Klohn Crippen Berger

Ministry of Energy and Mines

Mount Polley Tailings Dam Failure

A Summary of Opinions in Support of CIM Investigation

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M09954A01.730

August 2015

Klohn Crippen Berger

August 24, 2015

Ministry of Energy and Mines
Suite 600 – 1810 Blanshard Street
Victoria, British Columbia
V8T 4J1

Al Hoffman
Chief Inspector of Mines

Dear Mr. Hoffman:

Mount Polley Tailings Dam Failure
A Summary of Opinions in Support of CIM Investigation

Please find attached the above titled report in support of the Chief Inspector of Mines Investigation into the Mount Polley Tailings Dam Failure. The report should be read in conjunction with the Klohn Crippen Berger (KCB) report on the "Mechanism of Failure" (KCB 2015). In general, the report supports the findings of the Independent Review Panel (IRP 2015) and provides additional details on the circumstances leading up to and surrounding the failure.

Please do not hesitate to contact the undersigned if you have any questions concerning this report.

Yours truly,

KLOHN CRIPPEN BERGER LTD.

Harvey N. McLeod, P.Eng., P.Geo.
Project Manager

HM:dl/jcp

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Ministry of Energy and Mines

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COMMON TERMS AND ABBREVIATIONS

A list of common terms and abbreviations that are referenced throughout this report is summarised below.

General

AMEC	Amec Foster Wheeler
BC MoE	British Columbia Ministry of the Environment
BGC	BGC Engineering Inc.
CDSA	Canadian Dam Safety Association
CDA	Canadian Dam Association
CIM	Chief Inspector of Mines
DSR	Dam Safety Review
DBE	Design Basis Earthquake
EIT	Engineer in Training
EOR	Engineer of Record
EPRP	Emergency Preparedness and Response Plan
FoS	Factor Of Safety
KP	Knight Piésold Ltd.
HSRC	Health, Safety and Reclamation Code for Mines in British Columbia
H:V	Horizontal : Vertical (slope)
IDF	Inflow Design Flood
IRP	Independent Review Panel
KCB	Klohn Crippen Berger Ltd.
MAC	Mining Association of Canada
MCE	Maximum Credible Earthquake
MDE	Maximum Design Earthquake
MPMC	Mount Polley Mining Corporation
MEM	Ministry of Energy and Mines
ME	Main Embankment
OBE	Operational Basis Earthquake
OMS	Operation, Maintenance, and Surveillance Manual
PE	Perimeter Embankment
PMP	Probable Maximum Precipitation
PMF	Probable Maximum Flood
QA	Quality Assurance
QC	Quality Control
SCP	Seepage Collection Pond
SE	South Embankment
TSF	Tailings Storage Facility
TSM	Towards Sustainable Mining (Mining Association of Canada)

Geotechnical

SPT	Standard Penetration Test
CPT	Cone Penetration Test
GLU	Glaciolacustrine Unit
UGLU	Upper Glaciolacustrine Units
UGT	Upper Glacial Till
MGT	Middle Glacial Till
LGLU	Lower Glaciolacustrine Unit
LGT	Lower Glacial Till (lodgment/basal)
kPa	kilopascal
kN	kilonewtons
VWP	Vibrating Wire Piezometer

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1 INTRODUCTION

1.1 General

This report presents a summary of opinions by Klohn Crippen Berger (KCB) in support of the British Columbia Ministry of Energy and Mines (MEM) for the Chief Inspector of Mines (CIM) investigation of the Mount Polley tailings dam breach, which occurred on August 4, 2014. This work was commissioned and authorized by MEM to assist in their investigations of the failure. KCB carried out the work at the request and direction of the CIM, specifically to provide engineering support. The investigation was conducted by the CIM and KCB participation and opinion was based on the information provided by the CIM or by information collected in the CIM interviews, as a geotechnical engineer for the purposes of informing the CIM. Should that information change (or if there is new or altered information with time, clarity, etc.) KCB should be requested to re-evaluate its opinions appropriately.

The Mount Polley mine, owned and operated by Mount Polley Mining Corporation (MPMC), located 11 kms from the town of Likely in the interior of British Columbia at N5819160 m and E595110 m (Figure 1.1).

Tailings are retained by a U-shaped dam abutting a natural slope on the northwest side. The tailings dam (Figure 1.2) comprises three embankments: the Main Embankment (ME) on the southeast side, South Embankment (SE) bounding the southwest side and Perimeter Embankment (PE) bounding the northeast side. The PE comprises a rockfill embankment raised in stages by the centreline and modified upstream construction methods, with an upstream “core” of compacted glacial till and filter zones to restrict seepage through the dam. The dam failed during construction of the Stage 9 raise to Elevation 970 m when the PE was 40 m high. At the time of failure, the exterior dam slope was nominally 1.3H:1V, which is the steepest slope that dumped rockfill can typically be placed.

The dam failed between Stations 4+110 and 4+350 at the highest section of the PE. The breach released 21 million m^3 of water and tailings solids which flowed into Hazeltine Creek and thence into Quesnel Lake. The pre-failure configuration of the PE is shown on Figure 1.3. The post-failure configuration of the PE is shown on Figure 1.4.

1.2 Government of British Columbia Independent Review Panel Inquiry

An Independent Review Panel (IRP) was formed by the Government of British Columbia shortly after the failure to investigate and report on the cause of the failure and to make recommendations on actions that could be taken to ensure that a similar failure does not occur. The IRP’s mandate is given in their Terms of Reference dated October 6, 2014 and their report was published on January 31, 2015 (IRP 2015).

KCB conducted the majority of the post-failure field investigations and supporting laboratory testing in the immediate breach area. That factual work was published in KCB Progress Report Nos. 1 through 4, which were distributed to MEM and MPMC as they were completed. These reports were also made available to the IRP by MPMC.

1.3 CIM Investigation

MEM, through the Chief Inspector of Mines (CIM) initiated a separate investigation to determine why the failure occurred and what were the root causes that may have contributed to the breach of the dam. The purpose of the CIM investigation was to also identify if there were any contraventions of the Health, Safety and Reclamation Code for Mines in British Columbia (HSRC) and, by understanding the circumstances surrounding the failure, to improve Regulatory Practices to reduce the risk of tailings dam failures.

As part of the investigation, CIM commissioned KCB to undertake a detailed forensic assessment that included site investigations, laboratory testing and technical analysis of the Mechanism of Failure (KCB 2015). Overall, the KCB report agrees with the IRP's opinion on the basic mechanism of failure of the tailings dam. That basic mechanism was a sliding failure through a lightly over-consolidated glaciolacustrine clay unit (the upper glaciolacustrine unit (UGLU)) in the foundation which dropped the embankment crest enough to allow the pond to overtop and, within hours, to completely breach a portion of the PE. This mechanism is manifested by physical evidence of dam displacements and shear movements in the dam foundation, and is supported by back-analyses using the engineering properties of the dam and foundation soils.

KCB also participated in interviews carried out by the CIM Investigation team, and provided technical support to the investigation for the geotechnical and tailings management aspects surrounding the dam breach.

1.4 Report Objectives and Organization

The objective of this report is to provide a summary of facts and our opinions on the engineering, construction, and management factors which may have influenced the Mount Polley dam breach.

This report relies on the factual data presented in the KCB Progress Reports, the Mechanism of Failure Report and the following additional work:

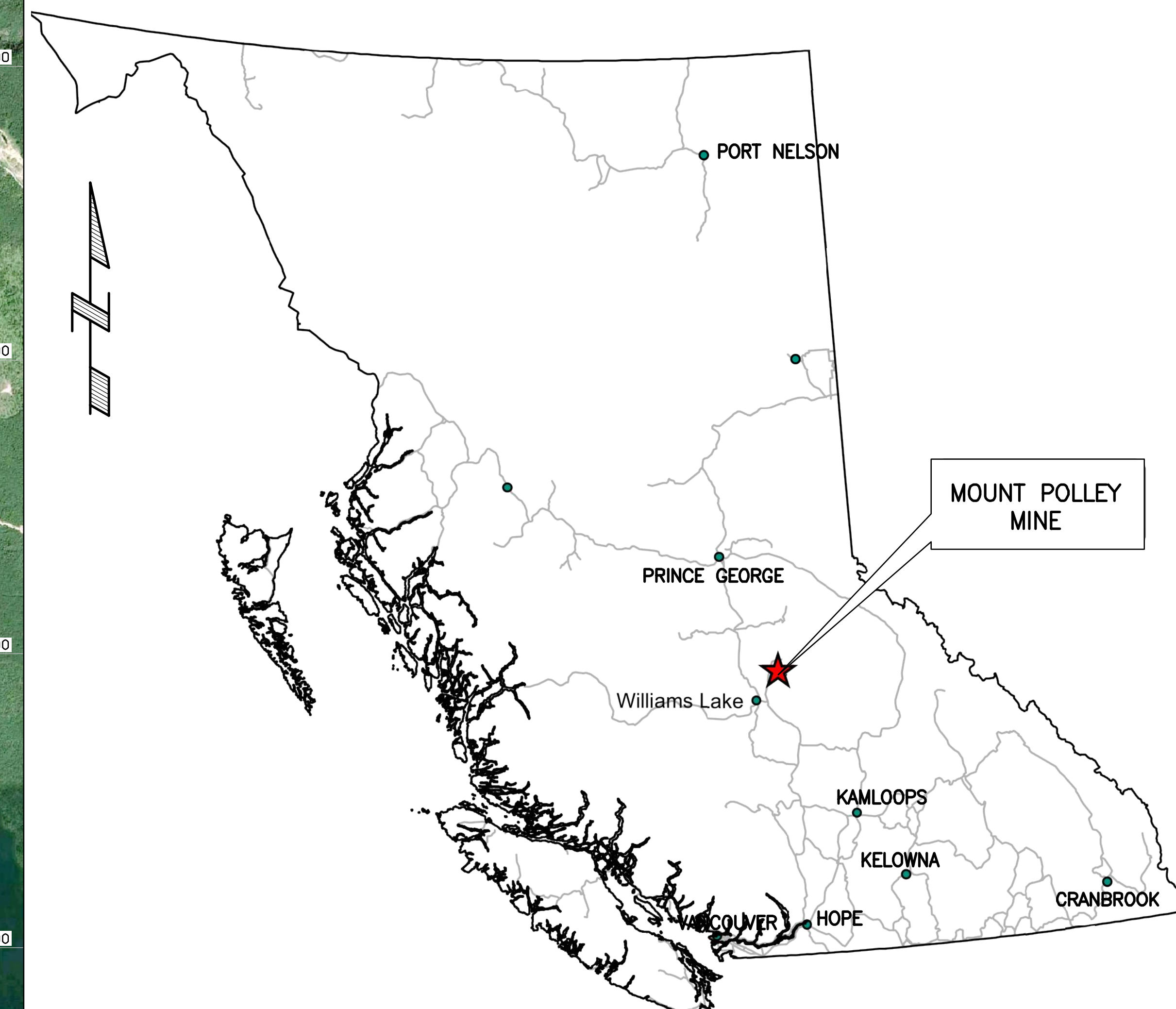
- the CIM interviews with key personnel involved in the construction, design and management of the TSF;
- review of available documents including, but not limited to: design reports, correspondence, permit information and dam inspections, construction documents, Tailings Storage Facility (TSF) management plan;
- analysis of data and, where appropriate, stability analyses; and
- review of management, design and regulatory practices.

