ongoing design were based on only four drillholes deeper than 8 m, none directly in the area of the breach. A sentinel control section was therefore not identified, and instrumentation could not be installed to monitor this sentinel section.

Foundation piezometers could not be installed after the downstream slope was constructed at an angle of repose slope (1.3H:1V). Access to the slope was impossible, and piezometers installed from the crest into the foundation would not have been at the correct locations to measure increased pore pressure below the advancing downstream slope. Piezometers downstream from the dam toe (as at Section D) were too far away from the slope to provide any useful information, as was also the case for the inclinometer.

The complex configuration of the internal embankment zoning made it very difficult, if not impossible, to install replacement piezometers in a specific fill zone at a specific depth. Most piezometers were installed during the construction phases, and many were damaged during those stages.

**Foundation piezometers could not be installed after the downstream slope was constructed at an angle of repose slope (1.3H:1V).**

**The complex configuration of the internal embankment zoning made it very difficult, if not impossible, to install replacement piezometers in a specific fill zone at a specific depth.**

## 5 | Panel Observations

While some piezometers provided very useful information (e.g., the tailings piezometers provided pore pressure values that could be applied to slope stability analyses), the Perimeter Embankment instrumentation overall could not have provided any warning of the looming failure. Nor did it provide any monitoring relevant to the critical failure mode.

It should be noted that if failure were to occur suddenly, deformation monitoring could not provide timely warning and a more defensive design would be appropriate. The failure mode encountered here was sudden without any surface evidence and is an example of this behaviour. In their design for the proposed Stage 10, BGC anticipated this issue and recognized that a berm would be required for the Perimeter Embankment.<sup>80</sup>

# 5 | Panel Observations

## 5.6 WATER BALANCE

### 5.6.1 INTRODUCTION

A clear distinction can be made between the water balance actions and outcomes during the Phase 1 Active Mining, the Care and Maintenance period and the Phase 2 Active Mining. **Table 5.6.1** provides a summary of the mining activities, the Mine areas and the water management operating conditions. Appendix G describes in more detail the design objectives, water balance models and their implementation as well as observations found in the TSF Annual Inspection Reports. The consequences of the operational conditions are presented in this section.

**TABLE 5.6.1: MOUNT POLLEY MINE LIFE**

YEAR	ACTIVITY	MINE PITS	WATER MANAGEMENT OPERATING CONDITIONS
1997 – 2001	Phase 1 Active Mining	Cariboo and Bell	Deficit
2001 – 2005	Care and Maintenance		Neutral
2005 – 2014	Phase 2 Active Mining	Wight, Springer, Southeast Zone, Pond Zone	Surplus

## 5 | Panel Observations

### 5.6.2 PHASE 1 ACTIVE MINING

From 1997 to 2001 MPMC mined the Cariboo and Bell pits. The area of disturbance in the mining area was quite small and the overall TSF water balance was in a deficit. Water from Polley Lake and surface runoff on-site helped to provide the annual operating requirements.

### 5.6.3 CARE AND MAINTENANCE

As a result of low copper prices, the Mine suspended operations from October 2001 to February 2005. A small staff was maintained at the Mine and they managed the TSF water balance carefully, making sure that sufficient freeboard was maintained. Towards the end of the Care and Maintenance period, mine development in preparation for start-up was underway and surface water accumulated in the TSF. It was recognized at this time that plans would have to be developed to discharge water to the environment.

### 5.6.4 PHASE 2 ACTIVE MINING

During the second phase of Active Mining, the footprint of the Mine was expanded to a total of four additional pits and associated infrastructure and waste rock piles. MPMC and the designers knew that there was a surplus of water in the TSF and that strategies had to be developed to discharge water. MPMC also understood the need for permitted discharge from the TSF.

In 2009 MPMC prepared a report entitled *Mount Polley Mine Technical Assessment Report for a Proposed Discharge of Mine Effluent*.<sup>81</sup> In this report, alternative discharge approaches were evaluated. The approach selected was to apply for a permit to discharge water to Hazeltine Creek. A permit amendment was granted on November 7, 2012 that allowed the discharge of up to 1.4 million cubic metres (m<sup>3</sup>) per year of filtered water to Hazeltine Creek. The maximum discharge is 35% of flow in the Creek and the window is April to October. In April of 2014 it was estimated that only 170,500 m<sup>3</sup> total discharge was possible, due to constraints of permit requirements.

Discharging small amounts of extra water to Hazeltine Creek did not have a significant impact on the water surplus. Permitting of a water treatment plant was pursued in late 2013 and the Terms of Reference for Discharge was issued by the Ministry of Environment on March 26, 2014. Completion of treatment plant construction was expected in September 2014 or later. This plant would allow total annual discharge of 3 million m<sup>3</sup>.

## 5 | Panel Observations

### 5.6.5 WATER BALANCE AND TSF CONSTRUCTION

During the life of the Mine, two water balance models were used. The first was compiled by KP and was used from start-up until about 2005. The second was based on a model modified by MPMC to account for the expanded footprint of Phase 2 Active Mining. MPMC updated the water balance regularly with site-specific climatic and operating data as well as bathymetric surveys of the TSF pool. The EOR reviewed the water balances throughout operations except from 2010 to 2014. The Panel could not find any documentation explaining the reason for this change in procedures.

The embankment of the TSF was raised on a regular basis, typically on an annual basis. The design engineers used the outcome of the water balance calculations by MPMC to select the height of the increase. The overall approach was well summarized by KP in 2005 in their report entitled *Design of the Tailings Storage Facility to Ultimate Elevation*:<sup>82</sup>

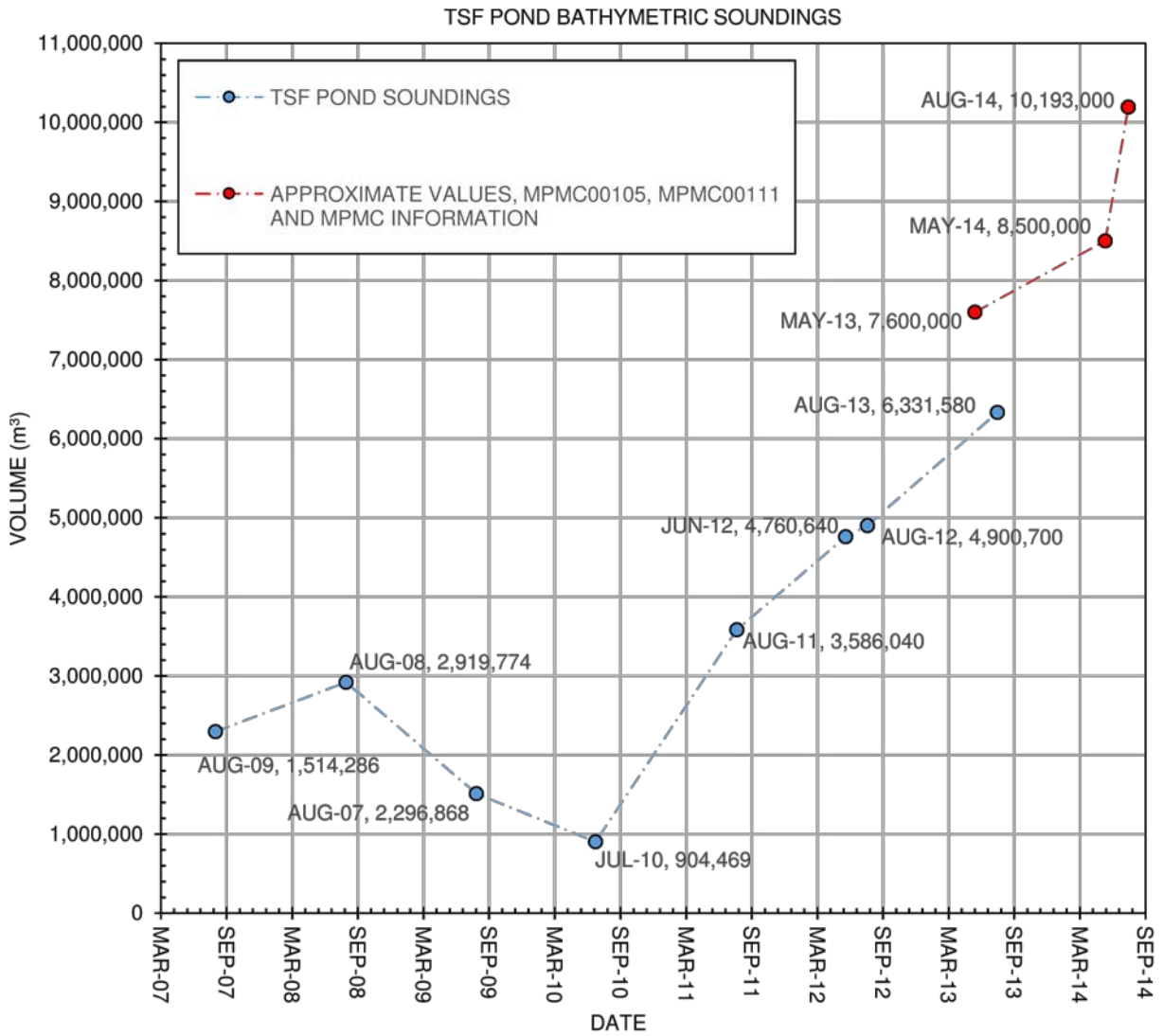
*Each embankment raise will provide incremental storage capacity for approximately one-year of production. The filling schedule incorporates sufficient live storage capacity for containment of runoff from the 24-hour PMP volume of 679,000 m<sup>3</sup> at all times, which would result in an incremental raise in the tailings pond level of about 0.39 m, with an additional allowance of 1 m for freeboard for wave run-up.*

The water balance model included the site-specific information to the date of analysis, and future conditions were based on average climatic conditions. They did not account for specific wet year conditions.

**Figure 5.6.1** shows the accumulation of water in the TSF as determined from bathymetric surveys. The figure also shows approximate volumes reported in the records for three dates (refer to Appendix G).

# 5 | Panel Observations

FIGURE 5.6.1: WATER ACCUMULATION IN TSF





## 5 | Panel Observations

### 5.6.6 OVERTOPPING IN MAY 2014

On Saturday May 24, 2014, a potential “dam breach” event occurred at the TSF as a result of a large rainfall, approximately 24 mm in 24 hours, followed by ongoing rain. On Monday May 26 the water level was at El. 966.3 m, which resulted in a freeboard of 0.7 m to the top of the constructed core at El. 967.0 m, as stated in the 2013 Annual Construction Report (refer to Appendix G). The core was found to have a few low spots at 966.3 m (Corner 3), 966.4 m (Corner 2), 965.5 m (Corner 5) and 966.2 m (at the pipe crossing on the Perimeter Embankment). Wet spots and standing water were observed at Corner 3 and the pipe crossing, but no major erosion due to large flows or direct seepage. All the low areas were addressed through emergency construction measures by Thursday, May 29 when the pool water level increased to El. 966.45 m. The top of the Perimeter Embankment was increased to El. 967.3 m. All water collection systems were diverted from the TSF and water was routed for storage in the Cariboo Pit.

The pond elevation was monitored on a daily basis from the end of May until the time of the breach. During that time, construction proceeded to increase the embankment height. On August 3, 2014, the day before the breach, the freeboard was 2.3 m.

### 5.6.7 COMMENTARY

The way in which the water balance was utilized with annual raises had significant limitations. Construction of annual embankment raises was based on water balance evaluations using average climatic conditions at the site. This does not provide a reliable approach to establishing adequate capacity for tailings and water storage. Uncertainties in the water balance input parameters combined with uncertainties in climatic conditions and construction schedules cannot provide a robust design for water containment. Construction delays due to site climate or availability of construction materials could impact the targeted capacity. Overtopping of the embankment occurred at selected locations in May 2014.

**The way in which the water balance was utilized with annual raises had significant limitations.**

As indicated in section 4, the Perimeter Embankment did not fail due to overtopping; however, storing large volumes of water in the TSF had other implications.

## 5 | Panel Observations

Throughout most of its term as the EOR, KP emphasized the importance of maintaining a beach width of at least 10 m. The Panel does not consider this to be a beach. Nevertheless, the principle was clear: the Mount Polley TSF embankments were not designed as water-retaining dams, and a beach would provide some stabilizing function. It was impossible to maintain beaches against all the embankments throughout the year during the last years of operation because of the large volumes of water stored in the TSF. Section 5.4 summarizes the chronic problems experienced in beach development.

MPMC was aware of the water surplus conditions at the start of Phase 2 operations. The pond volumes in **Figure 5.6.1** show that the last number of embankment raises were necessary to store water and not necessarily much higher tailings production. It is not clear to the Panel why it took so long to design and implement a water treatment strategy that would provide for a significant reduction in the amount of surplus water stored on the TSF.

**It is not clear to the Panel why it took so long to design and implement a water treatment strategy that would provide for a significant reduction in the amount of surplus water stored on the TSF.**

The pore pressure in the tailings piezometer at the breach location increased as a result of the higher pool elevation (refer to section 5.5). This happened despite the presence of the upstream toe drain. The higher pore pressure had a secondary effect on the overall slope stability.

Finally, the volume of water in the pool at failure, about 10 million m<sup>3</sup>, resulted in a much larger loss of solids from the TSF due to erosion than might have occurred if there was a smaller pool (refer to Appendix C). And a wider beach of unsaturated tailings might have delayed breach development long enough for emergency actions to have been taken.

# 5 | Panel Observations

## ENDNOTES

- 1) Klohn Crippen Berger, 2014, Site Investigation Progress Report No. 1, Report to BC Ministry of Energy and Mines.
- 2) Kulhawy, F.H. and Mayne, P.W., 1990, "Manual on Estimating Soil Properties for Foundation Design", Electric Power Research Institute (EPRI), EL-6800, 308 p.
- 3) MP10012
- 4) MP00054
- 5) MP10012
- 6) U.S. Dept. of the Navy, 1982, Soil Mechanics Design Manual 7.1, NAVFAC DM-7.1, 205 p.
- 7) MP00001
- 8) MP00005
- 9) MP00001
- 10) MP00054
- 11) MP00083
- 12) MP00069
- 13) MP00019
- 14) MP10009
- 15) MP00008
- 16) MP00012
- 17) MP00008
- 18) MP10032
- 19) Panel Interview, 12/12/14.
- 20) SUB00023
- 21) MP00011
- 22) MP00013
- 23) MP00014
- 24) MP00021
- 25) MP00118
- 26) MP00038
- 27) MP00021
- 28) MP00126
- 29) MP00026
- 30) MP00031
- 31) MP00076
- 32) MP00137
- 33) MP00139
- 34) Panel Interview, 12/12/14.
- 35) MP00028
- 36) MP00033
- 37) MP00077
- 38) MP00104
- 39) MP10035
- 40) MP00032
- 41) MP00036
- 42) MP00035
- 43) MP00222
- 44) MP00225
- 45) MP10034
- 46) MP00037
- 47) MP00181
- 48) MP00039
- 49) MP10012
- 50) MP00040
- 51) MP00217

## 5 | Panel Observations

- 52) MP00042
- 53) MP00224
- 54) MP00043
- 55) Panel Interview, 10/22/14.
- 56) MPMC00048
- 57) MP00045
- 58) MP000187
- 59) MP00044
- 60) Panel Interview, 10/22/14.
- 61) Panel Interview, 10/22/14.
- 62) MPMC00048
- 63) BGC00002
- 64) BGC00003
- 65) MP00192
- 66) BGC00004
- 67) MP00208
- 68) Peck, R.B., 1969, Advantages and limitations of the observational method in applied soil mechanics. *Geotechnique*, v. 19, no. 2, pp. 171-187.
- 69) Kenney, T.C. and Lau, D., 1985, Internal stability of granular filters. *Canadian Geotechnical Journal*, v.22, pp. 215-225.
- 70) Kenney, T.C. and Lau, D., 1986, Internal stability of granular filters: reply. *Canadian Geotechnical Journal*, v.23, pp. 429-423.
- 71) MP00019, note that the monitoring wells were for water level and quality measurements outside the TSF; these will not be further discussed in this section.
- 72) AMEC00164
- 73) MP00031
- 74) MP00044
- 75) MP00019
- 76) MP00040
- 77) AMEC00132
- 78) MP00037
- 79) MP00044
- 80) BGC 00007
- 81) MOE00001
- 82) MP00026

# 6 | Analysis of Breach Mechanics

## 6.1 INTRODUCTION

As demonstrated in sections 5.1 and 5.2, the breach of the Mount Polley Tailings Dam (the Dam) arose because of failure in the foundation of the Perimeter Embankment. According to Ministry of Energy and Mines (MEM) requirements, design with respect to overall stability must be compliant with CDA Guidelines. The specific guideline for a dam under construction and before reservoir filling requires a factor of safety (FS) of 1.3 where:

$$FS = \frac{\text{Available Strength}}{\text{Strength Required for Equilibrium}}$$

That is, the design requires a reserve resistance over and above that required to maintain equilibrium, and with this reserve resistance, it is expected that the structure will perform in a safe manner. This criterion has been accepted for tailings dams during construction, with a higher FS required if the dam has a long service life after it has been filled.