The report contains the following main sections:

- *Section 2* provides an overview of the interviews with MPMC employees, contractors, and consultants that worked on the TSF. Interviews with Imperial Metals corporate staff were also carried out. Key interview findings are summarized into engineering/construction and management observations.

- *Section 3* summarizes the 'state of knowledge' prior to the dam breach including site investigations, design basis, geotechnical parameters and stability analyses used. The purpose of this section is to document what factual data was available for the design leading up to the breach and to document the design basis used by the designers over the life of the TSF.
- *Section 4* details KCB's interpretation of this 'state of knowledge' prior to the breach. The purpose of this section is to assess, based on the same factual information, if the dam stability assessment would have been similar to the designer.
- *Section 5* summarizes KCB's assessment of the Mechanism of Failure, including key geotechnical parameters, and potential contributing causes to determine what, if any, role they may have had in reducing the factor of safety (FoS) of the dam and potentially contributing to the failure.
- *Section 6* reviews the MPMC operation and management practices and identifies elements relevant to the TSF breach.
- *Section 7* reviews the consultant design, and construction practices and elements that may be relevant to the TSF breach.
- *Section 8* reviews the regulatory practices and elements that may be relevant to the TSF breach.
- *Section 9* conclusions and recommendations.

PLAN VIEW
NTS



GENERAL LOCATION
NTS

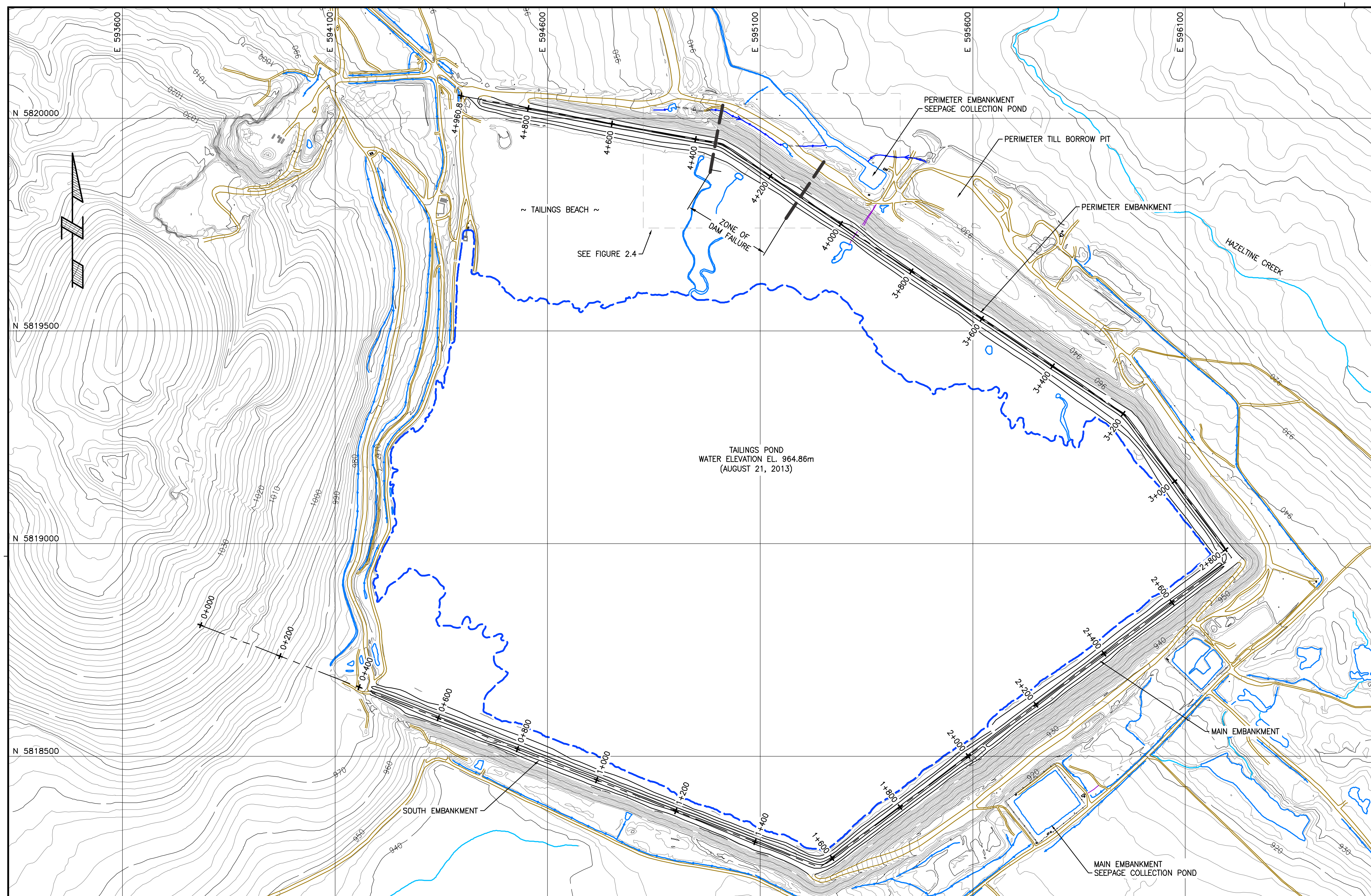
NOTES:

1. AERIAL VIEW OF POST–FAILURE MINE SITE TAKEN ON AUGUST 5, 2014. OVERLAID ON PRE–FAILURE REGIONAL AERIAL VIEW TAKEN ON JANUARY 2015.

CLIENT	BC MINISTRY OF ENERGY AND MINES
	Klohn Crippen Berger
PROJECT	MT. POLLEY DAM FAILURE
TITLE	SITE LOCATION
PROJECT No.	M09954A01
FIG. No.	1.1

Time: 10:00:19
Date: 15/07/2015
Scale: 1:2(PS)
Xrefs: BM–Tulsequah_Apr08

KCB–R–MLD



PLAN VIEW

LEGEND:

- SET OUT LINE (S.O.L.) FOR TAILINGS EMBANKMENT
- SITE ROAD
- EDGE OF TAILINGS WATER POND
- WATER STREAMS
- PONDS OR PONDED WATER

NOTES:

1. TOPOGRAPHY PROVIDED BY MPMC AND SURVEYED ON AUGUST 21, 2013 (LIDAR SURVEY).
2. DATUM: UTM NAD 83 ZONE 10.

SCALE 0 250 m

CLIENT
BC MINISTRY OF ENERGY AND MINES

Klohn Crippen Berger

PROJECT
MT. POLLEY DAM FAILURE

TITLE
TAILINGS STORAGE FACILITY
PRE–FAILURE TOPOGRAPHY
AUGUST 2013

PROJECT No. M09954A01

FIG. No. 1.2

KCB–R–MLD

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Date: 10/06/2015
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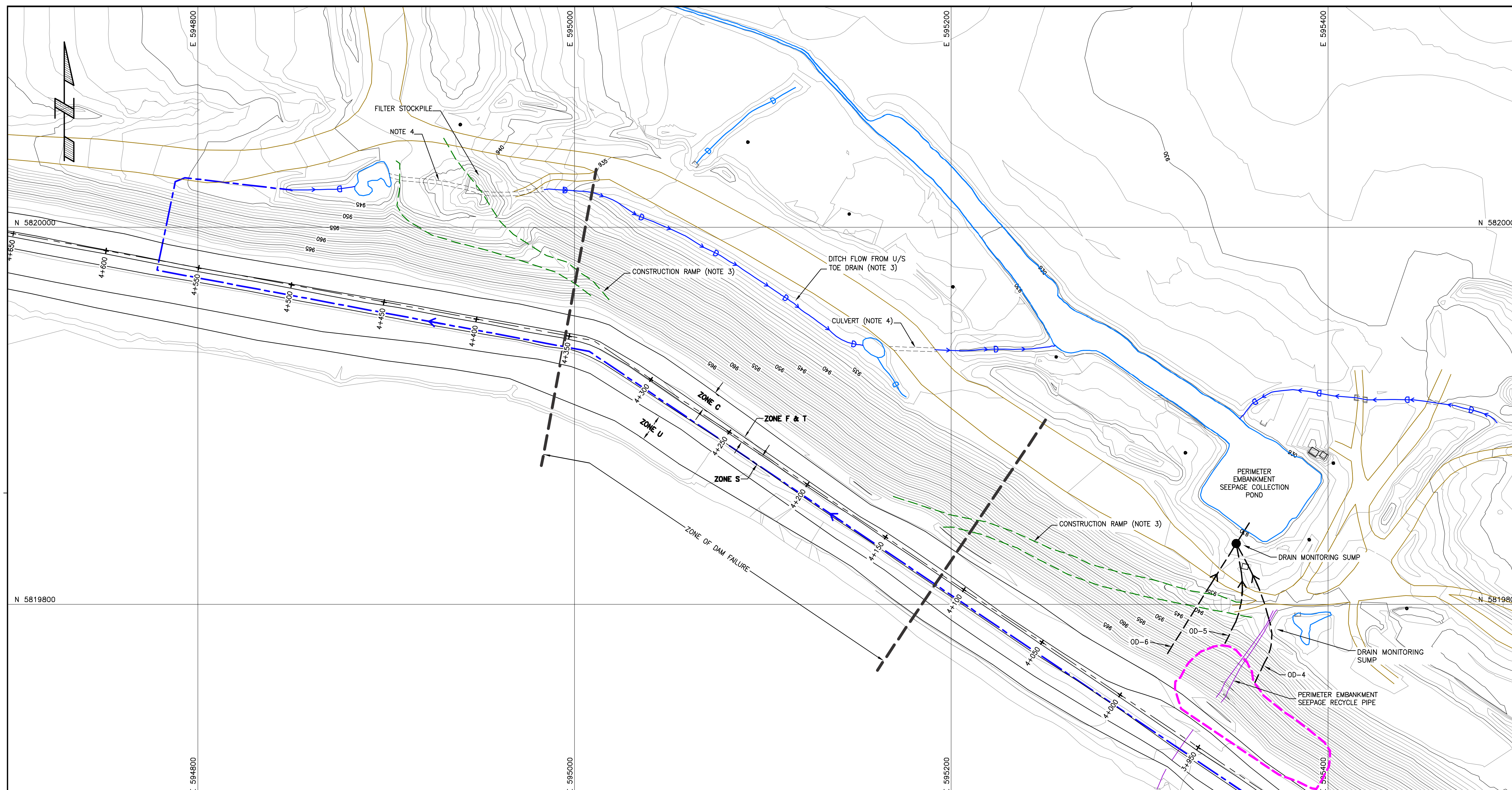
Drawing File: \\VCR-FS-V\Projects\M\M09954A01–MEM–Confidential Project\400 Drawings\Cad\Plans&Sections–WithAsBuilt\Figures–Report\MEM Inquiry Report\Fig1.2 – GeneralArrange–TSF–2013.dwg (persyem)
Xrefs: BM–Tulsequah_Apr08