Many potential failure modes have to be considered to meet the requirements that  $FS = 1.3$ . The CDA Guidelines are not prescriptive with respect to potential failure modes. It is the obligation of the designer, as EOR, to recognize the potential failure modes, to characterize the operational strength of the materials associated with these potential failure modes, to adopt an appropriate method of analysis to calculate the FS, and to ensure that the FS is equal to or greater than 1.3 during the construction of the dam.

The Perimeter Embankment failed during construction, and hence the  $FS = 1$ . It moved sufficiently to lose containment of the impounded water and tailings that flowed out and eroded most of the displaced embankment.

In the following analyses, calculations will show that shear strengths determined by the Panel to reflect undrained failure of the Upper GLU beneath the Upper Till of the foundation are consistent with the strength required for limiting equilibrium, i.e.,  $FS = 1.0$ .

A detailed explanation of the process leading to failure of the Dam will be presented, and comparisons will be made with the assumptions that underpin the design in order to highlight the deficiencies associated with it.

# 6 | Analysis of Breach Mechanics

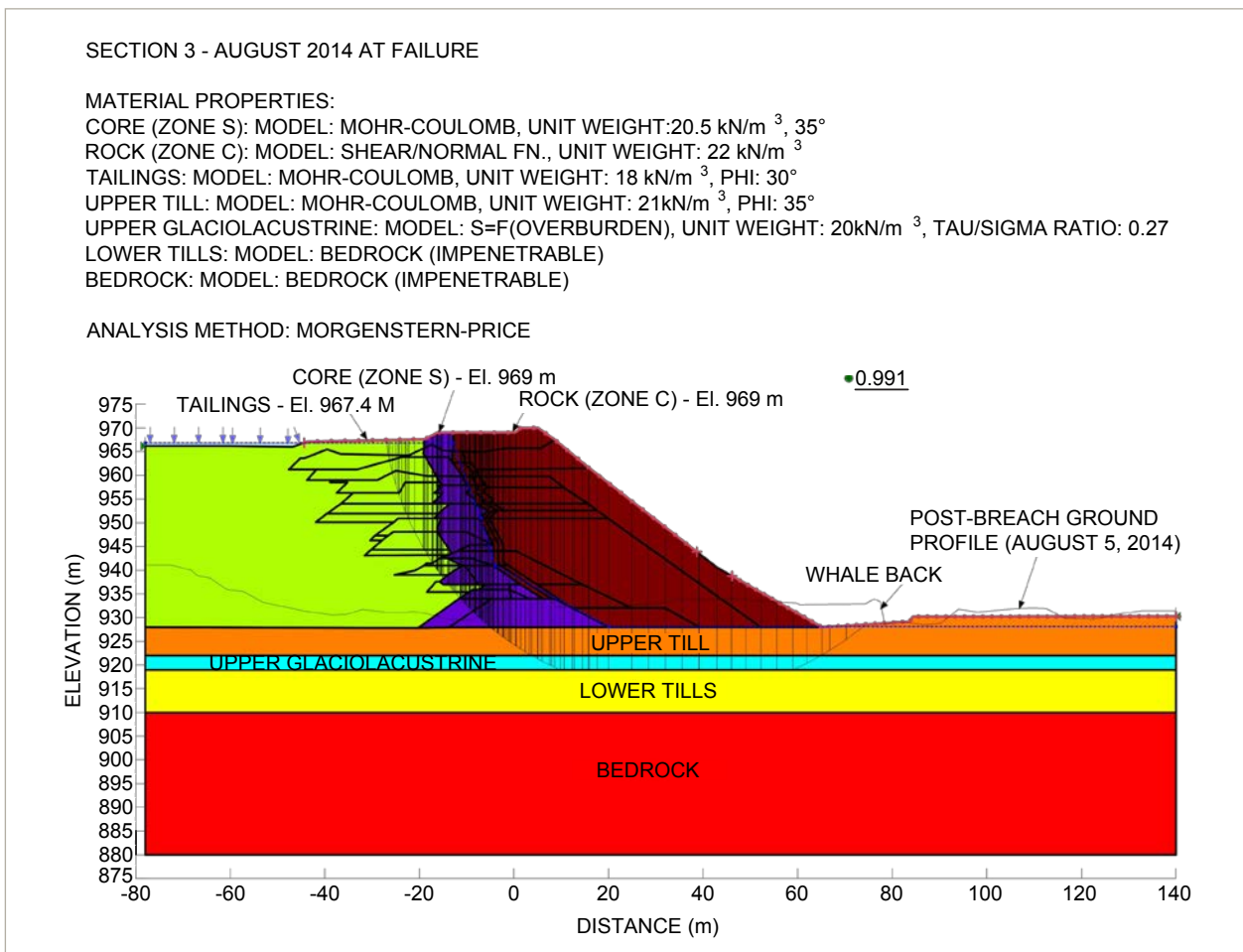
## 6.2 ANALYSES

### 6.2.1 LIMIT EQUILIBRIUM ANALYSES (2-D)

Analyses for purposes of designs are conventionally performed on two-dimensional (2-D) sections. Cross-section 3 (see section 5.2; Appendix D) was assumed to represent the stratigraphy more or less in the middle of the displaced mass.

**Figure 6.2.1** presents the detailed section. It is based on the last LiDAR survey of the embankment prior to failure, a detailed reconstruction of the top of the structure and pond elevation based on construction inspector reports, and it includes a shallow excavation at the toe of the embankment as reported to the Panel. More details associated with the compilation of this and related sections are presented in Appendix H.

**FIGURE 6.2.1: DETAILED SECTION USED FOR LIMIT EQUILIBRIUM ANALYSIS (HIGH WATER TABLE, UNDRAINED STRENGTH RATIO 0.27)**



## 6 | Analysis of Breach Mechanics

The Morgenstern-Price method of stability analysis and the SLOPE-W computer program were used for the computations. Both are recognized standard tools and were also used for the design of the structure at various stages.

Strength properties and densities for each stratum must be defined in order to calculate the FS. The values assumed are also displayed in **Figure 6.2.1**. Only the upper of the two GLUs defined in section 5.2 is included due to its high water content, its lower cone penetration testing (CPT) tip resistance, and overconsolidation ratio (OCR). Except for the strength of the Upper Till unit and the GLU, all strengths and densities are the same as those used in design studies. Based on pressuremeter testing, experience of the Panel members, and the limited pore pressure response during undrained triaxial testing, a frictional resistance of  $35^\circ$  was adopted for the Upper Till. Fully drained conditions are assumed up to failure. The magnitude of the undrained strength ratio in the GLU is then varied until FS=1 is obtained. For the case illustrated in **Figure 6.2.1**, this ratio is 0.27. In this case, the observed level of the pond is carried horizontally through the beach, which would, in most circumstances, be the design basis case (High Water Table case).

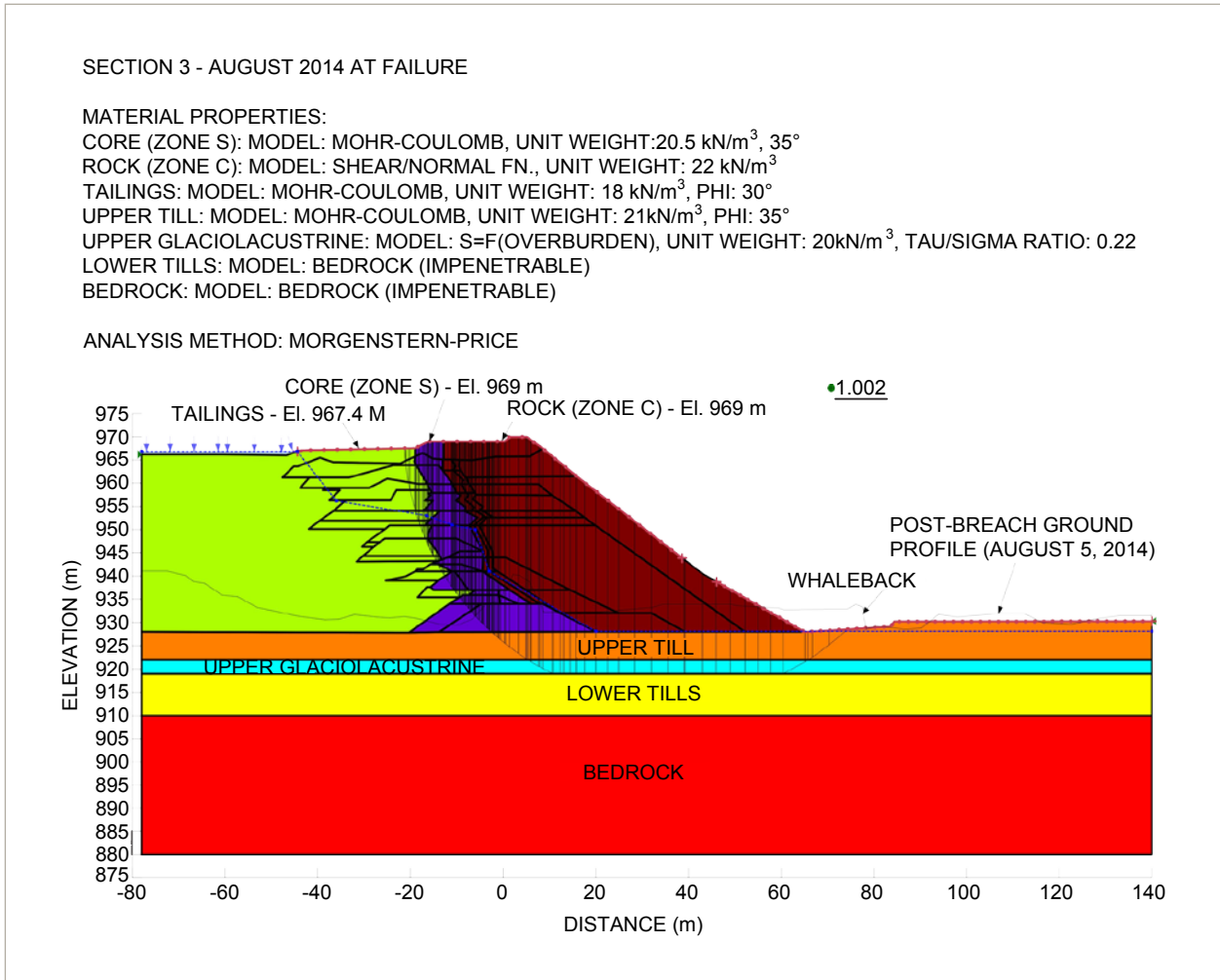
However, the installation of drainage at the upstream face of the core creates downward flow that will reduce the water pressure acting on the core of the Dam. A potential limiting case is shown in **Figure 6.2.2**, and the calculated strength ratio is 0.22 (Low Water Table case). The likely case is between these limits, with the Panel favouring a result above the average, say 0.25.

The calculated value represents the average resistance mobilized by the GLU at the instant of failure. It should be noted that this value lies sensibly in the middle of the range of the measured undrained strength (see **Table 5.3.1**), consistent with the hypothesis that the breach resulted from undrained failure of the GLU at an elevation of about 920 metres (m).

The actual available shear strength will vary with consolidation history as the Upper GLU responds to the stresses imposed by the embankment and the lateral loads transmitted by the impounded tailings and water. This will induce both normal and shear stresses in the Upper GLU. The Panel has not calculated these stresses in any detail. However, it is evident that the maximum applied stresses will substantially exceed the preconsolidation stress level associated with the Upper GLU. This response will reduce towards the breakout zone of the calculated slip surface and beyond, where the influence of applied stresses diminishes. Where the preconsolidation pressure has been overcome, the available shear strength will be that of a normally consolidated soil. Beneath the toe of the embankment and beyond, available strength will be higher, depending upon the local stresses and the preconsolidation stresses. The calculated average resistance reflects this distribution. Appendix E, Attachment 2 shows a comparison between vertical overburden stress and preconsolidation stress.

# 6 | Analysis of Breach Mechanics

**FIGURE 6.2.2: DETAILED SECTION USED FOR LIMIT EQUILIBRIUM ANALYSIS (LOW WATER TABLE, UNDRAINED STRENGTH RATIO 0.22)**





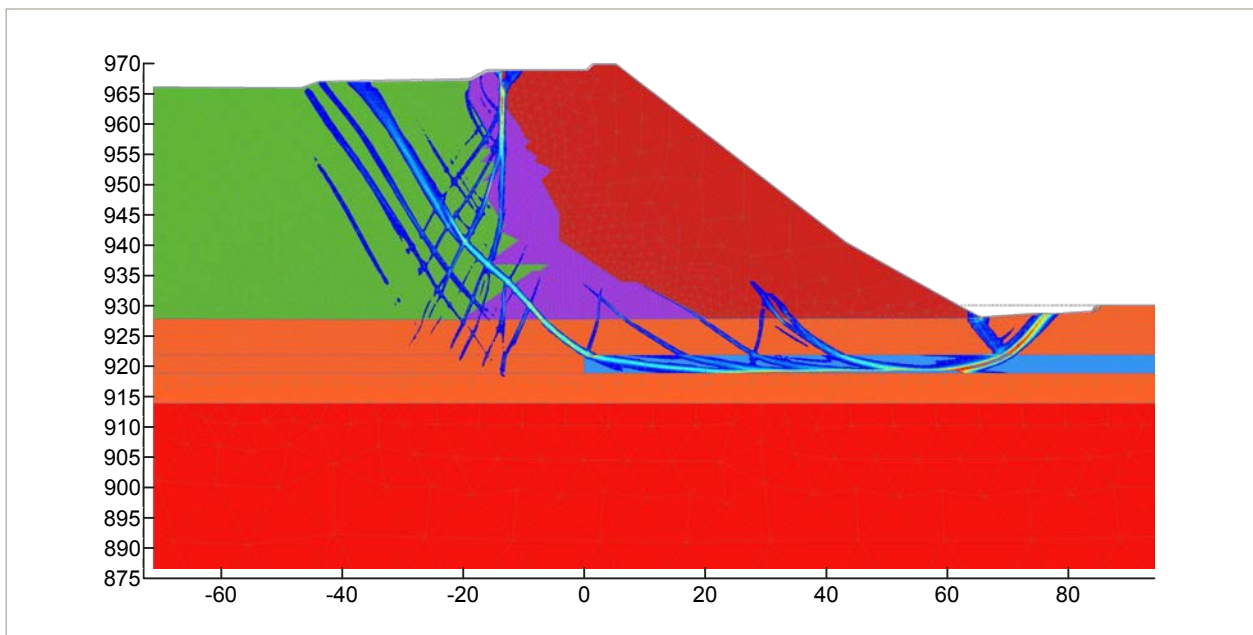
# 6 | Analysis of Breach Mechanics

## 6.2.2 DEFORMATION ANALYSES (2-D)

An alternate way of assessing the undrained failure mechanism is to calculate the deformation patterns that develop at failure. While not a routine design procedure, the means for conducting such analyses are facilitated by powerful numerical simulation tools. In this case, PLAXIS, a well-recognized computer program developed specifically to model soil deformations, was adopted.

**Figure 6.2.3** portrays the PLAXIS model at collapse. Prior to creating collapse, the model was constructed with essentially the same input parameters as used in the limit equilibrium analyses, except for the Upper GLU that is given a high strength to avoid yielding. The strength of the Upper GLU is then reduced until a deformation mechanism forms and the embankment collapses. This provides not only a measure of the strength of the Upper GLU at which failure occurs, but also an indication of the deformed shape arising from failure. In the model presented in **Figure 6.2.3**, collapse occurred at an undrained strength ratio of 0.29, which is to be compared with 0.27 calculated from the limit equilibrium analysis that incorporates the same boundary conditions. Lower undrained strength conditions would indicate significantly larger deformations. The figure also indicates the zones of localized strain that develop to facilitate motion. Variations of continuity of the Upper GLU with respect to this case yielded similar results.

**FIGURE 6.2.3: PLAXIS MODEL AT COLLAPSE (UNDRAINED STRENGTH RATIO 0.29)**



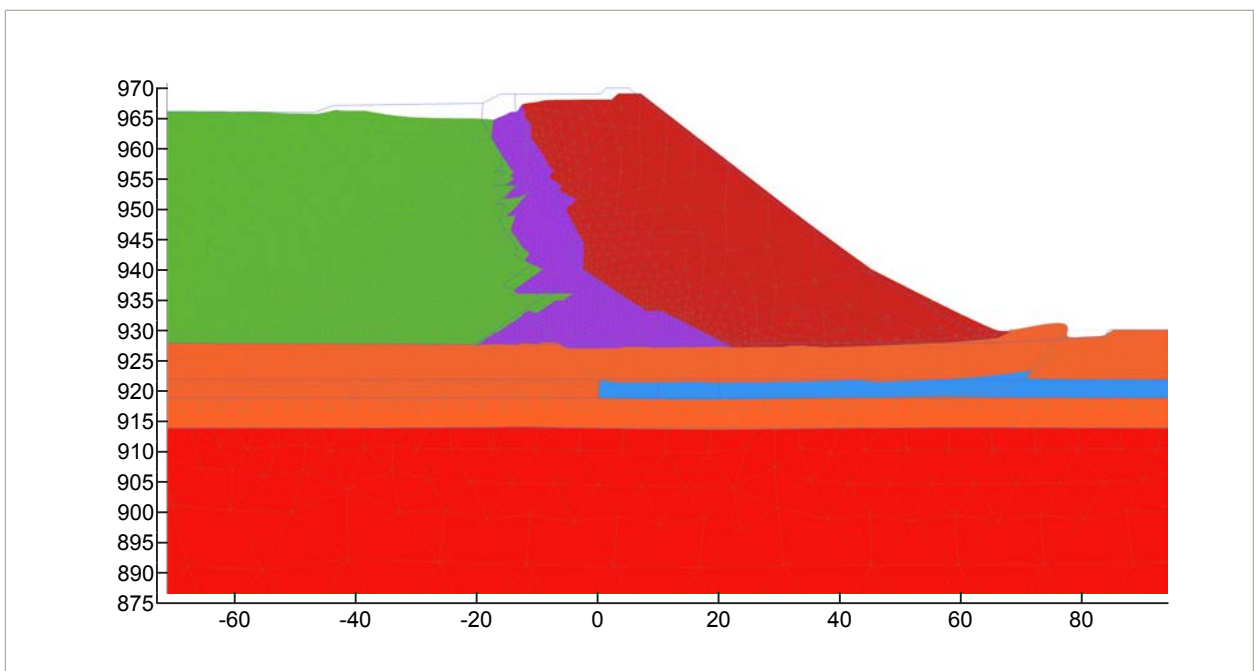
Both the entry and exit of the failure surface in the foundation correspond closely with field observations summarized in **Figure C4.2.2** in Appendix C.

## 6 | Analysis of Breach Mechanics

**Figure 6.2.4** is a scaled-up display to illustrate the calculated deformations. The rotational movement with a lesser lateral displacement are evident. Particularly striking is the thrust feature that occurs very close to the whaleback feature identified in section 5.1. Also significant is subsidence of the crest that allowed overflow to begin, initiating the breach process as described in section 5.1 and Appendix C.

The PLAXIS analyses provide compelling support for the hypothesis that the movements of the Perimeter Embankment arose due to the undrained failure of the Upper GLU.

**FIGURE 6.2.4: SCALED-UP FIGURE 6.2.3 TO ILLUSTRATE DEFORMATIONS**



## 6 | Analysis of Breach Mechanics

### 6.2.3 LIMIT EQUILIBRIUM ANALYSES (3-D)

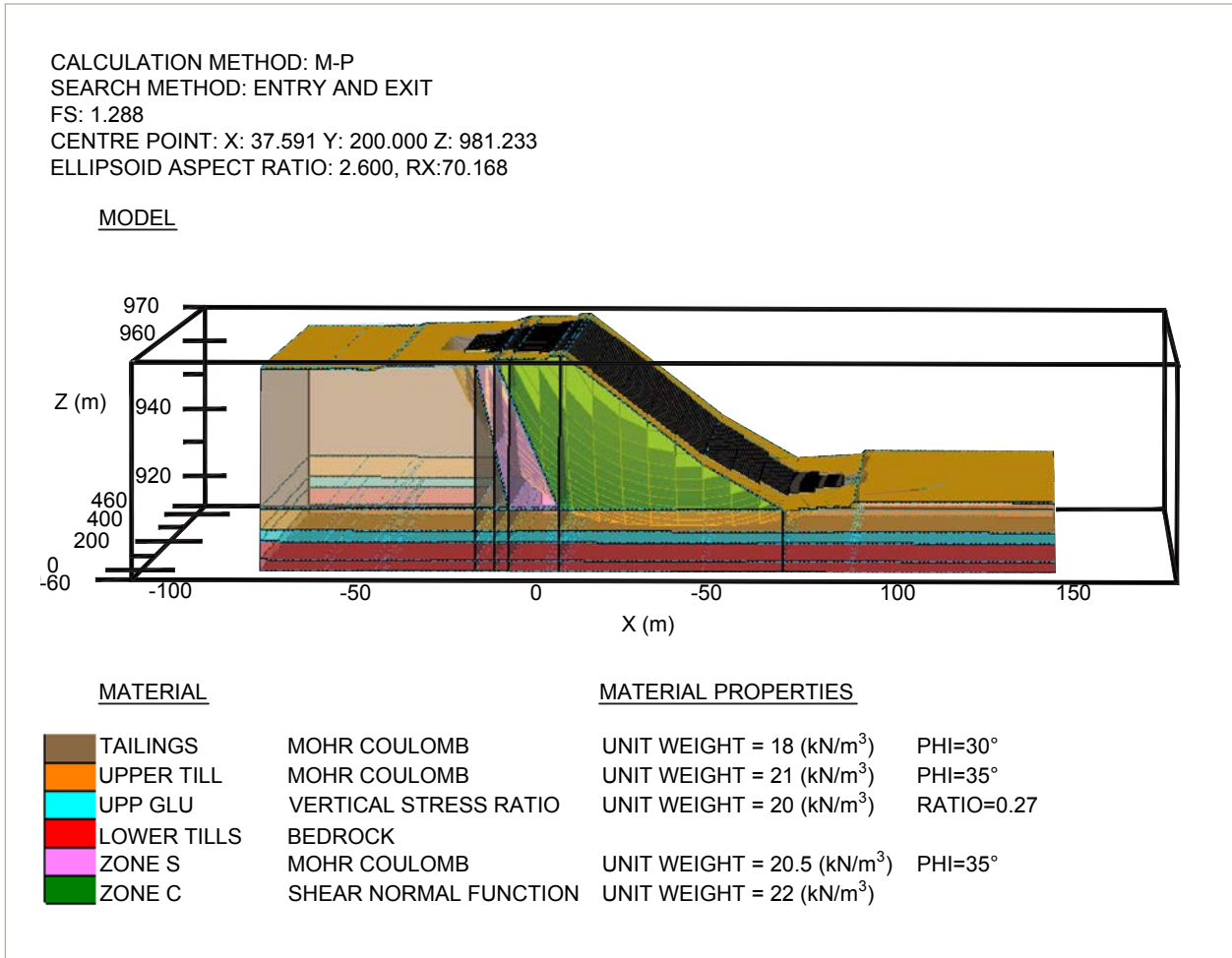
The length of the breach is relatively short compared to the height of the Perimeter Embankment at failure (~ 40 m). This is expressed as an Aspect Ratio (length/height) and is calculated to be 2.6. At this Aspect Ratio, three-dimensional restraints might be a significant factor influencing the analysis of the breach mechanism. At small Aspect Ratios, the side resistance acting on the potential moving mass increases in significance. This is ignored in the 2-D analyses described above, which are used routinely in design. Nevertheless, the Panel regarded it of value to assess three-dimensional considerations in order to fully explore the factors affecting the breach mechanism.

Three-dimensional limit equilibrium analyses have been conducted using the computer program SVSlope 3D, a widely accepted program for conducting such analyses. The geometry and boundary conditions are a three-dimensional extension of the case illustrated in **Figure 6.2.1**. All soil properties used in the 3-D analysis are the same as those employed in **Figure 6.2.1**.

**Figure 6.2.5** presents the 3-D case. The FS with an undrained strength ratio of 0.27 and an Aspect Ratio of 2.6 is calculated to be 1.3. This is a significant increase over the 2-D case, and it merits interpretation.

# 6 | Analysis of Breach Mechanics

**FIGURE 6.2.5: 3-D LIMIT EQUILIBRIUM ANALYSIS (UNDRAINED STRENGTH RATIO 0.27, ASPECT RATIO 2.6, FS IS ABOUT 1.3)**



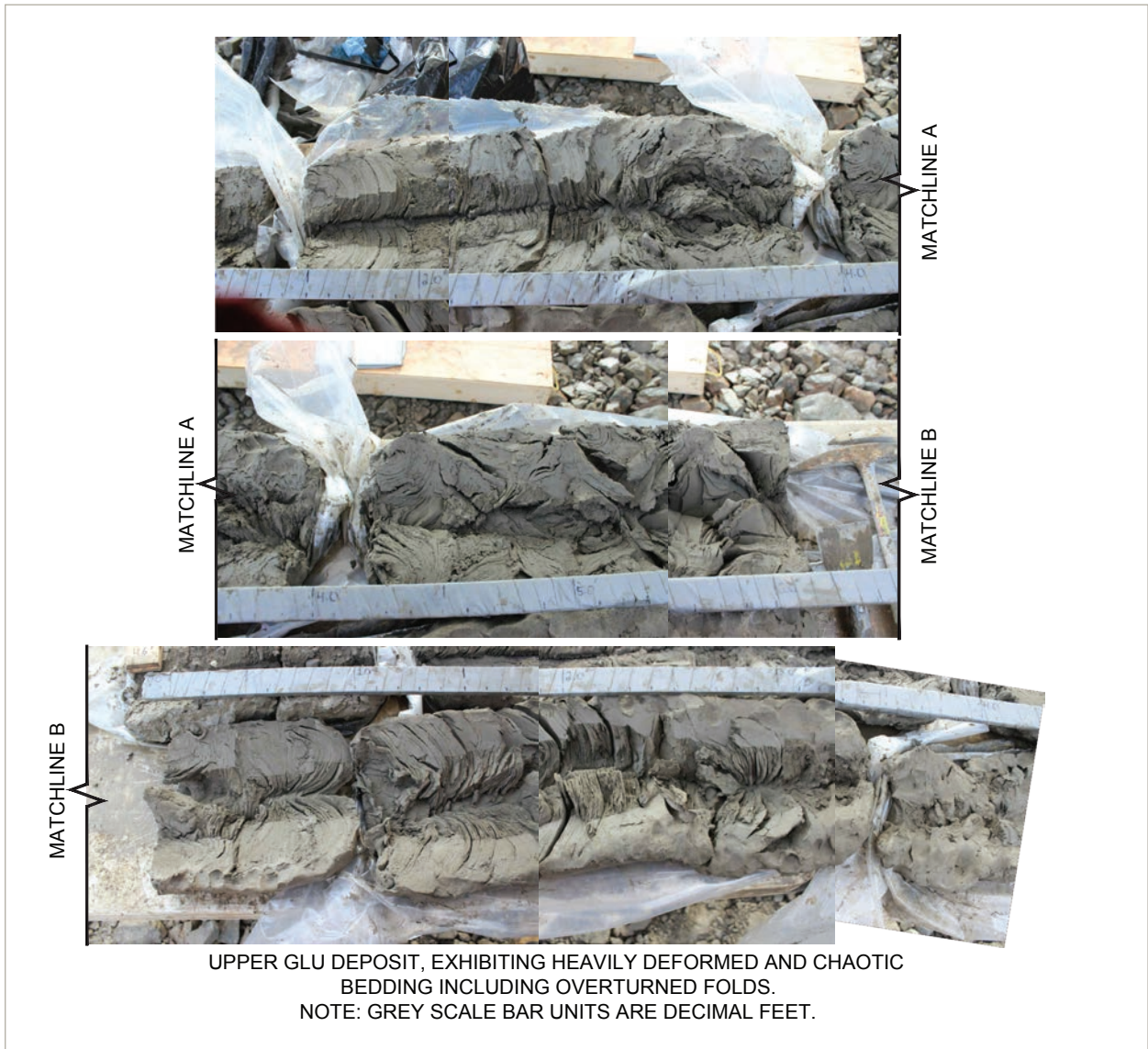
## 6 | Analysis of Breach Mechanics

As shown in Appendix D, Drawing D19, the sensitivities of the Upper GLU in the failure zone is about 1–3, based on CPT-measured tip resistances. Hence, as deformations developed in the Upper GLU, the available resistance reduced due to strain weakening and soil remoulding. Based on the observed sensitivity, it could have dropped to an undrained strength ratio of perhaps 0.13. Repeating 3-D limit analyses with these values yields an FS of about 1.1, which is close to collapse. Hence, as movements developed, the available resistance of the Upper GLU was reduced due to strain weakening to a degree that the three-dimensional restraints to movements at an Aspect Ratio of 2.6 were overcome. The idealizations involved in these 3-D analyses do not permit greater accuracy than expressed here. Going forward, a review of some of the assumed strength parameters that influenced the 3-D modelling and a more detailed representation of local geology that influence the 3-D results would be warranted. **Figure 6.2.6** displays visual evidence of the remolding processes that have occurred due to shearing of the GLU.

**As deformations developed in the Upper GLU, the available resistance reduced due to strain weakening and soil remoulding.**

# 6 | Analysis of Breach Mechanics

**FIGURE 6.2.6: TYPICAL SHEARING IN THE UPPER GLU**

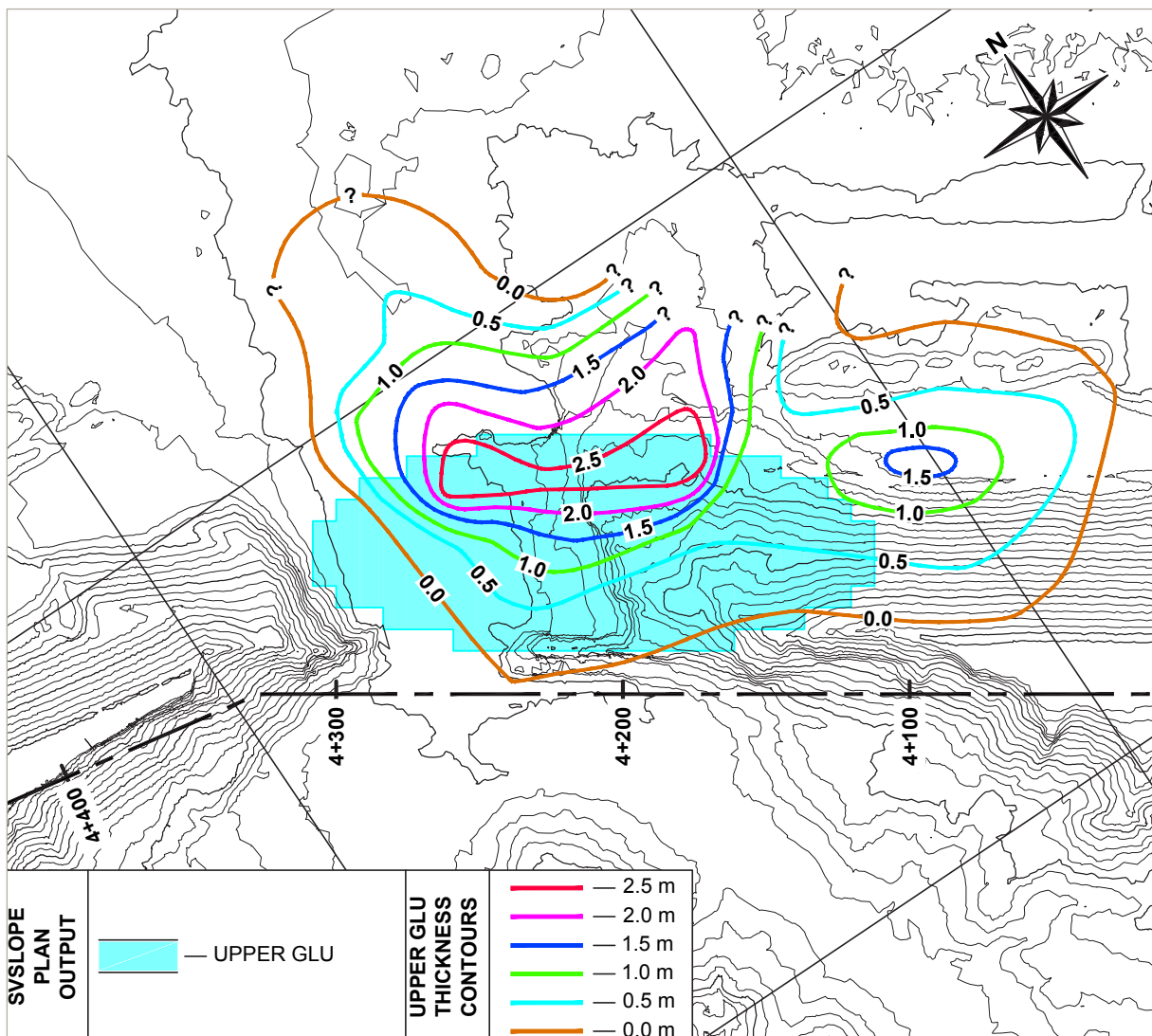




## 6 | Analysis of Breach Mechanics

Additional support for the insight provided by the 3-D interpretation can be found by comparing the footprint of the 3-D analysis with the Aspect Ratio of 2.6 where it intersects the Upper GLU. The distribution of the thickness contours of Upper GLU is presented in **Figure 5.2.6**. This comparison is shown in **Figure 6.2.7**, which indicates a striking fit between the extent of the Upper GLU mobilized in the 3-D analysis (shown in cyan) with the extent of the deepest portion of the Upper GLU.