PLAN VIEW

LEGEND:

- SET OUT LINE (S.O.L.) FOR TAILINGS EMBANKMENT
- SITE ROAD
- UPSTREAM TOE DRAIN
- DITCH
- OUTLET DRAINS
- CYCLONED SAND TRIAL AREA
- POWER POLES

NOTES:

1. TOPOGRAPHY PROVIDED BY MPMC AND SURVEYED ON AUGUST 21, 2013 (LIDAR SURVEY).
2. DATUM: UTM NAD 83 ZONE 10.
3. FEATURES TAKEN FROM "STAGE 9 TAILINGS EMBANKMENT CREST EL. 970m PLAN VIEW SHEET 4 TO 4" ; AMEC MARCH 11, 2013 USING TOPOGRAPHY FROM OCT. 2012 FLYOVER BY MPMC (MPMC) ; AND DRAWINGS 120 – TAILINGS STORAGE FACILITY STAGE 3C PERIMETER EMBANKMENT PLAN (MP00038)
4. RAMPS COVERED WITH ZONE C DURING STAGE 9 (2013) CONSTRUCTION. BORDERS SHOWN ARE TAKEN FROM AMEC'S 2012 DRAWING "2013.03, SHEET 4 OF 4".
5. ASSUMED BURIED WATER CONVEYANCE BASED ON FLOW LINES INDICATED IN AMEC 2013 AND BGC 2014 REPORTS.

SCALE 0 50 m

CLIENT: BC MINISTRY OF ENERGY AND MINES

Klohn Crippen Berger

PROJECT: MT. POLLEY DAM FAILURE

TITLE: EMBANKMENT LAYOUT AUGUST 2013

PROJECT No.: M09954A01

FIG. No.: 1.3

KCB-R-MLD

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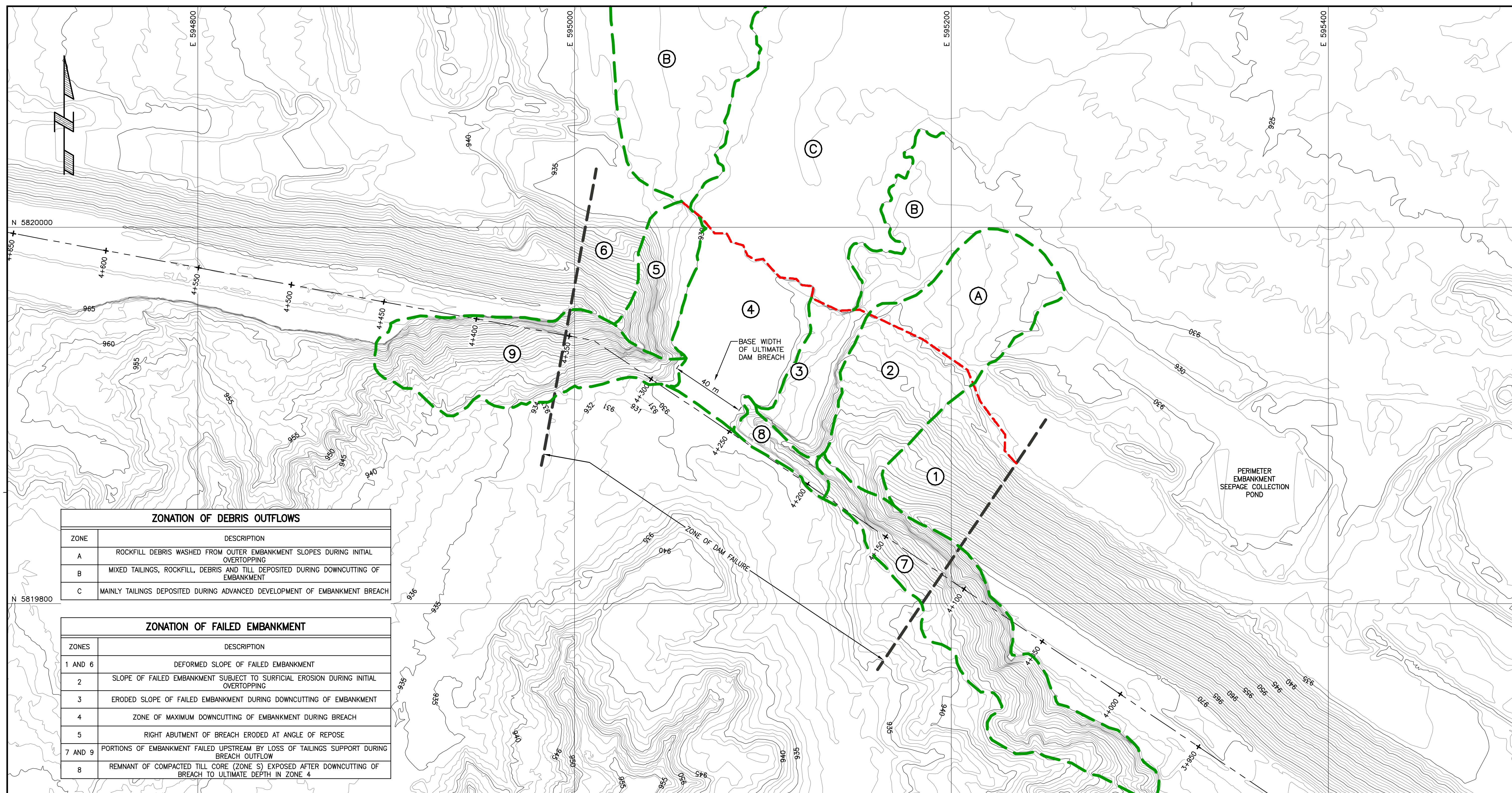
Xrefs: BM-Tulsequah_Apr08



ZONATION OF DEBRIS OUTFLOWS

ZONE	DESCRIPTION
A	ROCKFILL DEBRIS WASHED FROM OUTER EMBANKMENT SLOPES DURING INITIAL OVERTOPPING
B	MIXED TAILINGS, ROCKFILL, DEBRIS AND TILL DEPOSITED DURING DOWNCUTTING OF EMBANKMENT
C	MAINLY TAILINGS DEPOSITED DURING ADVANCED DEVELOPMENT OF EMBANKMENT BREACH

ZONATION OF FAILED EMBANKMENT

ZONES	DESCRIPTION
1 AND 6	DEFORMED SLOPE OF FAILED EMBANKMENT
2	SLOPE OF FAILED EMBANKMENT SUBJECT TO SURFICIAL EROSION DURING INITIAL OVERTOPPING
3	ERODED SLOPE OF FAILED EMBANKMENT DURING DOWNCUTTING OF EMBANKMENT
4	ZONE OF MAXIMUM DOWNCUTTING OF EMBANKMENT DURING BREACH
5	RIGHT ABUTMENT OF BREACH ERODED AT ANGLE OF REPOSE
7 AND 9	PORTIONS OF EMBANKMENT FAILED UPSTREAM BY LOSS OF TAILINGS SUPPORT DURING BREACH OUTFLOW
8	REMNANT OF COMPACTED TILL CORE (ZONE S) EXPOSED AFTER DOWNCUTTING OF BREACH TO ULTIMATE DEPTH IN ZONE 4

PLAN VIEW

SCALE 0 50 m

LEGEND:

- SET OUT LINE FOR TAILINGS EMBANKMENT
- LIMIT OF CRACKING AND UPTHRUST GROUND AT EMBANKMENT TOE

NOTES:

1. TOPOGRAPHY PROVIDED BY MPMC AND SURVEYED ON AUGUST 21, 2013 (LIDAR SURVEY).
2. DATUM: UTM NAD 83 ZONE 10.

CLIENT: BC MINISTRY OF ENERGY AND MINES

Klohn Crippen Berger

PROJECT: MT. POLLEY DAM FAILURE

TITLE: PLAN OF DAM FAILURE AND BREACH ZONE

PROJECT No. M09954A01

FIG. No. 1.4

KCB-R-MLD

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Date: 10/08/2015
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Xrefs: BM-Tulsequah_Apr08

2 INTERVIEWS

2.1 General

Interviews were conducted as part of the CIM Investigation to gain an understanding of the conditions at the site and to understand the knowledge that various persons had and the management practices that were carried out. The information collected during the interviews was taken at face value and the documentation in this section of the report reflects what was said during the interviews. The information was openly related during the interviews and documentation focuses on the geotechnical aspects and the management of the TSF. KCB's role was to summarize the geotechnical aspects of the interviews.

The majority of the interviews were carried out at the mine site over the periods of August 18 to 20, August 25 to 29, September 2 to 5, September 10 to 12, and October 14 to 15, 2014. Additional interviews were held with the consulting engineers over the period of October 21 to October 31, in Vancouver and on November 4, 2014 in Prince George. Senior managers and executives of MPMC and Imperial Metals Ltd. were interviewed in Vancouver between January 22 and 28, 2015. Follow up interviews with Knight Piesold (KP) were held on January 21, 2015 and May 6, 2015.

Interviews were carried out with the following groups:

- MPMC employees: Equipment operators, shifters, health and safety, engineering, TSF inspectors, electrical, maintenance, and trainers.
- Construction contractors: Peterson Construction – equipment operators and supervisors.
- Consultants: Amec Foster Wheeler (AMEC), Knight Piésold Ltd. (KP), and BGC Engineering Inc. (BGC).
- Senior managers and executives of MPMC and Imperial Metals Ltd.

The purpose of the interviews was to understand what happened and what might have been the cause of the failure and not to assess blame. CIM objectives of the interviews were to:

- gain an understanding of the site conditions and possible mechanism of failure;
- collect information that could be used to improve future policy decisions and state of practice;
- understand the communication and management systems that were in place; and
- determine if there were any contraventions of the MEM: Health, Safety and Reclamation Code (HSRC).

All of the interviews were recorded by CIM for permanent record purposes. The purpose of this section of the report is to summarize the key elements from the interviews under two main categories:

1. Engineering and construction practices which are relevant to the mechanism of failure, site conditions and design.
2. Practices which are relevant to the management of the TSF, and which may have had some influence on the construction and operations oversight and safety of the facility.

The summaries are not exhaustive and not all the details of the interviews have been included. Only those which KCB feels may contribute to the understanding of the failure circumstances have been noted.

2.2 Interviews with MPMC and Contractor Staff

2.2.1 General

Summaries of the interviews are included in Table 2.2, Table 2.3, and Table 2.4 for the engineering and environment staff, shifters and operating staff, and contractors, respectively. The key observations of the interviews are described in the following sections.

2.2.2 Failure Initiation and Response

Timeline of Failure

Witness observations were helpful in establishing the potential time line of the failure event and providing water level records (obtained from the Mill Control room and shown on Figure 2.1) for the Perimeter Sump station. The timeline is summarized as follows:

August 3, 2014

10:30 PM	As part of normal operating procedures, a MPMC electrician, [redacted], drove to the Perimeter Sump Station and turned on the No. 2 pump at approximately 10:30 PM. He then drove back to the process plant along the crest of dam, near the breach, at approximately 10:45 (interview record dated September 12, 2014).
11:40 PM to 12:10 AM	According to the pumping rate records, the water level in the sump started to level out, i.e. water was flowing into the sump at a rate equal to the pumping, which is not normal.

August 4, 2014

12:10 AM	Water level starts to modestly increase – interpreted to be from water being released from the TSF. The time is also near when the sand cat operator, [redacted], working at the SE thought he" heard some water". The two observations suggest that water was flowing over the dam.
1:00 AM to 1:06 AM	Water level rapidly rises in the Perimeter Sump, a short interim spike within that time also suggests that there may have been a surge of water, before the

	sump water level was exceeded. The rapid rise suggests that the dam had breached further.
1:08 AM	Time reported by a number of staff as to when the power went out, which is near the time of the rapid rise in the sump water level. The power going out appears to be the result of the dam failure inundating the power lines located approximately 300 m downstream of the breach.
1:40 AM to 2:10 AM	Staff went to the TSF to check the power lines and reclaim water lines and realized that failure of the dam was in progress.
2:20 AM	Emergency calls started being made to all potential parties and the dam area was being cordoned off.
4:30 AM to 4:45 AM	Checking on Polley Lake – lake level rose 1 m.
5:15 AM	Breach flow was observed as also reporting to Hazeltine Creek. Starting to get light outside.
5:30 AM	Reclaim barge on tailings – water still "roaring" out of the TSF. Muddy water flowing out of the breach.
6:00 AM	Helicopters on site and breach flow was observed as "going over the displaced block of soil on the downstream side of the dam". The "displaced block of soil" was later confirmed to be up-thrust native till, and referred to as the "whaleback" by the IRP (2015).

<u>August 5, 2014</u>

12:00 AM	Flow had reduced but was still "significant".
4:00 PM	Flow had abated.

The times expressed above should be considered approximate, particularly when based on the memory of the interviewees. However, the Perimeter Sump Pump water level record, which is shown on Figure 2.1, does provide an accurate record of the pond level with time. The time clock on the water level recorder was checked on September 12, 2014 and was noted to be approximately 7 minutes behind (slower) than actual time. The recorded timed above correlate with the detailed account of the event provided in the CIM interviews.

It should be noted that the timeline provided by [redacted] differs from what he provided to the IRP where he stated that he noticed that the Perimeter Pond was going to alarm in high level within the hour at 11:00 and he drove the dam and started the 2nd pump at 11:30 and did not notice anything out of the ordinary while he was down there (IRP interview record dated August 13, 2014).

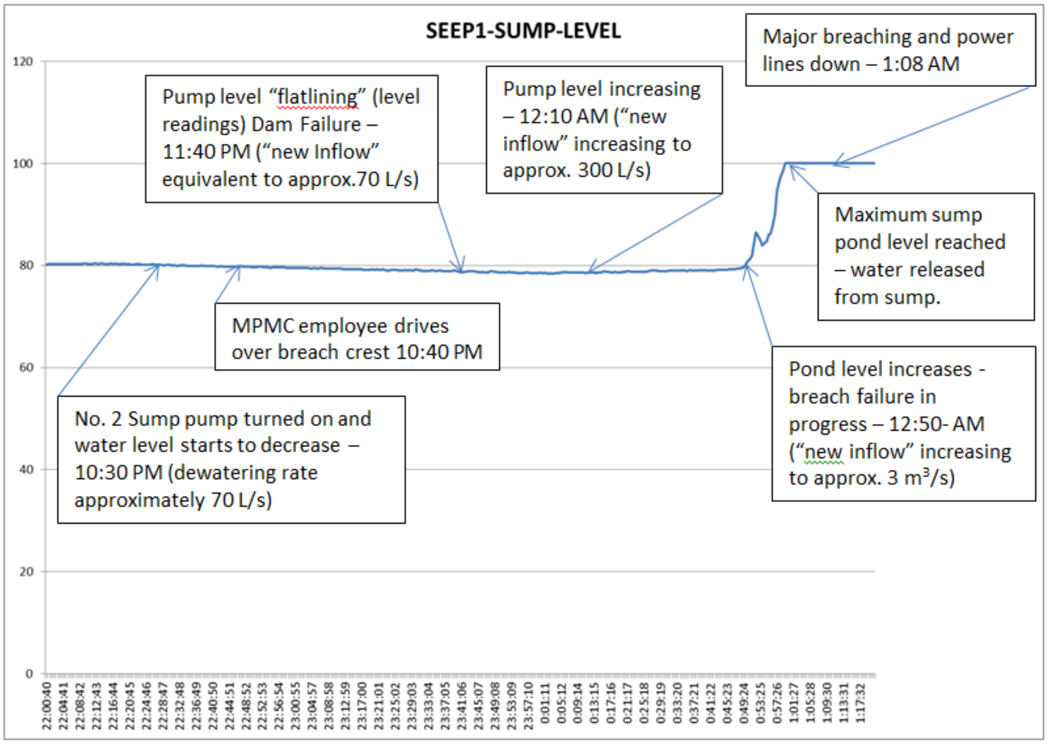


Figure 2.1 Perimeter Sump Water Level Record

MPMC Response to the Failure

The definitive realization that the dam had failed was made around 2:00 AM on August 4th and MPMC staff reacted by cordoning off the area and contacting key personnel. Some of the observations collected from the interviews regarding the response to the failure and implementation of safety measures include the following:

- The first call was made at approximately 2:20 AM to Don Ibey, the responsible person for "call-out, who did not respond.
- The first senior management staff, Art Frye, was contacted by 2:30 AM.
- The "back-up" for the call-out person [redacted] was notified at 4:00 AM and took control of the response at 4:30 AM.

2.2.3 TSF Construction and Reporting Structure

MPMC Reporting Structure for the TSF

The reporting structure and responsibilities of the various MPMC staff with respect to the TSF at the time of the breach comprised the following (as described in the Operations Maintenance and Surveillance Manual (OMS) -2013 (MPMC 2013b). This is further discussed in Section 6 of this report.

Luke Moger, Tailings Project Manager, was responsible for oversight of the activities for the TSF, although in 2014 some of the responsibilities were being transitioned to Nicholas Bergeron. Luke

Moger had taken over the position from Ron Martel, [redacted]. The TSF responsibilities included:

- co-ordinating the construction contractors, e.g. supervision/overview of Peterson Construction;
- co-ordinating with mine operations and the shifters with respect to the use of mine equipment, mine rock and filter material for the dam;
- supervising the MPMC dam inspectors;
- co-ordinating and overviewing the work of consultants: e.g. planning of site investigations, instrumentation requirements, design reports, construction reports, site visits by support engineers and head office staff, etc.;
- liaising with Environment Group with respect to water management;
- liaising with mine operations with respect to long range planning for mine rock requirements for the TSF; and
- internal reporting on TSF.

MPMC dam inspectors were summer students (civil technology and engineering). The inspectors report to Luke Moger/Nicholas Bergeron. The inspector's responsibilities included:

- quality control (QC) testing and transfer of the daily QC reports to the consultant;
- advising the contractor and MPMC if tests are not within the specifications;
- control material placement, e.g. lift thickness, geometry, etc.; and
- co-ordination with AMEC support engineers for off-site laboratory testing.

Luke Moger had two reporting functions: 1) on the TSF he reported to Dale Reimer, the Mine Manager; and, 2) on other matters he reported to Art Frye, Mine Operations Manager (see Section 6.1). Dale Reimer (Mine Manager) reported to Don Parsons, Chief Operating Officer of Imperial Metals on tailings aspects (see Section 6.1).