**FIGURE 6.2.7: COMPARISON OF THE 3-D ANALYSIS WITH THE THICKNESS CONTOURS OF THE UPPER GLU**



# 6 | Analysis of Breach Mechanics

## 6.3 TRIGGER ANALYSIS

### 6.3.1 INTRODUCTION

Both the 2-D and 3-D analyses discussed above reflect a simplified interpretation of how failure began and subsequently progressed. They indicate that the foundation was brought to failure under fully drained conditions until the undrained strength was reached and the collapse of the embankment subsequently mobilized the undrained shear strength. After initial failure, the Upper GLU behaved in a strain-weakening manner, reducing its resistance as reflected by the observed sensitivity of the deposit. Ultimately, the increased load associated with the weakening material overcame the residual resistance of the stronger zones, allowing the unconstrained 3-D mechanism to develop. The calculations presented provide average undrained strength ratios at failure that are generally consistent with the magnitudes observed in the laboratory.

In order to understand the failure mechanism in more detail, it is of value to address two questions:

- 1) Was the loading path to failure fully drained?**
- 2) Was the shear strength at failure mobilized uniformly?**

### 6.3.2 PORE PRESSURE HISTORY

To address the first question, it is possible to calculate the pore pressure development and dissipation during embankment construction. If the pore pressures remain high, the available shear strength is reduced accordingly. This type of evaluation is an integral part of any stability assessment involving stage construction on soft constructed soils, such as are present beneath the breach zone.

Calculations involve the estimates of stresses on a structure, the magnitude of pore pressure reaction, and its subsequent dissipation with time as construction proceeds through the various stages to completion. The data obtained from consolidation testing (see Appendix E) are used to calculate the rate of pore pressure dissipation. Pore pressures dissipate as a result of water flow to drainage boundaries, and in the case of Upper GLU, dissipation will be enhanced by horizontal flow reflecting the laminated structure of the Upper GLU. Details of the pore pressure predictions for both one-dimensional (vertical only) and two-dimensional (vertical and horizontal) water flow are presented in Appendix H. In the latter case, some estimates of anisotropy of the flow parameters have also been made.



## 6 | Analysis of Breach Mechanics

The calculated values at the time of failure suggest that an average excess pore pressure of about 50 kPa might exist in the potential shear zone. This is a small percentage of the applied load and, if it does exist, is not particularly consequential. Moreover, in the experience of the Panel, laboratory tests tend to underestimate the coefficients of consolidation in place due to scale effects, and it is likely that the potential for lateral drainage in the analyses due to stratigraphic variations has been underestimated. The Panel concludes that the loading path to failure has been essentially drained with transient episodes of undrained loading. The small peak of pore pressure development beneath the crest of the embankment in 2014 may have had some impact on the ultimate trigger, as the embankment was close to failure at this time.

Loading the Upper GLU to failure under predominantly drained conditions also implies the imposition of shear stresses as well as vertical stresses. As shown in Appendix E, Attachment 5, consolidating specimens under a shear stress not only has an effect on available resistance, but also reduces the subsequent tolerable strain to failure. Given the high stresses that acted on the Upper GLU prior to the final construction campaign in 2014, it would have taken only a small additional load to initiate undrained failure, and little incremental deformation. This is consistent with the collapse of the embankment without any apparent warning.

**Given the high stresses that acted on the Upper GLU prior to the final construction campaign in 2014, it would have taken only a small additional load to initiate undrained failure, and little incremental deformation. This is consistent with the collapse of the embankment without any apparent warning.**

## 6 | Analysis of Breach Mechanics

### 6.3.3 PROGRESSIVE FAILURE

The strength at failure will only be mobilized uniformly if it does not vary with deformation. Failure will begin initially at a position where the local stresses equal the strength. As additional load is applied, yielding spreads to adjacent locations because resistance is limited at locations that have already yielded. This spreading of the yield zone migrates until a failure mechanism develops and unrestrained movement occurs with mobilization of a uniform shear strength.

However, as emphasized in section 5.3, the Upper GLU exhibits strain-weakening behaviour. After yielding has been initiated, the local resistance reduces with increasing load, requiring stress transfer to accommodate not only the influence of additional externally applied load, but also the influence of the reduced capacity of already failed material to resist the applied stresses. The transfer process proceeds to ultimate failure, but the average resistance at ultimate failure is less than the peak resistance.

This process is known as progressive failure. Once progressive failure has been initiated, the development of ultimate collapse can be sudden, depending on the shape of the whole stress-strain relation. As noted in section 6.2.3, the observed sensitivity of the Upper GLU indicates that it might display an ultimate resistance of one-half to one-third of its peak value.

While the mechanics of progressive failure are generally understood, the ability to calculate it is a complex undertaking and is generally reserved for research endeavours or other special studies. Progressive failure analyses have not been undertaken in this study, but Lobbestael et al. (2013) provide a useful overview and example of how progressive failure calculations might be performed. The Panel is of the view that progressive failure was involved in the initiation of collapse of the Perimeter Embankment and subsequent motion. Its influence is embedded in the back-calculated average resistance.

# 6 | Analysis of Breach Mechanics

## 6.4 FAILURE MECHANISM

The Panel's Terms of Reference require it to: "report on the cause of the failure of the tailings storage facility at the Mount Polley Mine."

The failure of the tailings storage facility (TSF) was caused by deformation of the Perimeter Embankment that allowed the containment to be breached between survey stations 4+200 and 4+300. The deformation arose because of inadequate resistance of a continuous layer of glaciolacustrine clays (Upper GLU) that existed at about El. 920 m, beneath the overlying till. The GLU deposit had properties that became increasingly contractive when sheared, following consolidation under the applied embankment loads to a normally consolidated state. This made the Upper GLU disposed to undrained failure. Moreover, the Upper GLU exhibited strain-weakening properties when sheared, such that overall resistance of the formation reduced as deformation developed, ultimately overcoming all of the resistance of the stabilizing elements in the section. Hence, the root cause of the breach was the undrained failure of the Upper GLU under the imposed load of the Perimeter Embankment on August 4, 2014.

**The root cause of the breach was the undrained failure of the Upper GLU under the imposed load of the Perimeter Embankment on August 4, 2014.**

# 6 | Analysis of Breach Mechanics

## 6.5 CAUSES OF FAILURE

As outlined in the Terms of Reference, it is expected that the Panel will “identify any technical, management, or other practices that may have enabled or contributed to the mechanism(s) of failure. This may include design, construction, maintenance surveillance and regulation of the facility.”

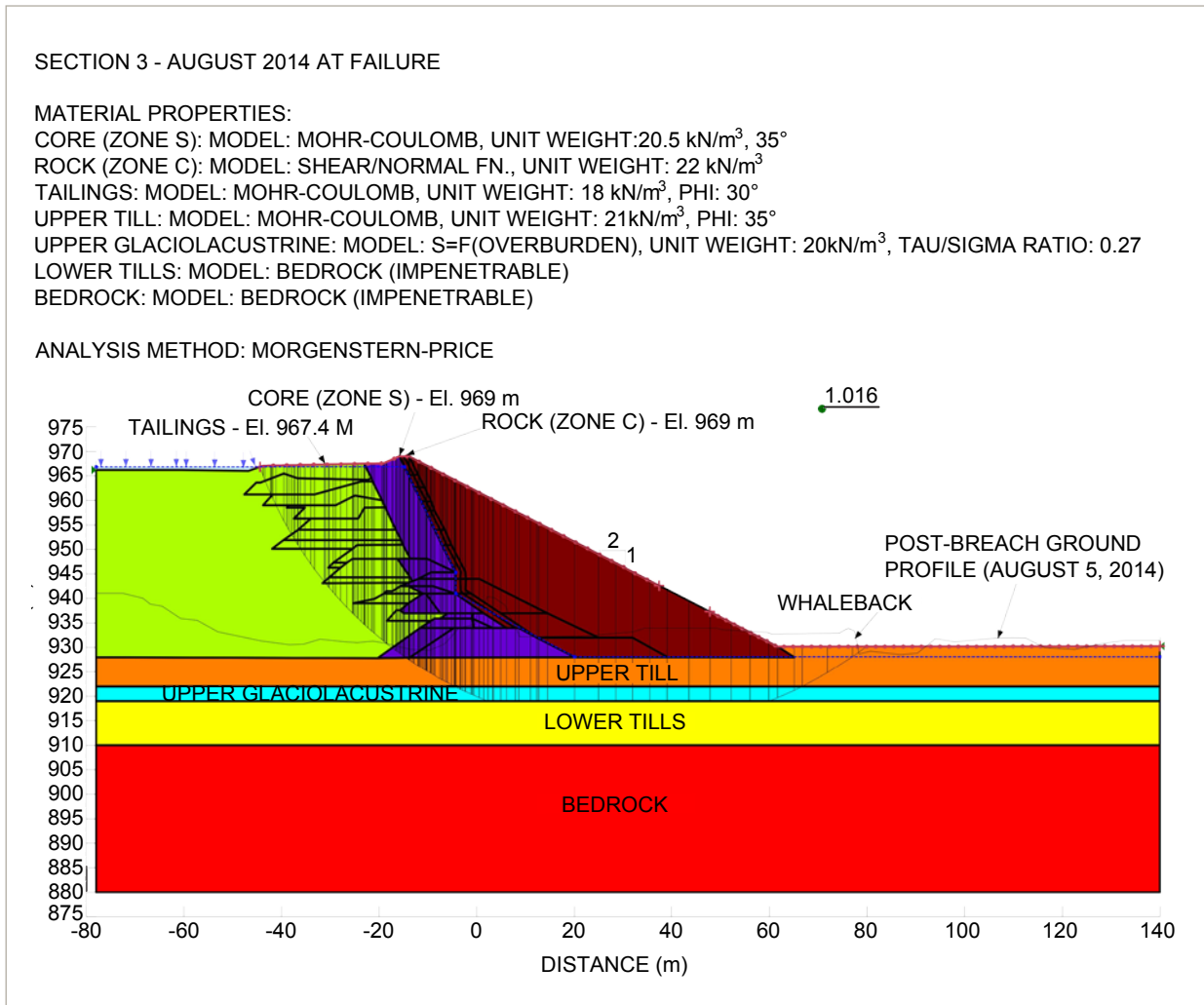
The dominant contribution to the failure resides in its design. The design did not take into account the complexity of the sub-glacial and pre-glacial geological environment associated with the Perimeter Embankment foundation. As a result, foundation investigations and associated site characterization failed to identify a continuous GLU layer in the vicinity of the breach and to recognize that it would be disposed to undrained failure when subjected to the stresses associated with the Dam.

**The design did not take into account the complexity of the sub-glacial and pre-glacial geological environment associated with the Perimeter Embankment foundation.**

At the time of Stage 4 (2006 – 2007), Knight Piésold (KP) had proposed a design for the Perimeter Embankment with a 2H:1V downstream slope and raises of the core and filter with a parallel inclined alignment to El. 965 m. This design has been projected in **Figure 6.5.1** to the core elevation at the time of failure (El. 969 m), and adopting an undrained strength ratio of 0.27 and a high water table, the calculated FS is 1.02. At El. 965 m, the FS is 1.04, much less than the design target of 1.3. Based on the back-calculated undrained strength ratio, the design was doomed to fail.

# 6 | Analysis of Breach Mechanics

**FIGURE 6.5.1: 2-D LIMIT EQUILIBRIUM 2H:1V SLOPE TO ELEVATION 969 M (UNDRAINED STRENGTH RATIO 0.27, HIGH WATER TABLE, FS 1.02)**



Hence, the omissions associated with site characterization may be likened to creating a loaded gun. Notwithstanding the large number of experienced geotechnical engineers associated with the TSF over the years, the existence of this loaded gun remained undetected.

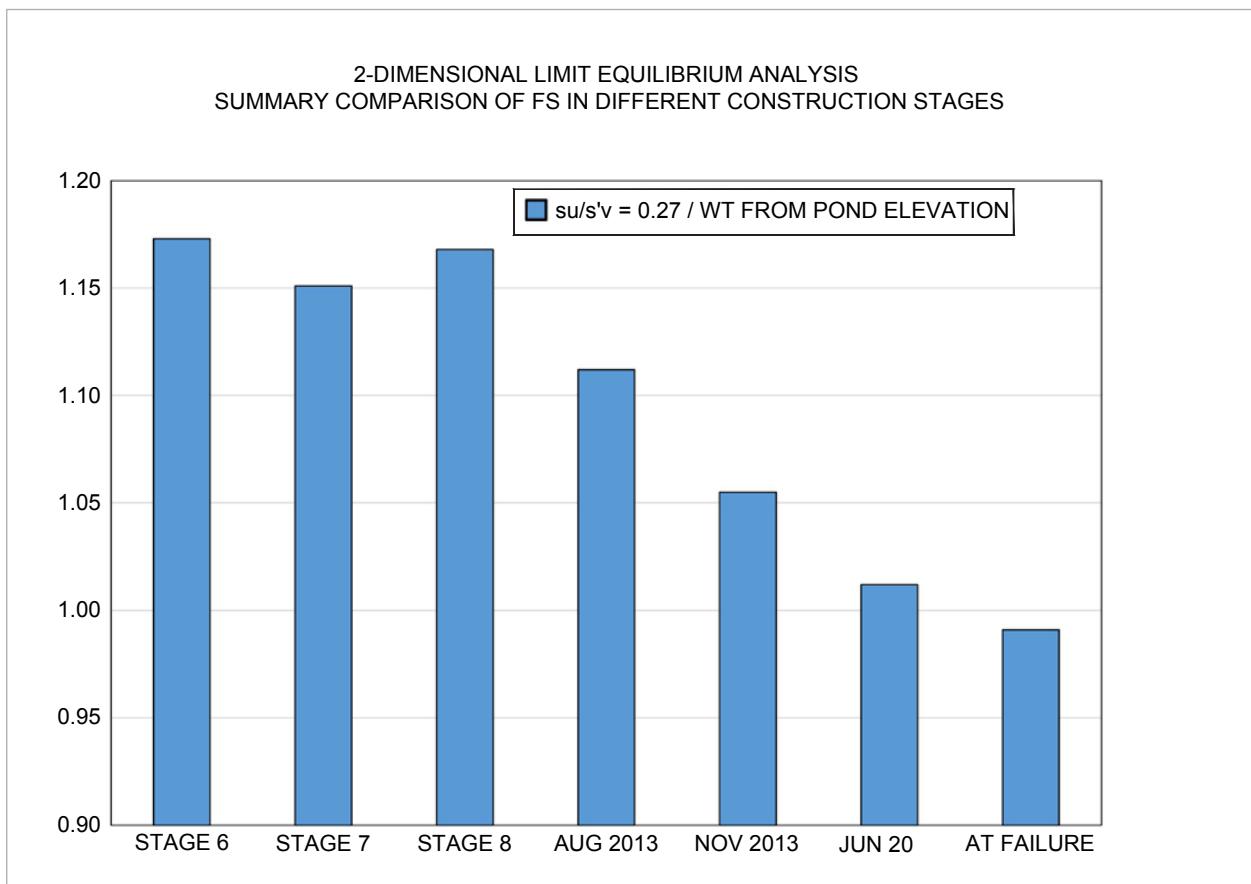
**The omissions associated with site characterization may be likened to creating a loaded gun.**

## 6 | Analysis of Breach Mechanics

The lack of recognition of a critical potential failure mode resulted in a misapplication of the Observational Method and, therefore, a false appreciation that the structure was performing as intended during stages of raising. The Observational Method is a powerful tool to manage uncertainty in geotechnical practice. However, it relies on recognition of the potential failure modes, an acceptable design to deal with them, and practical contingency plans to execute in the event observations lead to conditions that require mitigation. The lack of recognition of the critical undrained failure mode that prevailed reduced the Observational Method to mere trial and error.