Use of AMEC Construction Support

AMEC support engineers, located in Prince George, were used to train the MPMC Inspectors and to conduct periodic quality assurance (QA) visits to the site. Additional QA was provided by either the Project Manager or Senior Geotechnical Engineer from Vancouver, typically on an annual basis. Some of the observations collected from the interviews on the efficiency of the QA/QC system include the following:

- The process for responding to QC non-compliance aspects was not always consistent, e.g. limited compaction of areas around low test results; out of specification filter gradations appear to have been always accepted without any clear reconciliation.
- The MPMC Inspectors reported to MPMC, not to AMEC and, therefore there is no direct responsibility link with the Support Engineer.

- The AMEC Support Engineers were Engineers in Training (EIT) with limited dam construction experience.

2.2.4 MPMC Construction and Operation Responsibilities

MPMC played an integral role in the construction and operation of the TSF and some of the key observations from the interviews include the following:

- Mine rock planning: Short and long range planners were responsible for planning rock allocation to the TSF. Annual plans were prepared for approval by the Board of Directors.
- Water balance: MPMC were responsible for annual water balance assessment and determination of required dam elevation to meet tailings storage and freeboard requirements.
- Use of large trucks and over-steepened slopes: The use of the large haul trucks influenced the required width of the dam fills as the larger trucks require specific operating widths for safety reasons. This aspect, in combination with the higher dam crest required to store water, was one of the influencing factors in steepening the exterior slopes of the dam to allow the crest to be raised.
- Excavation of toe buttress: The area downstream of the breach was excavated in late fall of 2013. The excavation was to remove unsuitable foundation soils to provide a good foundation for the buttress rockfill that was to be placed in 2014. The excavation was observed by the interviewees to be up to 2 m in depth, 20 m width, along the length of the area of the breach. Conflicting observations of the dimension of the excavation was reported by a number of interviewed staff, as summarized in Table 2.1. The excavation was filled with water, and was connected to the perimeter seepage collection pond by a French drain.

Table 2.1 Summary of Observations on Buttress Excavation (MPMC Staff Interviews)

Staff	Buttress Excavation Description (approximately)[1]
[redacted]	1 m deep x 20 m wide
[redacted]	20 m x 30 m in area
[redacted]	3 m to 4 m deep; 20 m wide
[redacted]	1.4 m deep at dam toe to 4 m below road; 20 m wide
[redacted]	2 m below road
[redacted]	4 ft to 5 ft deep; 15 ft to 20 ft wide

1. Recorded in CIM interviews.

2.2.5 MPMC Employee and Contractor Interviews

Table 2.2 summarizes the interviews carried out with MPMC Senior Operating, Engineering and Environment staff. Table 2.3 summarizes interviews carried out with MPMC shifters and operating staff and Table 2.4 summarizes interviews with Contractor staff.

Table 2.2 [redacted]

[redacted]

[redacted]

Table 2.3 [redacted]

[redacted]

[redacted]

[redacted]

Table 2.4 [redacted]

2.3 Consultant Interviews

Three main consulting engineering firms were involved with the design and construction of the TSF and a summary of the key personnel over the life of the mine is shown in Table 2.5. Interview summaries with KP, AMEC and BGC are presented in Table 2.6, Table 2.8, and Table 2.9, respectively.

Table 2.5 [redacted]

[redacted]

One of the key design aspects is the understanding of the foundation geology of the PE. The foundation soils comprise glacial tills, glaciofluvial and glaciolacustrine (GLU) materials of various physical distribution and engineering properties. The main observations collected from the interviews with respect to the PE foundations include:

Knight Piésold

- "Probability or the possibility for continuous low strength layers within a flat lying lacustrine deposit was not thought to be a major concern along the Perimeter Embankment" (Interview).
- "GLU not expected in Perimeter Dam. GLU is variable and not continuous". Perimeter Embankment had 15 m of glacial till then silty material." (Interview).

- “Foundation conditions at the Perimeter dam were mainly till, removed soft soils near surface” (Interview).

AMEC

- “GLU discontinuous with varying thickness and occurrence” (Interview).
- “2011 site investigation addressed the issue of the GLU and instrumentation, concluded “it wasn’t an issue. [redacted] drove the 2011 site investigation and design review (Interview)
- “GLU in borrow pits was variably extensive, more extensive under the till in the Perimeter borrow area – to 10 m depth” (Interview)

BGC

- “believed the GLU in the Perimeter Dam was discontinuous and at depth”. “subglacial melt out tills that would be consistent with the variability of the GLU and glaciofluvial material, as they are quite heterogeneous and variable in that sort of glacial environment” (Interview).
- “GLU in the Perimeter Embankment was isolated, some 80 cm thick. Could not find evidence of pre-shearing.” ” (Interview).

The comments noted in the above section illustrate the variable degree of understanding that different design engineers had of the foundation conditions for the PE. In general, the design engineers felt that there was either no GLU deposits or that they were located at depth, or that the GLU deposits were discontinuous and of variable occurrence. The presence of actual evidence to support the assessment of the foundation conditions is addressed in detail in Section 4 of this report.

Limited laboratory testing of the GLU was carried out. KP reported carrying out tests of the GLU collected from test pits downstream of the ME, which indicated shear strengths in the low 20°s. The test results were sent to MPMC and MEM, but it does not appear that these results were provided to AMEC or BGC when these companies assumed responsibility as the Engineer of Record (EOR) for the facility. No shear strength tests were carried out on the GLU below the PE (AMEC 2012a) and shear strength was determined on the basis of correlations with index tests. No evidence of pre-shearing was observed by any of the design consultants.

Table 2.6 [redacted]

[redacted]

[redacted]

[redacted]

[redacted]

Table 2.7 [redacted]

[redacted]

Table 2.8 [redacted]

[redacted]

[redacted]

[redacted]

[redacted]

Table 2.9 [redacted]

[redacted]

[redacted]

[redacted]

2.4 MPMC Management Interviews

Corporate Staff

The corporate staff from Imperial Metals Corp. that oversaw the MPMC operations included:

- Bryan Kynoch: President
- Don Parsons: Chief Operating Officer - Corporate responsibility for the MPMC TSF
- Steve Robertson: Vice President Corporate Affairs

The corporate officers have had a history of close involvement with MPMC operations and were generally aware of the management structure and the operations of the facility. They were aware that Luke Moger reported to Art Frye with respect to the TSF. There was little or no awareness of the MAC Guidelines and how they were being implemented at the mine, other than they thought they were being implemented.

Corporate staff believed that it was MEM's responsibility to provide a "third eye" with respect to overseeing the safety of the TSF, and they believed that the EOR was responsible for the safety of the TSF. They were not aware of a risk assessment being carried out for the TSF. There seemed to be little awareness of the May 2014 overtopping event at Corner 3.

Senior Mine Staff

The senior mine staff that were interviewed included:

- Dale Reimer: Mine Manager
- Art Frye: Operations Manager
- Tim Fisch: Mine Manager from 2006 to 2013
- Wally Rennie: Senior Safety Coordinator2006 to 2014

Senior mine staff indicated they relied on Luke Moger to look after the TSF and that Luke Moger reported to Art Frye. Additionally, the current mine manager was not sure what Luke Moger's position was but relied on Luke Moger and Art Frye to look after the TSF. The mine manager was not familiar with the MAC Guidelines or the TSM initiative that Luke Moger was proceeding with. The last version of the OMS manual (2013) was unsigned by the previous mine manager and the new mine manager had limited knowledge of it.

Senior staff indicated that they relied on the EOR and that they were "comfortable" with the level of expertise of Luke Moger and the MPMC Inspectors.

Senior staff believed it was MEM's role to "tell MPMC what is right and what is wrong".

Table 2.10 [redacted]

[redacted]

[redacted]

[redacted]

[redacted]

3 STATE OF KNOWLEDGE – PRIOR TO DAM BREACH

3.1 General

The purpose of this section is to document the state of knowledge that was available and used by the design consultants for the design of the dam, prior to the breach. This section includes a summary of the following:

- timeline of Construction and Design Studies;
- summary of the site investigations carried out over the period of 1989 to 2012;
- geology model(s) developed by the consultants and selection of the design cross sections for stability analyses;
- geotechnical parameters used for design;
- design criteria; and
- stability analyses carried out by the consultants for the Stage 6, Stage 9 and Stage 10 dam elevations.

This section is a factual summary based on the various reports prepared over the life of the facility.

3.2 Timeline of Construction and Design

Embankment construction at the TSF was divided into “stages” for the purposes of design, permitting and construction. Design updates were issued periodically by the design engineers. Since TSF raises were permitted incrementally by MEM, the design documents were submitted to the MEM by MPMC for permitting support.

A TSF site options assessment and feasibility study was carried out in 1989 by KP and a preferred tailings site was selected. The first detailed design report was completed in 1995, after which construction drawings were issued. Construction of the TSF began in May, 1996 with the first structural fill placed in August, 1996. In October 2001, production at the mine ceased due to economic reasons and the TSF was placed in care and maintenance. In August 2004, mine production resumed and the TSF went back into operation. Stage 9a construction, which constituted a raise of the facility from El. 967 m to El. 970 m, was ongoing and nearly complete in the summer of 2014 when the PE was breached. Major milestones in the construction and operation of the TSF are shown on a timeline in Figure 3.1.

The design of the TSF was sub-contracted by MPMC to an external engineering consultant throughout operations. In addition to the design of the TSF, the consultant also performed the following functions:

- regular dam safety inspections and preparation of dam safety inspection reports;
- preparation of annual construction summary/record reports;

- construction QC and QA support (the degree of support varied over the years); and
- preparation of construction documents.

The principal engineer from the consulting company also assumed EOR responsibility for the facility. The companies involved in the design of the facility are summarized in Table 3.1. AMEC took over from KP in 2011 and BGC was scheduled to take over as EOR for the Stage 10 raise, above El. 970 m, which was not reached by the time of failure.