**Figure 6.5.2** shows the variation of the calculated FS with each stage, from Stage 6 to failure, based on the as-built section for each stage. El. 965 m corresponds approximately to the height of the structure at the end of the 2013 construction season. At this stage, the FS is calculated to be only about 1.05, which is similar to the FS for the original design with a 2H:1V slope.

**FIGURE 6.5.2: VARIATION IN FS FOR EACH STAGE FROM STAGE 6 TO FAILURE**



## 6 | Analysis of Breach Mechanics

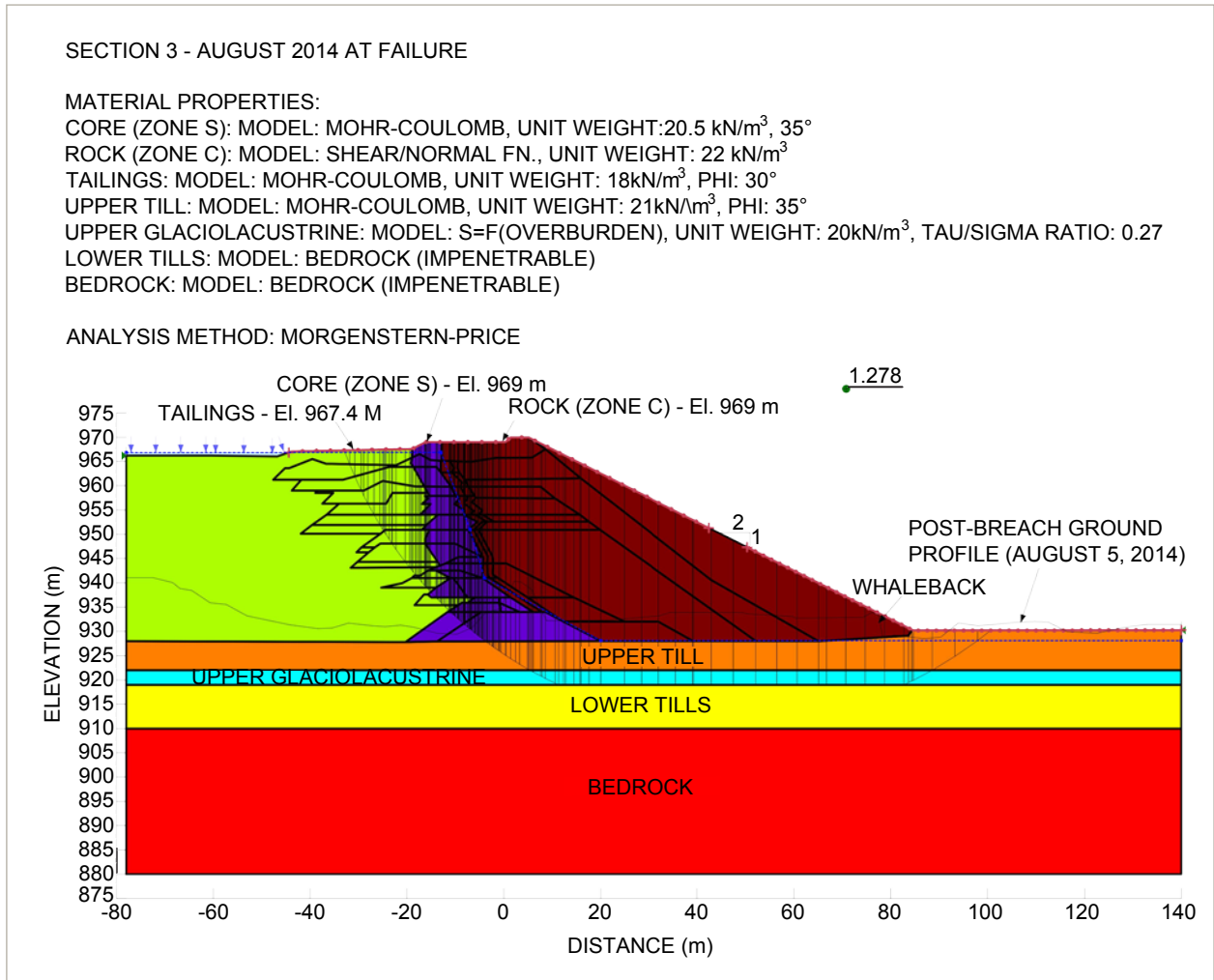
Prior to 2014, the Zone C fill began to be constructed as an angle of repose slope of 1.3H:1V. This appeared to have been an expedient measure and, as illustrated in **Figure 6.5.2**, ultimately resulted in failure of the Perimeter Embankment on August 4, 2014. If constructing unknowingly on the Upper GLU stratum, and not recognizing the potential undrained failure constituted loading the gun, building with a 1.3H:1V angle of repose slope over this stratum pulled the trigger. It appears that the 1.3H:1V

slope began as an expedient temporary measure to facilitate construction during Stage 5. It became more or less permanent for subsequent phases, although concerns had been raised before the failure. The circumstances associated with the relative permanency of the 1.3H:1V slope are not well understood by the Panel. The complex issues that prevailed during construction are summarized in section 5.4. **Figure 6.5.3** indicates that, had the downstream slope incorporating the widened crest been flattened to 2H:1V, the FS would have been 1.28. This was close to the required value of 1.3, and the embankment would not have failed. Moreover, the slope of 2H:1V was required, in any case, to support reclamation and closure criteria.

**If constructing unknowingly on the Upper GLU...constituted loading the gun, building with a 1.3H:1V angle of repose slope over this stratum pulled the trigger.**

# 6 | Analysis of Breach Mechanics

**FIGURE 6.5.3: PANEL'S STABILITY ANALYSIS FOR 2H:1V SLOPE WITH WIDENED CREST (FS = 1.28)**





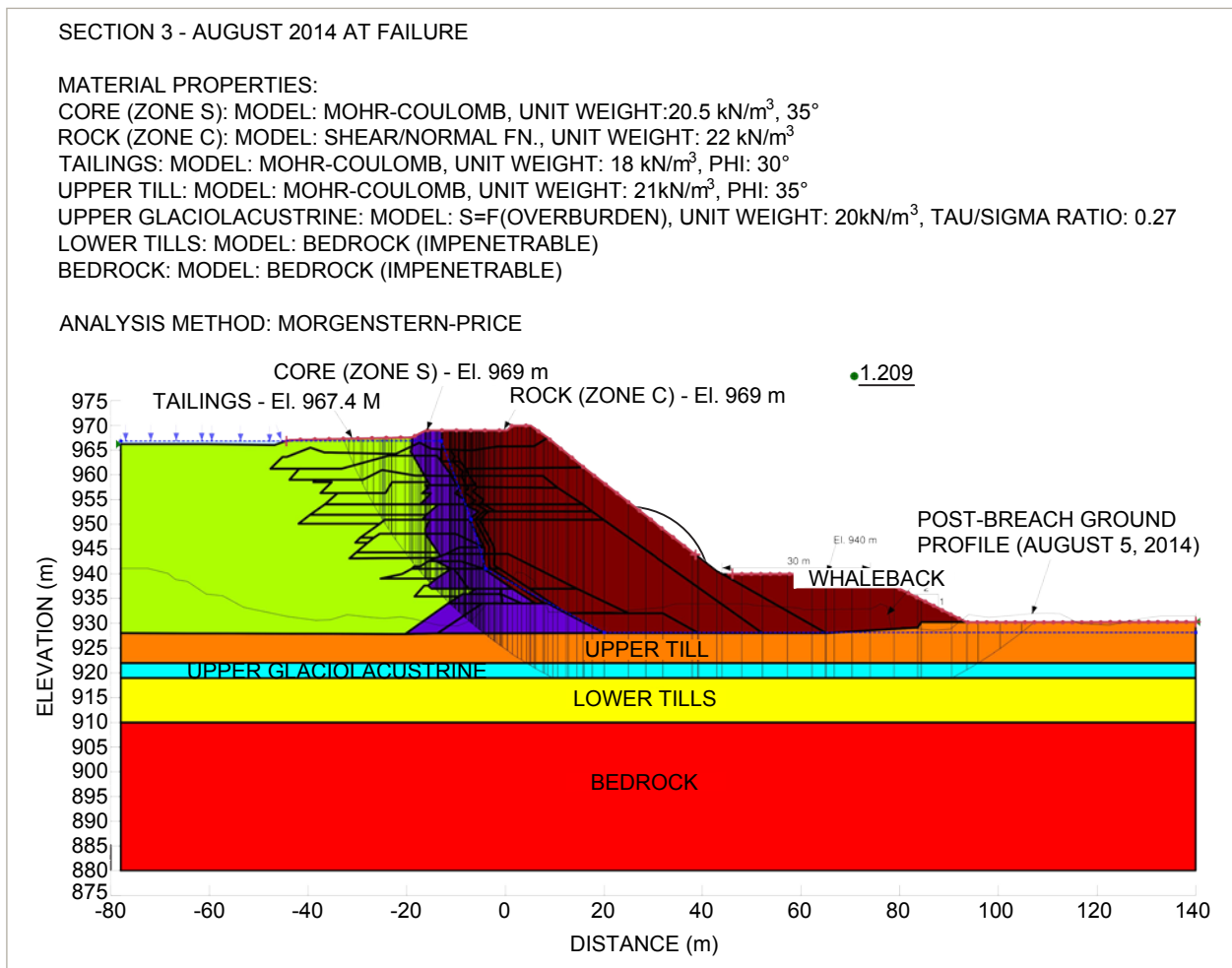
# 6 | Analysis of Breach Mechanics

## 6.6 PREVENTION OF FAILURE

The Terms of Reference (TOR) authorize the Panel to comment on “what actions could have been taken to prevent this failure.”

Looking specifically at the failure as documented in section 5.4, it was deemed desirable to increase the target FS to 1.5 since the TSF was operating more or less continually at full capacity. No significant progress to this end was made in Stage 9 before failure occurred. BGC’s design report for Stage 10, issued on July 25, 2014, indicated the buttress required to meet the new design objectives that they identified. Had it been in place as shown on **Figure 6.6.1**, the FS would have been 1.2 and the failure would have been prevented.

**FIGURE 6.6.1: PANEL STABILITY ANALYSIS FOR BGC BUTTRESS ON STAGE 9**



## 7 | Management Practices

The Panel is cognizant that management practices have had a significant influence on the design, construction and operation of the tailings storage facility (TSF). For example, the Panel has already drawn attention to water balance protocols and the growth of water inventory in the TSF due to the timing associated with the implementation of water treatment and discharge. It has pointed out that the recurrent adoption of a 1.3H:1V downstream slope for the Perimeter Embankment may have been due to limited material availability or other aspects related to mine planning. The details are not clear. What is clear is that multiple changes were made in the section of the dam in response to the limited time horizons adopted in mine and water planning.

The Panel has been advised that Mount Polley Mining Corporation (MPMC) were in the midst of becoming Mining Association of Canada (MAC) compliant and that tailings management issues were reported to the Board of Directors. It has not identified any flaws in this reporting structure.

However, in conducting its inquiry, the Panel limited itself to relying on interviews and on the documents that it received from the various stakeholders, which were sufficient to determine root cause of the breach. The Panel did not conduct its process according to formal legal procedures. To do so would have extended the length of this investigation and would have entered into an assessment of roles and responsibilities, which is beyond the Panel's authorization. As a result, the Panel is not able to offer an adequate assessment of the role of management and oversight in its contribution to the cause of the failure. In particular, the Panel has not explored the relationship between the designers and owner, contractual or otherwise. Accordingly, the Panel is unable to ascertain the circumstances that contributed to key decisions.

## 8 | Regulatory Oversight

### 8.1 ROLES AND RESPONSIBILITIES

This section describes the regulatory roles and responsibilities for impoundments and diversions at mines in B.C. A Memorandum of Understanding (MOU) is in place between the Ministry of Energy and Mines (MEM), the Ministry of Forests, Lands and Natural Resource Operations (MFLNRO) and the Ministry of the Environment (MoE) to clarify the regulation of these facilities. This MOU and other documents related to Mine Tailings are available on the Geotechnical page of the MEM website:

<http://www.empr.gov.bc.ca/MINING/PERMITTING-RECLAMATION/GEOTECH/Pages/default.aspx>

The MOU clearly places the responsibility for the engineering aspects of the Mount Polley tailings storage facility (TSF), seepage collection ponds and diversions on the shoulders of MEM, while the water quality of any discharges is the responsibility of MoE. Two permits are in place for the TSF and associated facilities: Permit M-200 from MEM and Permit 11678 from MoE.

MEM permits are issued by the Chief Inspector of Mines of B.C. The Manager of Geotechnical Engineering and the Manager Environmental report to the Deputy Chief Inspector of Mines, Permitting. The Manager Geotechnical Engineering has a staff of two geotechnical engineers and one reclamation specialist, while the Manager Environmental has a staff of three geoscientists. This staff of eight is responsible for inspection of operating mines and permitting of new mines in B.C. Apart from TSF-related activities, they also have regulatory responsibility for open pits, underground workings, and mined rock and overburden piles. The Geotechnical Manager and staff also participate in secondary activities including the Canadian Dam Association (CDA) Regulatory Committee and coordination with the Association of Professional Engineers and Geoscientists of British Columbia (APEGBC) in development of the *Professional Practice Guidelines for Dam Safety Reviews* for mining dams in B.C.<sup>1</sup> The latter publication is available on the above-mentioned website.

The ongoing activities of the geotechnical staff include:

- Review of geotechnical aspects of proposed mining projects in the Environmental Assessment process.
- Review of geotechnical aspects of *Mines Act* Permit applications during the approval and permit conditions development process.
- Review of permit amendment applications for dam raises, mine expansions, etc.
- Geotechnical site inspections of operating and closed mines.
- Review of geotechnical reports submitted under the Code, including annual dam safety inspections for mining dams and diversions.

## 8 | Regulatory Oversight

Filling the positions at MEM has been challenging at times. The present Manager of Geotechnical Engineering joined MEM in October 2011 following a period of over 3 years when the position was open. A senior geotechnical position was made redundant in 2003, but a new geotechnical inspector position was created in 2007. This position was also vacant for about 2 years until filled in September 2012. A third position was posted in May 2014 and filled in early October 2014. To attract qualified personnel, MEM has to compete with industry salaries, which is a challenge, especially during a booming mining cycle. To help accomplish these tasks, MEM has appointed four consulting professional engineers as Contract Inspectors to inspect tailings dams and other mining facilities.

An annual inspection schedule is developed for all the inspectors. The target is to inspect about 30 mines on an annual basis. Mount Polley is one of these mines.

## 8 | Regulatory Oversight

### 8.2 REGULATORY INTERACTIONS RELATED TO MOUNT POLLEY MINING CORPORATION (MPMC) TAILINGS STORAGE FACILITY (TSF)

**Table 8.2.1** lists the dates of the geotechnical inspections completed at Mount Polley from 1995 to 2014. Annual inspections were completed during the Phase 1 operations and were resumed after start-up of Phase 2 operations until 2008. There were no geotechnical inspections during 2009, 2010 and 2011, which is the same period as the vacancy of the Geotechnical Manager’s position.

**TABLE 8.2.1: GEOTECHNICAL INSPECTIONS AT MOUNT POLLEY**

DATE OF INSPECTION	TYPE OF INSPECTION PERFORMED	INSPECTOR
Sept 20, 1995	Geotechnical	G. Headley
Oct 19, 1995	Geotechnical	G. Headley
Oct 19, 1995	Geotechnical	C. Brawner
July 9 and 13, 1996	Geotechnical	G. Headley
Aug 26, 1996	Geotechnical	G. Headley
Sep 27–28, 1996	Geotechnical	G. Headley
May 27, 1997	Geotechnical	G. Headley
Jun 4, 1998	Geotechnical	G. Headley
Jun 17, 1999	Geotechnical	G. Headley
Aug 17, 2000	Geotechnical	G. Headley
April 25, 2001	Geotechnical	C. Carr
Feb 3, 2005	Geotechnical	C. Carr
Oct 13, 2005	Geotechnical	N. Rose
Aug 30, 2006	Geotechnical	N. Rose
July 31, 2007	Geotechnical	N. Rose
Jun 7, 2008	Geotechnical	D. Apel
Apr 12, 2012	Geotechnical – site visit	G. Warnock
Sept 24, 2012	Geotechnical	M. Cullen
Sept 13, 2013	Geotechnical	M. Cullen
Dec 4, 2014	Geotechnical	M. Cullen

## 8 | Regulatory Oversight

Most of the inspection reports did not identify any concerns with the TSF, except in the following cases. Based on the inspection of April 25, 2001 the inspector observed: *“The Ministry would strongly support the installation of two slope inclinometers at the downstream toe buttress to monitor potential dam and/or foundation movement. The slope inclinometers should extend through the underlying glaciolacustrine sediments.”*<sup>2</sup> MPMC responded that this matter was forwarded to Knight Piésold (KP).<sup>3</sup> These inclinometers were installed in July 2001 (refer to Appendix F).

On October 13, 2005,<sup>4</sup> narrow beach widths were observed on the southwest side of the pond. On August 30, 2006,<sup>5</sup> wide beach widths were observed and MEM requested a specific specification for beach width. MPMC responded, quoting KP:<sup>6</sup> *“The tailings embankments have been designed to remain stable for any condition and therefore there is not a ‘requirement’ for a minimum beach width in terms of embankment performance.”*

On July 31, 2007,<sup>7</sup> the inspection found two concerns that were Departures from Approval. First, Zone 5 material contained particles as large as 12 inches, which had to be removed to satisfy the specification of 4 inches. In addition, there was no beach in the vicinity of the southeast corner and MPMC was told that the beach must be re-established and that more frequent monitoring of the piezometers must be conducted in that area.

While the examples above illustrate the role of the Regulator in matters of construction and performance, the Regulator also reviewed design. The following design-related issues were brought up by the Regulator:

- The shear strength associated with the lacustrine materials in the well log GW96-1A.<sup>8</sup>
- Testing on residual strength.<sup>9,10</sup>
- Need to migrate factor of safety (FS) from 1.3 to 1.5.<sup>11</sup>

In each case, the Engineer of Record (EOR) responded to the inquiries and these instances illustrate the limited ability of the Regulator to influence the design issues.

## 8 | Regulatory Oversight

### 8.3 PANEL ASSESSMENT

The roles and responsibilities of the Ministry of Energy and Mines (MEM) to regulate impoundments and diversions at mines are well defined and agreed upon with other Ministries. Within MEM, the roles and responsibilities of the geotechnical engineering group responsible for regulating the design, construction and operational aspects of TSFs are also clearly defined. This small group of professionals covers a large portfolio of existing facilities, permitting of new facilities and environmental assessments for proposed projects.

The Panel finds that the MEM Geotechnical Staff and the Contract Inspectors are well qualified to perform their responsibilities. The team is well organized and has clear targets and schedules for annual inspections. The Panel considers the technical qualifications of the MEM Geotechnical Staff as among the best that it has encountered among agencies with similar duties.

MEM geotechnical engineers addressed significant issues during the reviews and inspections of the Mount Polley TSF. They had insightful questions for the designers at many instances during their review of the design documents, as noted above. The EOR responded to these questions based on their observations and understanding of site conditions. The EOR is responsible for the overall performance of the structure as well as the interpretation of site conditions. The Regulator has to rely on the expertise and the professionalism of the EOR as the Regulator is not the designer.

Despite having a strong regulatory process and personnel, the Perimeter Embankment of the Mount Polley TSF still failed. As indicated in earlier sections, it was a sudden failure without precursors. Additional inspections of the TSF would not have prevented the failure.

However, the question remains as to the expectations from the Regulator in the future. The relationship between the Regulator and the EOR can result in different opinions being expressed that are not easy to resolve without independent input. In such circumstances, independent external advice could be sought as further described in section 9.0. There is a difference between regulating construction and regulating design after it has been approved. The Regulator by observation and experience has the capacity to regulate construction but does not have the capacity to modify the design. Regulators are not normally recruited with specific dam design experience and are limited by statute in their capacity to take on design responsibilities. This role resides with the EOR.

## 8 | Regulatory Oversight

It is difficult to review the adequacy of a constructed facility without having limits of measurable indicators that define its performance. Measurable indicators of safe and orderly design and construction are needed for all existing and future tailings facilities that can be monitored and interpreted to evaluate this performance. Section 9.0 provides further elaboration of Quantitative Performance Objectives (QPOs) as a means of accomplishing this.

### ENDNOTES

- 1) Panel Interview, 12/12/14.
- 2) MP00170
- 3) MP00218
- 4) MP00174
- 5) MP00175
- 6) MP00216
- 7) MP00177
- 8) MO00137
- 9) MP00139
- 10) MO00222
- 11) MP00187



# 9 | Where Do We Go From Here?

## 9.1 PERFORMANCE OF B.C. TAILINGS DAMS

Central to the Panel's Terms of Reference (Appendix A) is to recommend actions for preventing future tailings dam failures:

*"... the Panel may make recommendations to government on actions that could be taken to ensure that a similar failure does not occur at other mine sites in B.C."*

Fulfilling this mandate starts by considering the tailings dams that currently exist in the province. In particular, this involves how many there are and how they have performed. Appendix I describes the Panel's efforts in this respect. It found that there are currently 123 active tailings dams, those that contain surface water in their impoundments along with tailings.

Active tailings dams were tracked through the years from Ministry of Energy and Mines (MEM) records. In the 46-year period since 1969, there was a total of 4,095 years of active operation and 7 failures, where failure is considered to be breach of the dam resulting in release of tailings and/or water. This corresponds to a failure frequency of  $1.7 \times 10^{-3}$  per dam per year. In other words, statistically there is approximately a 1-in-600 chance of a tailings dam failure in any given year, based on historical performance over the period of record.

While these numbers may seem small, their implications are not. If the inventory of active tailings dams in the province remains unchanged, and performance in the future reflects that in the past, then on average there will be two failures every 10 years and six every 30. In the face of these prospects, the Panel firmly rejects any notion that business as usual can continue.

**The Panel firmly rejects any notion that business as usual can continue.**

# 9 | Where Do We Go From Here?

## 9.2 GETTING TO ZERO

In risk-based dam safety practice for conventional water dams, some particular level of tolerable risk is often specified that, in turn, implies some tolerable failure rate. The Panel does not accept the concept of a tolerable failure rate for tailings dams. To do so, no matter how small, would institutionalize failure. First Nations will not accept this, the public will not permit it, government will not allow it, and the mining industry will not survive it.

Clearly, improvements to current practice provide an essential starting point on the path to zero failures. But the Panel's evaluation of portfolio risk shows that incremental changes will not be sufficient to achieve this objective.

Appendix I explains why. Ultimately, the problem stems from how many active tailings dams there are in the province. To ensure against future failures for all of them would require roughly a hundredfold reduction or more in the current failure frequency. While advances in practices, procedures and policies are imperative, the Panel does not expect these measures by themselves to achieve this degree of improvement. The path to zero needs an added dimension, and that dimension is technology.

**The path to zero needs an added dimension, and that dimension is technology.**

Tailings dams are complex systems that have evolved over the years. They are also unforgiving systems, in terms of the number of things that have to go right. Their reliability is contingent on consistently flawless execution in planning, in subsurface investigation, in analysis and design, in construction quality, in operational diligence, in monitoring, in regulatory actions, and in risk management at every level. All of these activities are subject to human error.

Human error is often, if not always, found to play a key role in technological failures. And human error will always be with us, as much as we might wish it to be otherwise. This is why failures invariably bring about improvements in technology that help compensate for human error. In perhaps the most notorious containment failure, double-hulled tankers were mandated after the *Exxon Valdez* oil spill. Similarly, improvements to rail tank cars are being adopted in the wake of the Lac-Mégantic tragedy. But tailings dams have no such redundancies. Without exception, dam breaches produce tailings releases. This is why best practices can only go so far in improving the safety of tailings technology that has not fundamentally changed in the past hundred years.

**Failures invariably bring about improvements in technology that help compensate for human error.**

## 9 | Where Do We Go From Here?

Improving technology to ensure against failures requires eliminating water both on and in the tailings: water on the surface, and water contained in the interparticle voids. Only this can provide the kind of failsafe redundancy that prevents releases no matter what. In terms of portfolio risk, Appendix I shows that this works by reducing the inventory of active tailings dams subject to failure in the first place. Simply put, dam failures are reduced by reducing the number of dams that can fail.

Thus, the path to zero leads to best practices, then continues on to best technology.

# 9 | Where Do We Go From Here?

## 9.3 BEST AVAILABLE TAILINGS TECHNOLOGY

### 9.3.1 BAT PRINCIPLES

While best practices focus on the performance of the tailings dam, best available technology (BAT) concerns the tailings deposit itself. The goal of BAT for tailings management is to assure physical stability of the tailings deposit. This is achieved by preventing release of impoundment contents, independent of the integrity of any containment structures. In accomplishing this objective, BAT has three components that derive from first principles of soil mechanics:

- 1. Eliminate surface water from the impoundment.**
- 2. Promote unsaturated conditions in the tailings with drainage provisions.**
- 3. Achieve dilatant conditions throughout the tailings deposit by compaction.**

The first of these, eliminating surface water, not only precludes release of water itself, but also eliminates fluvial tailings transport mechanisms like those illustrated in Appendix C during the Mount Polley breach. The second, promoting unsaturated conditions by drainage, reduces the possibility for, and the quantity of, high-mobility flowslide release of tailings. And the third, achieving dilatant conditions by compaction, further reduces flowslide potential by improving the properties of the tailings mass. Thus, underpinning these principles are multiple redundancies that provide defence in depth.

The Panel recognizes that eliminating water from the tailings deposit will not eliminate the need for storage of mine and processing water elsewhere. But Mount Polley has shown the intrinsic hazards associated with dual-purpose impoundments storing both water and tailings. The Panel considers that security can be more readily assured for conventional water dams that are designed and constructed for their own purpose and that preventing tailings release is the overriding imperative.

## 9 | Where Do We Go From Here?

### 9.3.2 BAT METHODS

The overarching goal of BAT is to reduce the number of tailings dams subject to failure. This can be achieved most directly by storing the majority of the tailings below ground—in mined-out pits for surface mining operations or as backfill for underground mines. Both methods require integrating tailings planning into

mine planning. This has not been common practice in the industry to date, as the Mount Polley case has shown, and the synergies to be achieved are mostly unexplored. Apart from this, surface storage using filtered tailings technology is a prime candidate for BAT.

**The overarching goal of BAT is to reduce the number of tailings dams subject to failure.**

Demonstrated technology for producing and placing filtered tailings (sometimes termed “dry stack” tailings) is well-known in the industry. Its adoption and design practices are documented in the literature.<sup>1,2</sup> Using various kinds of equipment, the water content of the tailings is reduced before they leave the mill. The specified degree of water removal can vary, but is sufficient to allow transport by truck or conveyor to the tailings facility and compaction. Compaction is necessary to prevent liquefaction flowslides that can and have occurred in loosely placed dewatered materials due to infiltration of ponded surface runoff. The Panel recognizes that creating dry tailings may increase the amount of water requiring treatment or storage.

Filtered tailings technology embodies all three BAT components described in section 9.3.1. Most commonly used in dry climates where economy in water consumption is important, it has also been adapted to cold regions.<sup>3</sup> This method has been used since start-up of the Greens Creek mine in Alaska under conditions not unlike coastal B.C.<sup>4</sup> The Greens Creek facility is shown in **Figure 9.1.1**.

Variations on this technology are easily envisioned, for example separation, dewatering, and gravity drainage of sand tailings by cycloning to reduce quantities requiring filtration dewatering. The Panel believes that additional enhancements are ripe for development if there is incentive to do so.

In some cases, clayey ore may pose difficulties in dewatering. And most filtered tailings operations to date have been relatively small. But some new operations will be producing filtered tailings at a rate of 68,000 tonnes per day—almost three times the production of Mount Polley—in facilities that will reach heights of 150 metres (m). As demonstrated by the Greens Creek case and others, there are no overriding technical impediments to more widespread adoption of filtered tailings technology.

**There are no overriding technical impediments to more widespread adoption of filtered tailings technology.**

## 9 | Where Do We Go From Here?

**FIGURE 9.1.1 FILTERED TAILINGS FACILITY, GREENS CREEK, ALASKA**



The chief reason for the limited industry adoption of filtered tailings to date is economic. Comparisons of capital and operating costs alone invariably favour conventional methods. But this takes a limited view. Cost estimates for conventional tailings dams do not include the risk costs, either direct or indirect, associated with failure potential. The Mount Polley case underscores the magnitude of direct costs for cleanup, but indirect losses—notably in market capitalization—can be even larger.<sup>5</sup> Nor do standard costing procedures consider externalities, like added costs that accrue to the industry as a whole, some of them difficult or impossible to quantify. Full consideration of life cycle costs including closure, environmental liabilities, and other externalities will provide a more complete economic picture. While economic factors cannot be neglected, neither can they continue to pre-empt best technology.

**While economic factors cannot be neglected, neither can they continue to pre-empt best technology.**

## 9 | Where Do We Go From Here?

### 9.3.3 BAT FOR CLOSURE

Closure of tailings deposits is subject to two fundamental considerations: physical stability and chemical stability. Although the former is the object of the Panel's investigation, no treatment of tailings technology can ignore the latter. Matters related to physical and chemical stability reside in different domains and have developed independently, each with their own goals and methods. These two aspects converge in the context of BAT.

In short, the most serious chemical stability problem concerns tailings that contain sulfide minerals, particularly in metal and coal mining. In the presence of oxygen, these sulfides react to produce acid that then mobilizes a variety of metals in solution. There are a number of ways to arrest this reaction, and one is to saturate the tailings so that water replaces oxygen in the void spaces. This saturation is most conveniently achieved by maintaining water over the surface of the tailings. Hence, so-called water covers have sometimes been adopted for reactive tailings during operation and for closure.

It can be quickly recognized that water covers run counter to the BAT principles defined in section 9.3.1. But the Mount Polley failure shows why physical stability must remain foremost and cannot be compromised. Although the tailings released at Mount Polley were not highly reactive, it is sobering to contemplate the chemical effects had they been. No method for achieving chemical stability can succeed without first ensuring physical stability: chemical stability requires above all else that the tailings stay in one place.

Filtered tailings technology adopts a different approach to chemical stability. Rather than arresting the reaction, it retards the transport of reaction products. Seepage gradients are greatly diminished by eliminating surface water. This has a beneficial effect not only on sulfide reaction products; it also equally reduces transport of soluble constituents such as arsenic, sulfates and selenium, if present in the tailings.

Moreover, the technology for alternative dry covers is well advanced. Using different cover designs for different climatic conditions, soil covers placed over the tailings deposit further reduce infiltration, retard oxygen entry, or both. Cover placement and reclamation can proceed concurrently with operation, as shown in the foreground in **Figure 9.1.1** at Greens Creek.

Yet other technologies attack the chemical effects of sulfide minerals by removing them from the tailings. Doing so using conventional metallurgical processes has been shown to be technically and economically feasible.<sup>6</sup> These same techniques can be used, in effect, to manufacture clean tailings cover material free from sulfides.<sup>7</sup>

This shows that the physical stability objectives of BAT are not incompatible with chemical stability. A variety of complementary technologies are available for achieving both.