Table 3.1 Summary of Design Consultants and Engineers of Record

Design Engineering Company	Year	Design Stages	Dam Elevation[1] (m)	Senior Design Engineer	Engineer of Record
Knight Piésold.	1995 to 2010	1 to 6	958	Ken Brouwer, P.Eng.	Ken Brouwer, P.Eng.
AMEC Earth and Environmental	2011	7	960	Todd Martin, P.Eng.	Todd Martin, P.Eng.
AMEC Earth and Environmental	2012	8	963.5	Todd Martin, P.Eng.	Daryl Dufault, P.Eng.
AMEC Earth and Environmental	2013	8 to 9	967	Steve Rice, P.Eng.	Laura Fidel, P.Eng.
AMEC Earth and Environmental	2014	9	970	Andrew Witte, P.Eng.	Steve Rice, P.Eng.
BGC Engineering Inc.	2014	10	972.5	Todd Martin, P.Eng.	

Note: 1) Natural ground elevations vary from approximately 915 m to 930 m.

On several occasions, external consultants performed independent technical reviews of the TSF design or performed dam safety reviews (DSR). The reports issued by external reviewers are summarized in Table 3.2.

Table 3.2 Summary of External Geotechnical Reviews

Reviewer	Report Title	Date of Report Issue
C.O. Brawner Engineering Ltd.	"Mt. Polley Tailings Dam" – memorandum issued to Tim Eaton, P.Eng. Manager, Geotechnical Engineering, Mine Review and Permitting Branch, Ministry of Energy, Mines and Petroleum Resources. Review carried out on the Stage 1 Starter Dam. (C.O. Brawner 1995a)	October, 1995
MAJM Corporation Ltd.	"Geotechnical Review, Drainage Aspects Main Embankment Tailings Storage Facility Mt. Polley Project, British Columbia" Review was carried out on the Stage 1 Starter Dam. (MAJM 1997)	March, 1997
AMEC	"Final Report- Dam Safety Review – Mount Polley Mine – Likely, British Columbia". (AMEC 2006)	December, 2006

From Stages 1 to 6, construction QC testing and QA inspections were carried out by KP personnel on a full-time basis. An exception to this was made in the later years of the Stage 6 raise where some QC work was subcontracted to a local testing company (GeoNorth Engineering).

Beginning in Stage 7, the day-to-day QC was taken over by MPMC personnel reporting to the MPMC Project Manager with QA technical support from AMEC. The key roles included:

- On site QC was carried out by summer students working for MPMC. AMEC personnel spent time on site at the beginning of the construction seasons to train and audit MPMC QC staff.
- QC support was provided by AMEC technical personnel from Prince George.
- AMEC made additional trips to site as required for QA overview and to oversee critical aspects of construction such as the embankment core key trench construction.

This arrangement continued until the TSF breach.

BGC started carrying out engineering projects on the TSF in 2012 and were transitioning to become the EOR for the Stage 10 construction, scheduled to begin in August 2014. A timeline summary of the dam raise stages and corresponding engineering firms is presented on Figure 3.1.

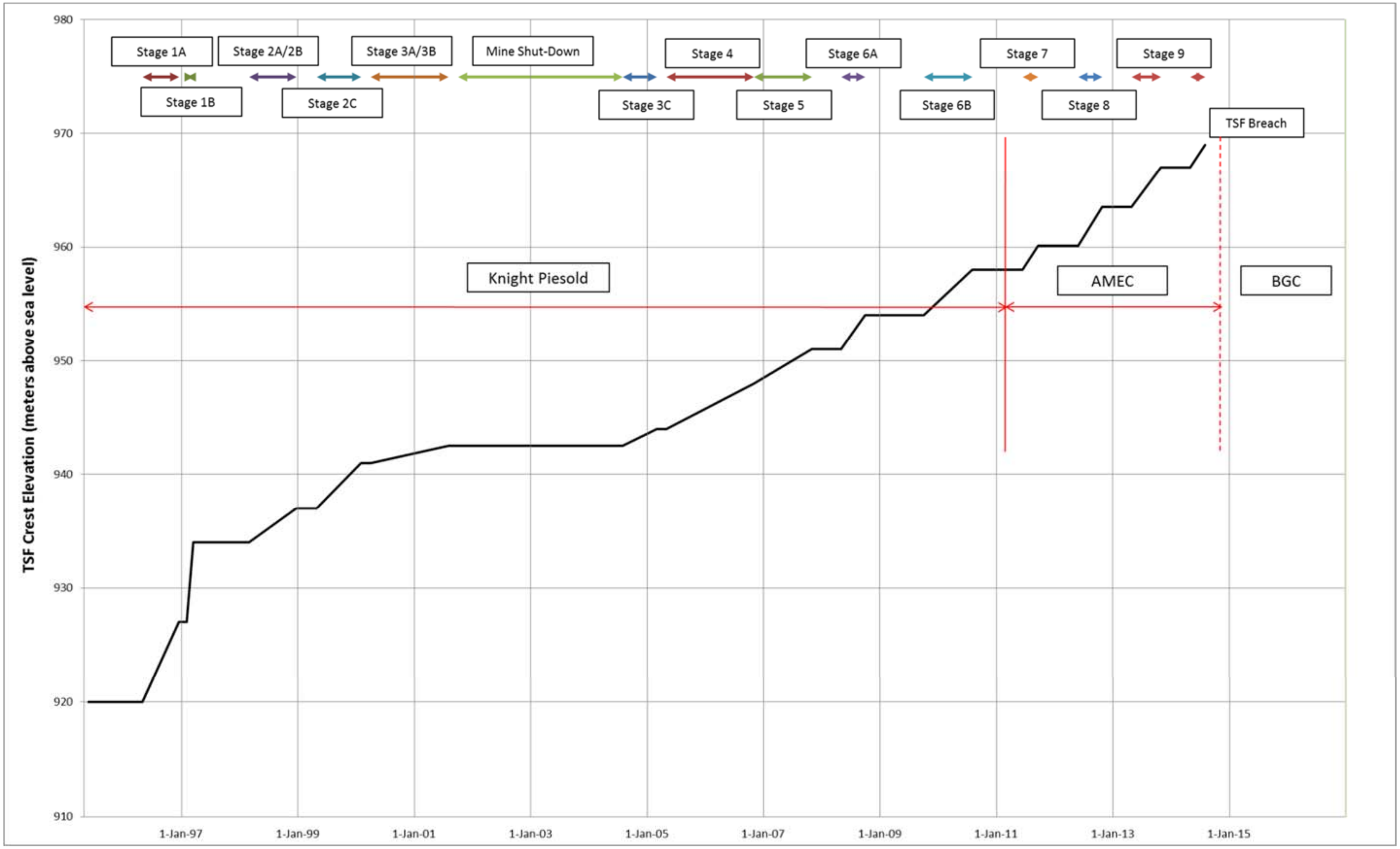


Figure 3.1 Timeline Summary of Project Stages

3.3 Site Investigations (1989-2012)

3.3.1 General

Site investigations in the vicinity the TSF were carried out prior to the TSF construction and as part of ongoing design during operations. The investigations were performed for a variety of purposes which included: embankment foundation characterization, borrow source determination, basin liner delineation, hydrogeological characterization and instrumentation installation. The investigations comprised test pitting and drilling by a variety of methods. Investigations carried out by each consultant are discussed in the following sections. A summary of the site investigations and the reference reports used in preparing this section are presented in Table 3.3 and Table 3.4.

3.3.2 Knight Piésold

1989 Investigations

The first site investigations performed by KP at the TSF were undertaken as part of the options study in 1989 and MPMC condemnation drilling. The investigation included deep drill holes and test pits inside and outside of the TSF basin. The drill holes were advanced using a tricone bit through the overburden and were diamond cored through the bedrock. Direct sampling was not carried out and the soil stratigraphy was interpreted from the drill cuttings. One drill hole, DH89-231, was drilled near Station 4+175 in the PE.

Test pits were excavated to a maximum depth of approximately 5 m. Samples were collected from two of the test pits for index testing.

1995 Investigations

A test pit program was undertaken in 1995 to provide information for the initial design of the TSF and locate fill borrow sources for construction. Initially, eleven pits were excavated inside the TSF, with one, TP95-27, excavated within the footprint of the PE. In response to external review comments on the TSF design (C.O. Brawner 1995a), an additional 40 test pits were excavated in the ME footprint and around the perimeter of the ME Seepage Collection Pond (SCP) to gain additional information on foundation conditions. Soil samples were collected from select pits from the first round of investigation for laboratory index testing and soil strength testing.

1996 and 1997 Investigations

Investigation data collected in 1996 and early 1997 were presented in the KP Updated Design Report (KP 1997b). Drilling programs during 1996 and 1997 comprised the following:

- 6 groundwater monitoring wells were installed around the perimeter of the TSF, downstream of the embankments. The holes were drilled through overburden with a tricone bit and Standard Penetration Tests (SPTs) were performed at 3.1 m to 6.1 m intervals in most holes. GW96-1 and GW96-2 were drilled downstream of the PE at stations 4+020 and 3+260, respectively.

- Vibrating wire piezometers (VWPs) were installed in the foundation of the ME at 3 locations. The holes were drilled with a hollow stem auger. Continuous SPTs were performed in the top 10 m and samples were collected. However, sample testing results, if any, were not reported.
- 4 pressure relief wells were drilled using a solid stem auger in the foundation of the ME. 2 samples collected in the GLU were tested for moisture content.
- 5 seismic Cone Penetration Tests (CPT) soundings were performed in the foundation of the ME.
- 4 pressure relief trenches were excavated in the ME foundation, however data collected during the excavations were not reported.
- 25 shallow holes were drilled in Borrow Area No. 1 (Original Borrow Area) within the TSF basin to determine the maximum depth of the glacial till. The holes were drilled with solid and hollow augers. Split spoon and auger flight grab samples were taken for laboratory testing but testing results were not reported.

A large number of test pits were excavated in 1996/early 1997 both inside and outside the TSF basin primarily to determine the required extent of the natural basin liner and to assess the suitability of construction fill borrow sources. The number of test pits excavated and their locations are summarized below:

- 19 east of the PE and 14 at the west end of the reclaim barge channel to locate a borrow source for filter sand;
- 28 in the upper natural basin liner area, upstream of the ME near the abutment with the SE;
- 14 in the lower natural basin liner area, upstream of the ME;
- 13 in Borrow Area No. 3 (Alternate Borrow Area) southeast of the TSF;
- 4 in the reclaim barge channel;
- 2 downstream of the ME toe; and
- 83 in Borrow Area No. 1. 20 samples were collected from the excavations for index testing.