## 9 | Where Do We Go From Here?

### 9.3.4 BAT RECOMMENDATIONS

Implementation of BAT is best carried out using a phased approach that applies differently to tailings impoundments in various stages of their life cycle.

- **For existing tailings impoundments.** Constructing filtered tailings facilities on existing conventional impoundments poses several technical hurdles. Chief among them is undrained shear failure in the underlying saturated tailings, similar to what caused the Mount Polley incident. Attempting to retrofit existing conventional tailings impoundments is therefore not recommended, with reliance instead on best practices during their remaining active life.
- **For new tailings facilities.** BAT should be actively encouraged for new tailings facilities at existing and proposed mines. Safety attributes should be evaluated separately from economic considerations, and cost should not be the determining factor.
- **For closure.** BAT principles should be applied to closure of active impoundments so that they are progressively removed from the inventory by attrition. Where applicable, alternatives to water covers should be aggressively pursued.

As discussed in section 9.2, best technology is only one of the two components necessary for safety improvement. The complementary aspects of best practices are presented in the following sections.



# 9 | Where Do We Go From Here?

## 9.4 BEST APPLICABLE PRACTICES (BAP)

The safety of any dam, water or tailings, relies on multiple levels of defence. The Panel was disconcerted to find that, while the Mount Polley Tailings Dam failed because of an undetected weakness in the foundation, it could have failed by overtopping, which it almost did in May 2014. Or it could have failed by internal erosion, for which some evidence was discovered. Clearly, multiple failure modes were in progress, and they differed mainly in how far they had progressed down their respective failure pathways.

Accordingly, recommendations for future BAP require considerations that go beyond stability calculations. It is important that safety be enhanced by providing for robust outcomes in dam design, construction and operations. As discussed below, this has implications for corporate responsibility, enhanced regulatory capacity, expanded technical review, and improvements in professional practice.

**It is important that safety be enhanced by providing for robust outcomes in dam design, construction and operations.**

### 9.4.1 CORPORATE GOVERNANCE

In response to several international tailings dam failure incidents in the 1990s, the Mining Association of Canada (MAC) established a task force in 1996 to promote safe, environmentally responsible management of tailings and mine waste. The task force concluded that the main priority should focus on improvement of tailings management, which resulted in the establishment of the MAC Tailings Working Group. The outcome of this initiative were several guides related to the management of tailings facilities; the development of operations, maintenance and surveillance manuals; and auditing and assessment of tailings management facilities.<sup>8</sup> The guides themselves are available from the MAC.<sup>9</sup> They are now embraced by the Towards Sustainable Mining (TSM) initiative launched by MAC in 2004.

Compliance with the TSM initiative is an element of BAP for the mining industry today. Accordingly, mining operations in B.C. proposing to operate a tailings storage facility (TSF) should either be required to be a member of MAC—ensuring adherence to the TSM—or be obliged to commit to an equivalent program, including the audit function. Tailings management is often not a core skill in many mining organizations. Embracing MAC's TSM initiative will ensure awareness of responsibilities at the highest corporate levels.

## 9 | Where Do We Go From Here?

At the same time, many in the industry have reacted to the Mount Polley failure with incredulity, asking how it could have happened with programs such as MAC's in place. This serves as a reminder that these programs should not instill a sense of overconfidence and cannot themselves be seen as a substitute for more fundamental changes in technology.

### 9.4.2 CORPORATE TSF DESIGN RESPONSIBILITIES

In the experience of the Panel, TSF design studies submitted to Regulators are often lacking in detail regarding the factors that need to be considered in assuring safety of the facility. This applies equally to appropriate tailings technology and to performance metrics for confirming orderly construction and operations.

At Mount Polley, the only quantitative performance objectives were those implied in its design criteria. A list of potential failure modes was compiled in the 2006 Dam Safety Report, but these were generic and not tied to specific site conditions. One of the lessons learned here is that future permit applications for TSFs must provide a more comprehensive assessment of potential geotechnical problems associated with the selected site. In addition, BAT for both tailings storage and closure considerations also needs to be incorporated in such proposals.

The Panel is of the view that the inclusion of these considerations and the declaration of Quantitative Performance Objectives (QPOs) are best incorporated early in project commitment at the bankable feasibility level. QPOs are intended to constrain the type of ad hoc design practices that characterized Mount Polley and strengthen regulatory capacity.

The Panel would require a bankable feasibility study and related permit application to have considered all technical, environmental, social and economic aspects of the project. Resolution of technical and environmental considerations would usually be supported by proven methods, although technology development studies would not be precluded if they have advanced far enough to warrant implementation in practice. The bankable feasibility study would be of sufficient detail to support an investment decision that might have an accuracy of  $\pm 10\%$ – $15\%$ .

## 9 | Where Do We Go From Here?

More explicitly, the bankable feasibility document would be required to contain the following:

- 1) A detailed evaluation of all potential failure modes associated with:
  - The geological conditions of the site
  - The uncertainties associated with this evaluation
  - The role of the Observational Method to manage residual risk
  - Mitigation measures in case worse than anticipated conditions are encountered.

This evaluation should be updated and incorporated into MEM requirements for annual inspection and construction review. This is to ensure that the evaluation would become a living document maintained throughout the life of the facility. It should be sufficiently well documented to survive changes in mine personnel, mine ownership or Engineers of Record (EORs), and it should be referenced as part of the Operations Maintenance and Surveillance (OMS) manual. The Panel anticipates that as-built reports would provide the basic information recording departures from what had been anticipated. An ongoing compilation should be maintained by the EOR as a separate document.

- 2) Detailed cost analyses of BAT tailings and closure options, so that alternative means of achieving BAT can be understood and accommodated. As discussed in section 9.3.2, this assessment should recognize that indirect and unquantifiable costs cannot be fully incorporated and hence the results of the cost analyses should not supersede BAT safety considerations.
- 3) A detailed declaration of QPOs, beyond those associated with regulatory compliance and ordinary design criteria. Examples of QPOs are numerical values and limits associated with:
  - Beach widths
  - Calibration of impoundment filling schedule
  - Water balance audits and calibration
  - Construction material availability and scheduling to ultimate height of structure
  - Instrumentation adequacy and reliability
  - Trigger levels for response to instrumentation
  - Performance data gathering, interpretation, and reporting intervals

## 9 | Where Do We Go From Here?

The Panel recognizes the need for a regulatory process that is responsive to changed conditions arising from market forces, reserves, regulatory revisions and technical issues. It is envisaged that such changes can be accommodated by staged approval for construction, as occurs at present. However, the stage applications should honour the declared QPOs or present a basis for their modification.

**The Panel recognizes the need for a regulatory process that is responsive to changed conditions arising from market forces, reserves, regulatory revisions and technical issues.**

### 9.4.3 INDEPENDENT TAILINGS REVIEW BOARD (ITRB)

The appointment of ITRBs to provide third-party advice on the design, construction, operation and closure has become increasingly common and is recognized to provide value.<sup>10</sup> The World Bank and other lenders groups are requiring the formation of an ITRB. International Finance Corporation/World Bank guidance and operating principles OP4.01 and OPR.37 establish the requirement to review the development of tailings dam design, construction and initial dam filling. Maintaining an ITRB through operations and closure will depend upon the scale and complexity of the facility. Some large corporations retain a third-party review board for ongoing advice on tailings operations to complement their internal technical audit systems.

**The appointment of ITRBs to provide third-party advice on the design, construction, operation and closure has become increasingly common and is recognized to provide value.**

ITRBs are not unique to the mining industry. They have a long history in water dam design and safety assessments. In British Columbia, BC Hydro has considerable experience with such Boards for safety assessment of both existing and new dam projects. In a mining context, an ITRB could be asked to provide opinions on the following:

- Whether the design, construction and operation of the TSF are consistent with satisfactory long-term performance.
- Whether design and construction have been performed in accordance with the Board's expectation of good practice.
- Whether safety and operation of the TSF conform to the Board's expectation of good practice.
- Whether there are weaknesses that would reasonably be expected to have a material adverse effect on the integrity of the TSF, human health, safety, and successful operation of the facility for its intended purpose.

## 9 | Where Do We Go From Here?

Experience has shown that the effectiveness of an ITRB in specific circumstances depends on the following:

- That it not be used exclusively as a means for obtaining regulatory approval.
- That it not be used for transfer of corporate liability by requesting indemnification from Board members.
- That it be free from external influence or conflict of interest.
- That there be means to assure that its recommendations are acted upon.

No ITRB can function successfully without unqualified support and commitment at the highest corporate levels. While it is essential that the Board be organized by Mine Operations, it is equally essential that its reports go to senior corporate management and Regulators. To establish and strengthen credibility, Board reports should also be open to other stakeholders. An important mechanism for accountability in response to Board recommendations is the creation of an Action Log that reviews corporate response to Board recommendations at each successive meeting.

**No ITRB can function successfully without unqualified support and commitment at the highest corporate levels.**

It is evident that the establishment of Independent Tailings Review Boards is an element of BAP, and the Panel is of the view that they have a role in improving current practice. But they should not be necessary for all tailings undertakings and MEM should consider, based on their current portfolio of operating and proposed TSFs, the conditions related to complexity and failure consequence that warrant an ITRB.

## 9 | Where Do We Go From Here?

### 9.4.4 MINISTRY OF ENERGY AND MINES (MEM)

As noted in section 7, the Panel was favourably impressed by the skill and commitment of MEM's geotechnical staff in carrying out their responsibilities. Nevertheless, it also considered what measures could be taken to improve regulatory operations.

With recent inspections of TSFs in the province in hand, the short-term need is to evaluate these facilities with respect to the following potential failure modes, in order of importance:

1. **Undrained shear failure** for dams with silt and clay foundation soils.
2. **Water balance adequacy**, including provisions and contingencies for wet years.
3. **Filter adequacy**, especially for dams containing broadly graded soils or mine waste.

One issue identified in section 8.0 is the ultimate reliance of the Regulator on the EOR to confirm that the facility is safe and is operating as intended. The Regulator is not the designer, and

**The Regulator is not the designer.**

this limits the degree of inquiry that is manageable. If Regulators were provided with more information in an ongoing manner, they would be better versed to engage the EOR. This is one of the benefits of having declared QPOs that can be monitored, as discussed in section 9.4.2. To this end, MEM should evaluate how to determine the QPOs associated with ongoing facilities and begin to apply them in practice.

Additionally, the Panel's compilation of the province's tailings dam inventory revealed limitations in MEM's capacity for information retrieval, especially for timely response to unexpected occurrences. Tailings dam data for each mine and each structure needs to be scanned electronically, compiled separately from permit files, and maintained in a readily accessible database.

# 9 | Where Do We Go From Here?

## 9.4.5 PROFESSIONAL PRACTICE

The Panel found it disconcerting that, notwithstanding the large number of experienced geotechnical engineers associated with the Mount Polley TSF, the overall adequacy of the site investigation and characterization of ground conditions beneath the Perimeter Embankment went unquestioned. This may reflect a regional issue, or possibly one of wider extent. Regardless, it calls for a concerted effort to improve professional practice in this area. The situation is reminiscent of the conditions that prevailed in B.C. that resulted in the Association of Professional Engineers and Geoscientists of British Columbia (APEGBC) Guidelines for *Legislated Landslide Assessment for Proposed Residential Developments in B.C.*

In the view of the Panel, the fundamental need is to improve the geological, geomorphological, hydrogeological and possibly seismotectonic understanding of sites proposed for tailings dams in B.C. This improved understanding should account for the likely scale associated with variability so that site investigations can be planned with enhanced reliability.

APEGBC appears to be well-suited for this task.

## 9.4.6 CANADIAN DAM ASSOCIATION (CDA) GUIDELINES

From its inception in 1995, the Mount Polley TSF adopted a minimum factor of safety (FS) of 1.3 during operations and 1.5 for closure. As chronicled in section 5.4, these FS criteria drove key decisions throughout the design process, and so the Panel is of the view that it would be helpful to comment on them.

CDA dam safety guidelines originally developed for water dams were subsequently adapted to tailings dams, with target factors of safety as indicated in **Table 9.4.1**.<sup>11</sup>

**TABLE 9.4.1 TARGET FACTORS OF SAFETY FOR SLOPE STABILITY IN CONSTRUCTION, OPERATION AND TRANSITION PHASES – STATIC ASSESSMENT (AFTER CDA, 2014)**

LOADING CONDITIONS	MINIMUM FACTOR OF SAFETY	SLOPE
During or at end of construction	>1.3 depending on risk assessment during construction	typically downstream
Long-term (steady state seepage, normal reservoir level)	1.5	downstream

## 9 | Where Do We Go From Here?

These 2014 guidelines vest responsibility for establishing appropriate FS criteria solely with the designer, subject to the designer's consideration of the following:

- The consequences of failure
- The loading conditions
- The strength parameters used

Hence, the CDA Guidelines are premised on proper evaluation of these factors. But in the case of Mount Polley, this premise was flawed. Few would argue that the failure consequences were anything less than catastrophic to those affected. The loading conditions did not account for the development of normally consolidated conditions in the foundation. And the strength parameters neglected undrained shearing. Furthermore, selection of FS criteria using risk analysis, as specified in **Table 9.4.1**, could not have succeeded because the operative failure mode went unrecognized.

Mount Polley illustrates that dam safety guidelines intended to be protective of public safety, environmental and cultural values cannot presume that the designer will act correctly in every case. To do so defeats the purpose of FS criteria as a safety net. In this, the CDA Guidelines are unable to achieve their intended purpose. Neither is the Province well served, to the extent that MEM has incorporated compliance with these guidelines as a statutory requirement.<sup>13</sup>

The Panel considers that tailings dam guidelines and criteria tailored to conditions in B.C. would more effectively meet the needs of the Province in protecting public safety. Those developed by the U.S. Army Corps of Engineers for water dams provide one example, among others, that might be used as a starting point.<sup>12</sup> This does not preclude adopting parts of the CDA Guidelines where appropriate as well as the CDA technical bulletin *Geotechnical Considerations for Dam Safety*.<sup>14</sup> The Panel anticipates that this will result in more prescriptive requirements for site investigation, failure mode recognition, selection of design properties, and specification of factors of safety.

**Tailings dam guidelines and criteria tailored to conditions in B.C. would more effectively meet the needs of the Province in protecting public safety.**



## 9 | Where Do We Go From Here?

### ENDNOTES

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# 10 | Conclusions

Based on the activities described and interpretations advanced in the preceding sections of the report, the Panel has developed the findings summarized below.

## 10.1 MECHANISM OF FAILURE

The breach of the Perimeter Embankment on August 4, 2014 was caused by shear failure of dam foundation materials when the loading imposed by the dam exceeded the capacity of these materials to sustain it. The failure occurred rapidly and without precursors.

Direct evidence of this failure mechanism is provided by an identified shear surface in surviving remnants of the dam core and by deformations consistent with shearing in a weaker glacially-deposited layer of silt and clay about 8–10 metres (m) below the original ground surface. This layer, its properties, and its extent received intense scrutiny during this investigation, and analyses using representative parameters provide indirect evidence that further supports this failure mechanism.

Deposited in a complex geologic environment, the weaker glaciolacustrine layer was localized to the breach area. It went undetected, in part because the subsurface investigations were not tailored to the degree of this complexity. But neither was it ever targeted for investigation because the nature of its strength behaviour was not appreciated.

Throughout, the design investigations took note of the stiff, dense character of foundation soils and used corresponding strength properties in stability analyses. But it was not recognized that this character would change, with a corresponding change in strength behaviour under the increased loading as the dam grew higher. Specifically, it was never recognized that the glaciolacustrine soils that were initially overconsolidated would become normally consolidated, requiring undrained shear strengths for stability analyses. This is the process that affected the weaker glaciolacustrine layer in the breach area that was not accounted for in the design of the dam.

Adding to the antecedent foundation conditions was the unprecedented steepness of the 1.3H:1V Perimeter Embankment slope. This was justified by design analyses without questioning its reasonableness. The higher Main Embankment had glaciolacustrine foundation soils with properties broadly comparable to those at the breach section. But here, the steep slopes were effectively flattened by the addition of a buttress, which explains why the failure did not occur at the highest part of the dam.