After the Updated Design Report was issued, additional test pits were excavated in 1997 to investigate borrow areas and to gather more information for the PE foundation conditions. The investigations included:

- 21 test pits in Borrow Area No. 4, within the TSF basin upstream of the PE. Moisture content samples were collected in each of the pits.
- 18 test pits in Borrow Area No. 2 (Future Borrow Area), located SE of the TSF downstream of the ME. Moisture content samples were collected in each of the pits.
- 22 test pits in Borrow Area No. 3. Moisture content samples were collected in each of the pits.

- 4 test pits in the PE foundation. The pits were excavated to a depth of approximately 5 m and moisture content samples were taken. No GLU was encountered in these pits.

1998

Investigations in 1998 were focused on borrow area determination and delineation of the basin liner extent. The investigations comprised:

- 12 drill holes in Borrow Area No. 4. Drill rig type and drilling logs were not reported by KP. 3 samples were collected for index testing.
- 11 drill holes in Borrow Area No 2. Drill rig type and drilling logs were not reported by KP. 4 samples were collected for index testing.
- 19 drill holes upstream and within the footprint of the SE to delineate the basin liner area. Drill holes were advanced to a maximum depth of approximately 7.5 m; the drilling method was not reported. 3 samples of glacial till were collected for index testing. 1 sample of GLU was collected for moisture content determination.

1999

Site investigation in 1999 included:

- 44 drill holes upstream of the SE near the right abutment to further delineate the natural basin liner extent. The depth of drilling varied but the maximum depth achieved was approximately 9 m. The drilling method was not reported by KP. 14 samples of glacial till and 2 samples of GLU were collected for index testing.
- 91 shallow holes drilled around the perimeter of the TSF to "evaluate the potential for seepage infiltration into foundation materials during hydraulic placement of cycloned sand" (KP 2000b, pg. 7). Drilling depth was variable but a maximum depth of 7.5 m was achieved. 23 of these holes were located just downstream or within the PE footprint with some within the future breach area. SPTs were performed in most holes, however no laboratory testing data was reported.
- 23 CPT soundings upstream of the PE and ME and downstream of the PE to assess the properties of the cycloned sand trial berms and the tailings beach.

2000

3 holes were drilled downstream of the SE to install groundwater monitoring wells. The holes were advanced to a depth of 21 m to 24 m using the ODEX drilling method. SPTs were performed in 2 of the holes. No soil samples were tested.

2001

Holes were drilled in 2001 for instrumentation installation and borrow source determination:

- 2 slope inclinometers were installed at the downstream toe of the ME to monitor foundation movements. The installations were drilled using the ODEX method to a depth of 24.5 m and 30.5 m. SPTs were performed at regular intervals through the overburden. No soil samples were taken for laboratory testing.
- 66 shallow holes were drilled southeast of the TSF near Borrow Area No. 2 to assess the suitability of the till in the area for construction material. Hole depth ranged from approximately 1 m to 12 m with moisture content samples collected in the majority of the holes.

2006

Three slope inclinometers were installed at the downstream toe of the ME. The instruments were installed in diamond drilled boreholes that were advanced to bedrock with depths ranging from 35 m to 42 m. SPTs were performed at regular intervals. Soil samples were collected using a Shelby tube sampler in 2 of the 3 holes, and disturbed samples were collected from the SPTs. Select samples were tested for index properties. One consolidation test was performed on a Shelby tube sample collected in SI06-02. It is unclear from the logs whether the soil samples would be described as glacial till or GLU.

2007

Two brass tube samples were collected in the GLU at approximately 2.5 m to 3.0 m depth in a test pit downstream of the ME adjacent to the ME SCP. Direct shear testing was performed on the samples. Index testing on the samples was limited to fines content (KP 2014, KP 2007c).

2008

A drilling program was undertaken to investigate a potential construction fill borrow source downstream of the PE. 11 holes were drilled by the sonic method to depths ranging from 11 m to 24 m. SPTs were performed periodically in most of the holes. Samples were collected but laboratory test results, if any, were not reported.

Table 3.3 Summary of Knight Piésold Geotechnical Investigations

Year	Type	Naming Convention	General Location[1]	Purpose of Investigation	Reference Report
1989	Test Pits	TPBxx	ME, PE, SE	Basin Characterization	KP 1990
1989	Drilling	MP89-2xx	ME, PE, SE	Overburden and Bedrock Characterization	KP 1990
1995	Test Pits	TP95-xx	ME, PE	Embankment Foundation and Basin Characterization	KP 1995a
1995	Test Pits	TP95ME-xx and TPS-xx	ME	Foundation and Seepage Collection Pond Characterization	KP 1996a
1996	Drilling	GW96-x	ME, PE, SE, Outside of TSF	Groundwater Monitoring Well Installation	KP 1997a

Year	Type	Naming Convention	General Location[1]	Purpose of Investigation	Reference Report
1996	Test Pits	96-xx	Natural basin Liner , Borrow Area No. 1 and No. 3, Reclaim barge area, Downstream of ME, Filter Sand Borrow Area East of PE	Basin Liner Delineation, Borrow Area Characterization, Foundation Characterization	KP 1997b
1996	Test Pits	Grid Convention (number/letter)	Borrow Area No. 1	Borrow Area Characterization	KP 1997b
1996	Drilling	PRW96-x	ME	Pressure Relief Well Installation	KP 1997b
1996	Drilling	96-A/B/C	ME	Vibrating Wire Piezometer Installation	KP 1997b
1996	Drilling	BH-xxx (Grid Convention)	Borrow Area No. 1	Borrow Area Characterization	KP 1997b
1996	CPT	CPT 96-x	ME	Foundation Characterization	KP 1997b
1997	Test Pit	TP97-PETP-x	PE	Foundation Characterization	KP 1997d
1997	Test Pit	TP97-BA4-xx	Borrow Area No. 4	Borrow Area Characterization	KP 1997d
1997	Test Pit	TP-xxF and TP-BGx	Borrow Area No. 2	Borrow Area Characterization	KP 1997d
1997	Test Pit	TP-xxA	Borrow Area No. 3	Borrow Area Characterization	KP 1997d
1998	Drilling	DH98-BA2-x	Borrow Area No. 2	Borrow Area Characterization	KP 1999b
1998	Drilling	DH98-BA4-x	Borrow Area No. 4	Borrow Area Characterization	KP 1999b
1998	Drilling	DH98-BL-x	Natural Liner Area Upstream of SE and in SE Footprint	Basin Liner Delineation	KP 1999b
1999	Drilling	DH99-x	Natural Liner Area Upstream of SE and in SE Footprint	Basin Liner Delineation	KP 1999b
1999	Drilling	DH99-x	Downstream of ME, PE, SE	Foundation Hydraulic Conductivity Assessment	KP 2000b
1999	CPT	CPT99-XX	ME, PE	Characterize trial cycloned sand berms and tailings beaches.	KP 1999c
2000	Drilling	GW00-x	Downstream of SE	Groundwater Monitoring Well Installation	KP 2001
2001	Drilling	SI01-xx	ME	Slope Inclinometer Installation and Foundation Characterization	KP 2001
2001	Drilling	DH01-xx	Borrow Area No. 2	Borrow Area Characterization	KP 2001
2006	Drilling	SI06-xx	ME	Slope Inclinometer Installation and Foundation Characterization	KP 2007a
2008	Drilling	KP08-xx	Downstream of PE	Borrow Area Characterization	KP 2009c

Notes: ME = Main Embankment; PE = Perimeter Embankment; SE = South Embankment

3.3.3 AMEC

2011

When AMEC assumed engineer of record responsibility in early 2011, 3 additional slope inclinometers and 11 vibrating wire piezometers were installed along monitoring sections around the TSF. 8 holes were drilled in the ME foundation, 4 in the PE foundation (at three locations) and 2 in the SE foundation. The holes were drilled using the sonic coring method with soil grab samples taken at regular intervals for laboratory index testing. Depth of drilling ranged from approximately 11 m at the SE to 49 m at the ME.

2012

Two slope inclinometers were installed downstream of the PE. One was positioned adjacent to the previously installed SI11-04 which was experiencing "compression failure deformation". The second was installed near Sta. 3+270, nearby VW11-09. AMEC did not issue final drill logs for the 2012 SI installations; only hand written field logs and simplified logs included in the inclinometer reading plots are available.

Table 3.4 Summary of AMEC Geotechnical Investigations

Year of Investigation	Type of Investigation	Naming Convention	General Location[1]	Purpose of Investigation	Reference Report
2011	Drilling	SI11-xx	ME, PE	Slope Inclinometer Installation and Foundation Characterization	AMEC 2012a
2011	Drilling	VW11-xx	ME, PE, SE	Vibrating Wire Piezometer Installation and Foundation Characterization	AMEC 2012a
2012	Drilling	SI12-xx	PE	Slope Inclinometer Installation	AMEC 2013a AMEC 2014b

Notes: ME = Main Embankment; PE = Perimeter Embankment; SE = South Embankment

3.4 Pre-Breach Geology Interpretation

3.4.1 General

The TSF site has a complex Quaternary glacial history which has resulted in the deposition of soil units that can be broadly classified as till, glaciofluvial and glaciolacustrine in origin. These units are widely distributed across the TSF area, both spatially and in elevation. Quaternary sediments are underlain by bedrock, which is of variable origin and composition within the TSF basin. This section provides a summary of the factual data from the consultant's site investigation reports pertaining to the site geology.

Quaternary geology was interpreted by KP, AMEC and BGC. A summary of their site-wide terminology/interpretations are shown schematically on Figure 3.2 and are discussed in the following sections. Terminology was not always consistently applied in the design reports and the spatial distribution of the deposits is complex. As a result, this summary should be regarded as a relatively broad generalization of the main units.