# 10 | Conclusions

## 10.2 CONTRIBUTING FACTORS

### 10.2.1 LONG-TERM PLANNING

A lack of foresight in planning for dam raising contributed to the failure. Successfully executing the raising plan required intimate coordination of impoundment water-level projections, production and transport of mine waste for raising, and seasonal constraints on construction. This made the tailings dam contingent at the same time on the water balance, the Mine plan, and the weather. But instead of projecting these interactions into the future, they were evaluated a year at a time, with dam raising often bordering on ad hoc and only responding to events as they occurred. The effects were twofold: a near overtopping failure in May of 2014, and restrictions on mine waste availability that produced the oversteepened slopes and deferred buttress expansion.

### 10.2.2 OBSERVATIONAL METHOD

The Observational Method was adopted as a design philosophy, but misapplied. For reasons not unrelated to planning shortcomings, instrumentation was relied upon to substitute for definitive input parameters and design projections. But the Mount Polley dam was ill-suited to this approach, for both practical and strategic reasons. The steep slopes and constant construction activity on the Perimeter Embankment prevented installation of instruments at optimal locations. More importantly, the instrumentation program was incapable of detecting critical conditions because, once again, the critical materials and their critical mode of undrained behaviour were not recognized.

# 10 | Conclusions

## 10.3 ROLE OF WATER

In light of its importance in planning and the near-overtopping incident, the role of water contained in the tailings storage facility (TSF) deserves special mention. First of all, overtopping did not cause the breach of August 4, 2014. However, the high water level acted in other ways that influenced both the failure and its effects.

High impoundment water levels were a major cause of chronic problems in maintaining a tailings beach around the perimeter of the dam. At the breach section, water was in direct contact with the upstream zone of tailings fill when failure occurred. This increased the piezometric level in the upstream zone above what it would have been had a wide tailings beach been present. The Panel's analyses show that this had some influence on dam stability, although it was not the dominant factor.

The high water level was the final link in the chain of failure events. Immediately before the failure, the water was about 2.3 m below the dam core. The Panel's excavation of the failure surface showed that the crest dropped at least 3.3 m, which allowed overflow to begin and breaching to initiate. Had the water level been even a metre lower and the tailings beach commensurately wider, this last link might have held until dawn the next morning, allowing timely intervention and potentially turning a fatal condition into something survivable.

Finally, the quantity of water had a great deal to do with the quantity of tailings released after the breach developed. It was water erosion that transported the bulk of the tailings, and these fluvial processes ended when the supply of water was exhausted. Had there been less water to sustain them, the proportion of the tailings released from the TSF would have been less than the one-third that was actually lost.

# 10 | Conclusions

## 10.4 REGULATORY FACTORS

The Panel examined regulatory activities by the Ministry of Energy and Mines (MEM) in relation to the failure and whether different actions on MEM's part might have prevented it. In particular, the Panel's attention was drawn to the period from 2009 to 2011 when no government inspections of the Mount Polley dam were performed. The Panel concludes that this lack of inspection was immaterial to the failure because there were no precursors that could have been detected, even on the eve of the breach. By definition, no amount of inspection can discover a hidden flaw.

The Panel also examined MEM's actions concerning factors that did have a material relationship to the failure. In this regard, MEM queried the designer about softer conditions in glaciolacustrine soils encountered in a groundwater well that were similar to those at the breach. Its inspector issued a "Departure from Approval" notice concerning the absence of an adequate tailings beach. The inspector questioned the designer's factor of safety  $FS = 1.3$  criterion, subsequently requiring its increase. The Panel found these actions to be appropriate and within the expected conduct of regulatory responsibilities.

It is not unreasonable to ask whether MEM could have acted sooner or more aggressively in these matters or even intervened in the design process, and perhaps this might have been warranted under the harsh illumination of hindsight. Yet the Panel considers that a bright line must be maintained between designer and Regulator. It is axiomatic that a Regulator cannot regulate its own activities. Were it to usurp the role of the designer, it would also usurp its own role.

## 10.5 POSSIBLE FAILURE PREVENTION

In fulfilling its Terms of Reference, the Panel considered what actions could have been taken to prevent the failure. From a purely technical perspective, apart from rectifying the deficiencies reviewed here, there is one that stands out.

The design for the next raise of the dam had been submitted only days before the failure. In it was a buttress that would have extended along the Perimeter Embankment, including the breach section. Although this buttress was still not designed using the appropriate stratigraphy or undrained strengths, the Panel determined that had it been in place, the failure would have been averted. The solution would have been correct, even if for the wrong reasons.

In keeping with its Terms of Reference, the Panel has developed these conclusions on the basis of technical factors specific to the Mount Polley failure. It must be left to others to determine how they might translate more broadly to legislative, administrative process, and policy areas.

# 11 | Recommendations

Recognizing that the path to zero failures involves a combination of best available technology (BAT) and best applicable practices (BAP), the Panel recommends the following:

**1) To implement BAT using a phased approach:**

- a. For existing tailings impoundments.** Rely on best practices for the remaining active life.
- b. For new tailings facilities.** BAT should be actively encouraged for new tailings facilities at existing and proposed mines.
- c. For closure.** BAT principles should be applied to closure of active impoundments so that they are progressively removed from the inventory by attrition.

See section 9.3.

**2) To improve corporate governance:**

Corporations proposing to operate a tailings storage facility (TSF) should be required to be a member of the Mining Association of Canada (MAC) or be obliged to commit to an equivalent program for tailings management, including the audit function.

See section 9.4.1.

**3) To expand corporate design commitments:**

Future permit applications for a new TSF should be based on a bankable feasibility that would have considered all technical, environmental, social and economic aspects of the project in sufficient detail to support an investment decision, which might have an accuracy of  $\pm 10\%$ – $15\%$ . More explicitly, it should contain the following:

- a.** A detailed evaluation of all potential failure modes and a management scheme for all residual risk.
- b.** Detailed cost/benefit analyses of BAT tailings and closure options so that economic effects can be understood, recognizing that the results of the cost/benefit analyses should not supersede BAT safety considerations.
- c.** A detailed declaration of Quantitative Performance Objectives (QPOs).

See section 9.4.2.

# 11 | Recommendations

## 4) **To enhance validation of safety and regulation of all phases of a TSF:**

Increase utilization of Independent Tailings Review Boards.

See section 9.4.3.

## 5) **To strengthen current regulatory operations:**

a. Utilize the recent inspections of TSFs in the province to ascertain whether they may be at risk due to the following potential failure modes and take appropriate actions:

- i. Undrained shear failure of silt and clay foundations
- ii. Water balance adequacy
- iii. Filter adequacy

b. Utilize the concept of QPOs to improve Regulator evaluation of ongoing facilities.

See section 9.4.4.

## 6) **To improve professional practice:**

Encourage the APEGBC to develop guidelines that would lead to improved site characterization for tailings dams with respect to the geological, geomorphological, hydrogeological and possibly seismotectonic characteristics.

See section 9.4.5.

## 7) **To improve dam safety guidelines:**

Recognizing the limitations of the current Canadian Dam Association (CDA) Guidelines incorporated as a statutory requirement, develop improved guidelines that are tailored to the conditions encountered with TSFs in British Columbia and that emphasize protecting public safety.

See section 9.4.6.

## 12 | Postscript and Acknowledgements

The Panel has been acutely aware of its responsibilities in conducting this investigation. It set out to be thorough, focusing on the technical issues, and to report its findings in an independent, open, transparent and timely manner. It is content that it has fulfilled its mandate. To do so required the digestion of thousands of pages of technical documents; field investigations involving mapping, drilling and sampling; complex laboratory tests; various theoretical analyses; and consolidation of its findings, conclusions and recommendations in a manner intended to be accessible to a variety of stakeholders. The Panel could not have met its objectives without the assistance of a number of dedicated and skilled individuals. The Panel wishes to acknowledge this assistance here.

First, and possibly foremost, the Panel expresses its gratitude to Mr. Kevin Richter, Assistant Deputy Minister, Ministry of Transportation and Infrastructure, who was appointed to lead the Secretariat for the investigation and his assistants, Stacy Scriver and Rupinder Prihar. Mr. Richter managed the business of the investigation with enormous skill, diplomacy and good grace. This allowed the Panel to focus on its main task and hence the Secretariat made a most valuable contribution to the collective effort.

The Panel retained Thurber Engineering Limited (Thurber) to undertake a wide variety of technical tasks acting under the direction of the Panel. These tasks involved site mapping, drilling and sampling, a wide suite of laboratory tests, a variety of analyses, and preparing material for inclusion in the report. The Thurber team was outstanding in its technical contributions and dedication to this assignment. The Panel was extremely pleased to work with such a skilled team that included the following:

- In Vancouver – David Regehr (Project Manager), Paul Wilson, Caleb Scott, Ben Singleton-Polster, Denny Ma, Andrea Lougheed, Paul Evans
- In Victoria – Stephen Bean, Warren Wunderlick, Suzanne Powell
- In Calgary – John Sobkowicz
- And others too numerous to mention

Deborah Lovett, QC, of Lovett & Westmacott was retained as legal advisor to the Panel and provided wise counsel throughout the period of the investigation.

Judith Brand provided senior editorial advice and Shawn Robins, Robins Communications, assisted the Panel in organizing its outreach activities.

While many have contributed to this report, the Panel retains sole responsibility for its content.



# 13 | List of Abbreviations

## ABBREVIATIONS

APEGBC:	Association of Professional Engineers and Geoscientists of British Columbia	OCR:	overconsolidation ratio
BAP:	best applicable practices	OMS:	Operations Maintenance and Surveillance
BAT:	best available technology	PMP:	Probable Maximum Precipitation
CDA:	Canadian Dam Association	QPO:	Quantitative Performance Objective
CPT:	cone penetration test	RCPT:	Resistivity Cone Penetration Test
CQA:	Construction Quality Assurance	S.O.L.:	setting out line
DSR:	Dam Safety Review	SPT:	standard penetration test
DSS:	direct simple shear	Thurber:	Thurber Engineering Limited
EOR:	Engineer of Record	TSF:	tailings storage facility
EPRI:	Electric Power Research Institute	TSM:	Towards Sustainable Mining
ESA:	effective-stress analysis	USA:	undrained strength analysis
FOIPPA:	<i>Freedom of Information and Protection of Privacy Act</i>	USBR:	U.S. Bureau of Reclamation
FS:	factor of safety	VST:	vane shear test
GLU:	Upper Glascioloacustrine Unit		
GSA:	grain size analyses		
ITRB:	Independent Tailings Review Board		
KCB:	Klohn Crippen Berger		
KP:	Knight Piésold		
LiDAR:	Light Detection And Ranging		
LI:	Liquidity Index		
LPT:	Large Penetration Testing		
MAC:	Mining Association of Canada		
MEM:	Ministry of Energy and Mines		
MFLNRO:	Ministry of Forests, Lands and Natural Resource Operations		
MoE:	Ministry of the Environment		
MOU:	Memorandum of Understanding		
MPMC:	Mount Polley Mining Corporation		

# 14 | Glossary of Technical Terms

## GLOSSARY

**Angle of repose:** the maximum slope steepness that dry granular material can sustain

**Anisotropy:** directional differences in properties, typically horizontal and vertical

**Anticlinal structure:** dome-shaped folding

**Applied load:** usually gravity stresses imposed by a structure; simplistically, its weight

**Arcuate headscarps:** semicircular and nearly perpendicular slopes delineating the upper end of a slope movement

**Artesian pressure:** water pressure sufficient to cause water to flow upwards out of the ground

**Bankable feasibility study:** a level of design sufficient for detailed cost estimates

**Bathymetric survey:** survey of underwater surfaces

**Beach:** a gently sloping surface of deposited tailings

**Bedding:** layering, commonly horizontal

**Blow count:** the number of drops of a heavy weight required to advance a sampler 30 cm into the ground

**Buttress:** a berm constructed at the bottom of a slope to increase its stability

**Chimney drain:** a zone of sand or gravel within a dam for collecting and conveying water

**Coefficient of consolidation:** a parameter used to calculate change of pore pressure with loading

**Crest:** the top of a dam or slope

**Critical failure surface in stability analysis:** the failure surface with the lowest factor of safety

**Cycloning:** separation of tailings into coarser and finer fractions

**Dendritic drainages:** branching stream channels

**Dip direction:** the direction in which a geologic structure slopes downward

**Direct shear:** type of test used to determine drained shear strength

**Direct simple shear:** type of test used to determine undrained shear strength

**Downcutting:** a natural process of excavation, usually by erosion

**Downthrow:** downward movement

**Effective stress:** the stress experienced by soil particles after the known pore pressure is subtracted

**Effective-stress strength:** the strength of a soil expressed only in terms of the effective stress

**En echelon scarps:** parallel steep slopes produced by ground movement

**Factor of safety:** the ratio of available strength to the strength required for equilibrium; a measure of stability

**Fines:** fine particles smaller than visible with the naked eye, typically less than 0.074 mm diameter

## 14 | Glossary of Technical Terms

**Flowslide:** high-velocity earth movement; mudflow

**Fluvial processes:** processes caused by or associated with rivers or streams

**Freeboard:** reservoir capacity reserved for storage of flood inflows, including wave height

**Grab samples:** disturbed samples

**Graben:** downdropped block within the ground

**Headscarp:** steep slope at the upper end of a landslide

**Hydraulic-cell deposition:** controlled discharge of tailings into a small, confined area

**Hydraulic fracturing:** cracking of soil caused by water pressure

**Inclinometer:** a device for measuring horizontal subsurface movements

**Internal erosion:** subsurface transport of soil particles by water

**Interparticle voids:** open spaces between soil particles

**Glaciofluvial:** associated with or deposited in a glacial stream

**Glaciolacustrine:** associated with or deposited in a glacial lake

**Lift lines:** boundaries between successive layers of compacted fill

**Loading:** the imposition of stresses or weight; see applied load

**Marker bed:** a prominent layer of soil or rock used as a reference

**Normally consolidated:** a state or condition of soil that is experiencing pressures equal to or exceeding the pressures that it has experienced in the past

**Oedometer test:** a test for measuring compression of soil under load

**Offtake:** a drain or pipe that discharges flow

**Orthophoto imagery:** aerial photograph looking directly down on the terrain

**Overconsolidation:** a state or condition of soil produced by past stresses greater than those that currently exist

**Overtopping:** water flowing over the crest of a retaining dam or structure

**Phreatic surface:** water table

**Piezometer:** a device for measuring subsurface water pressure

**Piping:** see internal erosion

**Pore pressure:** the pressure of water that exists within the voids of a soil mass; see interparticle voids

**Preconsolidation pressure:** the maximum pressure experienced by the soil in its past

**Pre-shearing:** the process or condition of having been previously sheared

**Relic erosional surface:** ground surface remaining after previous erosion

## 14 | Glossary of Technical Terms

**Residual strength:** strength of a soil after having been sheared; see also pre-shearing

**Rills:** small-scale gullies

**Runup:** the height of breaking waves on a slope

**Sand tailings:** coarser fraction of tailings

**Scarp:** a very steep, near-perpendicular slope at the head of a landslide; see also headscarp

**Scour:** erosion by surface water

**Seepage flow:** flow of subterranean water

**Sentinel section:** an instrumented section providing preliminary information; see also inclinometer, piezometer

**Shear:** a) the act or process of one surface sliding across another; b) a state of stress in the ground

**Shell:** a zone of material that supports the core of a dam

**Slickenside:** polished surface resulting from shearing

**Slimes:** finer fraction of tailings

**Slump blocks:** large masses subject to or transported by downslope movement

**Stereopairs:** aerial photographs producing a three-dimensional image

**Stratigraphy:** systematic or characteristic layering exhibited by soil or rock at a particular locale

**Substrate:** underlying soil

**Survey monuments:** fixed reference points for measuring relative movements

**Tailings:** finely ground rock particles remaining after extraction of valuable minerals

**Tailings beach:** see beach

**Till:** unsorted glacial sediment moved or deposited directly by the glacier

**Tip resistance:** the pressure measured at the tip of the cone during CPT testing

**Toe:** bottom of a slope

**Triaxial test:** type of test used here to determine drained and undrained strength

**Undrained strength:** the strength of a soil that incorporates the effect of pore pressures generated by shearing

**Undrained strength ratio:** the ratio of undrained strength to effective stress

**Vane testing:** an in situ test for measuring undrained strength of clays

**Varving:** thinly laminated layering

**Water balance:** an accounting of water inputs and outputs for determining water accumulation or deficit

**Whaleback:** a linear bulge or uplift

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