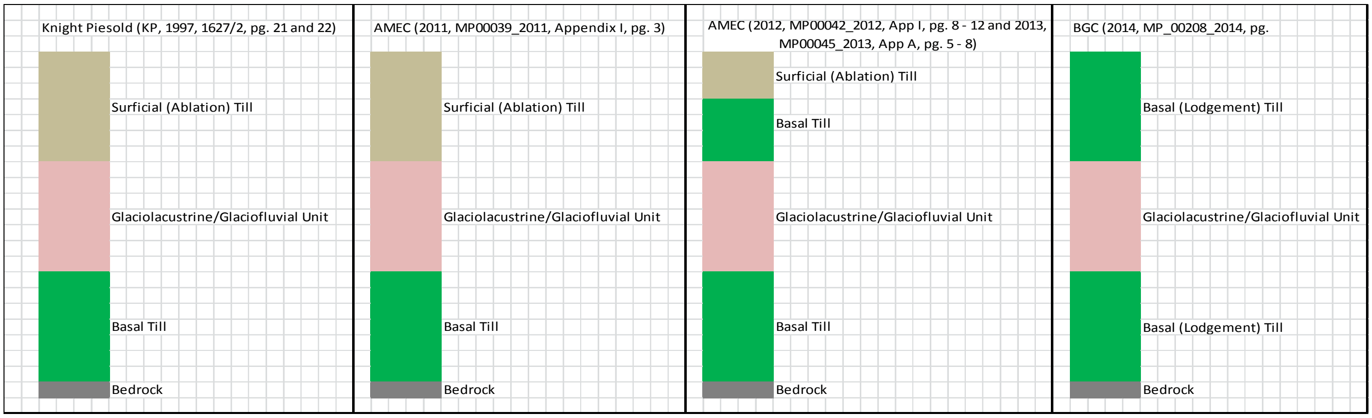


Figure 3.2 Classification of Glacial Soils (KP, AMEC, BGC)

3.4.2 Knight Piésold

KP described the Quaternary stratigraphy on site as follows, in order of increasing age (KP 1997b, pg. 20-24):

> ***Surficial (Ablation) Till Unit:***
> *A surficial layer of glacial till underlies all areas of the tailings basin investigated to date. This glacial till is typically comprised of 50 to 65 percent sandy silt (passing the No. 200 sieve). The surficial till unit is believed to be a melt-out or Ablation Till. It is commonly slightly weathered, firm to stiff, and wet over the top 0.5 to 1.0 metres in the lower areas of the tailings basin. The till is very stiff and is moist to very moist below 1.0 to 2.0 metres depth and at higher elevation. No appreciable fissuring was observed in the surficial till unit, likely due to the shallow groundwater table, which is typically less than 0.3 metres below the ground surface.*
>
> ***Glaciolacustrine/Glaciofluvial Unit:***
> *A glaciolacustrine/glaciofluvial sedimentary unit underlies the surficial glacial till. The unit is primarily comprised of glaciolacustrine layers (silt, some clay), with lesser fine grained glaciofluvial layers (sand). The glaciolacustrine/glaciofluvial sequence thickens from west to east and from north to south and terminates at El. 928 m (approx.). It is not present along the right abutment where the surficial till directly overlies bedrock. The glaciolacustrine/glaciofluvial sequence also appears to transform from a continuous sequence near the Main Embankment into thin (0.5 to 3.0 metre) layers within the glacial till unit to the northwest. The glaciolacustrine/ glaciofluvial sequence is generally 6 to 8 metres thick at the west and increases to as much as 25 metres towards the eastern edge of the tailings basin.*
>
> *The glaciolacustrine/glaciofluvial sequence consists predominantly of interbedded layers of silt with either clay or fine sand. The glaciolacustrine (silt, clay) sediments*

are often highly overconsolidated and very stiff to hard. The glaciolacustrine unit has a low permeability as a result of the fine-grained composition. Within the glaciolacustrine sediments, occasional seams of fine sand with only a trace of silt are present. These seams vary in thickness from 0.1 metres to greater than 3 metres.

Basal Till

The glaciolacustrine/glaciofluvial sequence is underlain by a very dense, well graded silt and sand glacial till unit believed to be a basal till. The basal till unit dips and thickens slightly from west to east and north to south, likely following bedrock topography. It is typically 10 to 20 metres thick, massive, highly consolidated and contains some gravel and trace to some clay. The basal till has a low permeability, estimate to be less than 10^{-6} cm/s.

A geological cross-section (Section 8), looking northwest to southeast, across the Perimeter Embankment is shown on Figure 3.3. The location of the section is shown in plan on Figure 4.2. The cross section (Section 8) identifies the presence of glaciolacustrine soils within the upper tills.

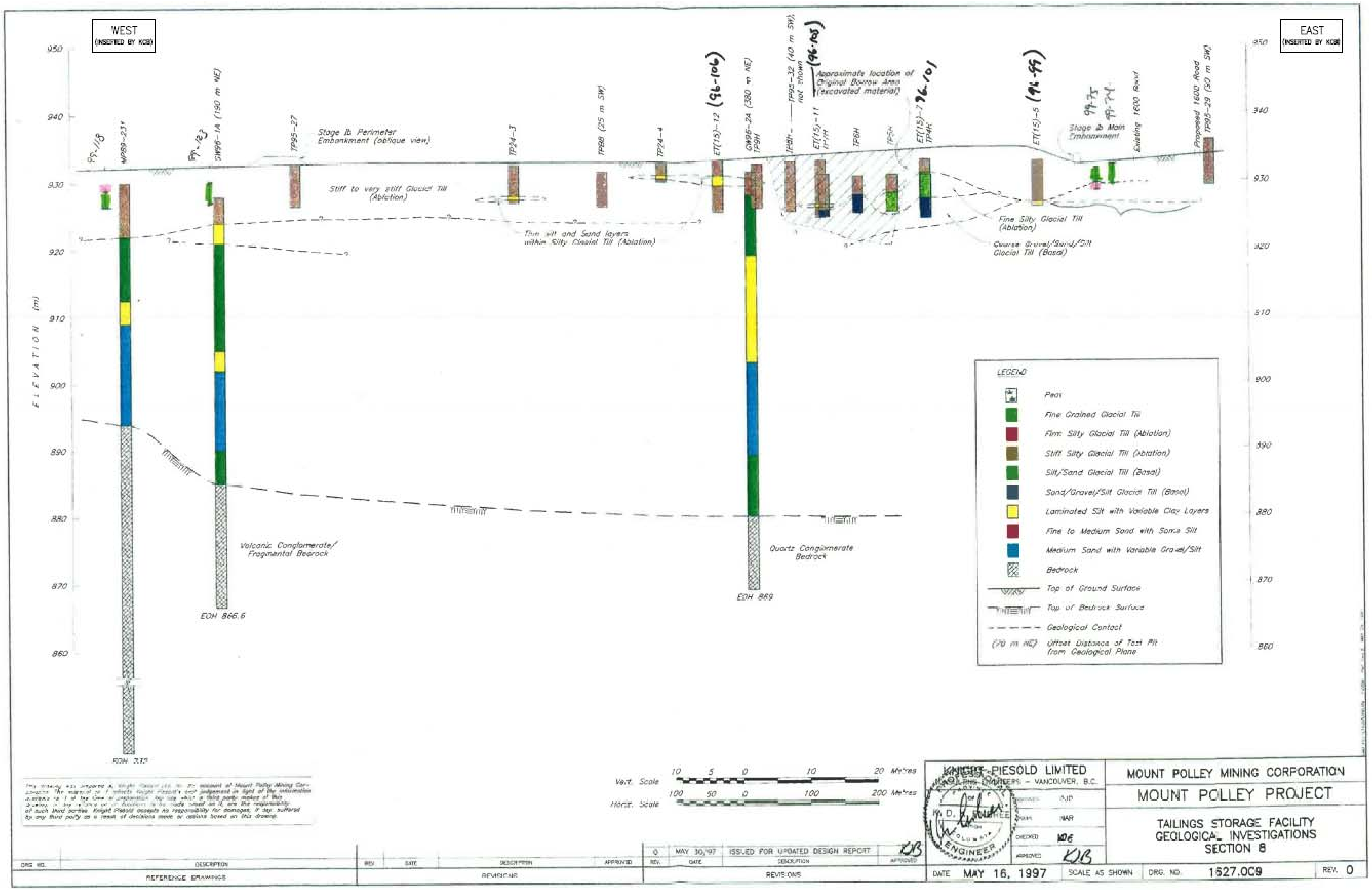


Figure 3.3 KP Geological Section Perimeter Embankment Area – Section 8 (KP 1997b)

KP summarized the foundation conditions below the embankments in the Stage 6 Design Report (KP 2007b, pg. 4) as follows:

> *The foundation conditions at the Main Embankment consist of low permeability glacial till material at surface underlain by fluvial and lacustrine silts up to 20 m thick. The foundation piezometers at the Main Embankment indicate that this area has slight artesian conditions (less than 3.0 m).*
>
> *The foundation conditions at the Perimeter Embankment consist of low permeability glacial till throughout that is generally in excess of 5 m thick.*
>
> *The foundation conditions at the South Embankment consist of a relatively thin, low permeability glacial till material overlying bedrock. Details of the site geological investigations can be found in the Knight Piesold Report "Updated Design Report", Ref. No. 1627/2, June 6, 1997.*

Differences were noted between the foundation stratigraphy assumed below the embankments for stability analysis and the site geological model discussed above. In their stability analyses, KP used the nomenclature "basal till" to describe the unit above the glaciolacustrine sediments or bedrock in the PE and SE foundations, respectively (KP 2005a).

The same assumption was made later in the Stage 6 Design Report, except glaciolacustrine sediments were assumed to directly underlie the basal till below the SE instead of bedrock (KP 2014). Below the ME, "loose to medium dense till" and "dense till" (KP 2005a), or "loose to medium dense till" and "dense to very dense till" were used to describe the till above the glaciolacustrine sediments (KP 2014). Till below the glaciolacustrine sediments was typically described as basal.

The geologic models adopted for stability analysis of the Stage 6 embankments are shown on Figure 3.4 and Figure 3.5 for the ME and PE, respectively. The stability models were not included in the original issue of the Stage 6 Design Report in 2007. New calculations were provided in December 2014 to KCB to support the MEM investigation. The model for the PE includes a glaciolacustrine layer at depth that was not identified in the text referenced in the above section of the Stage 6 Report.

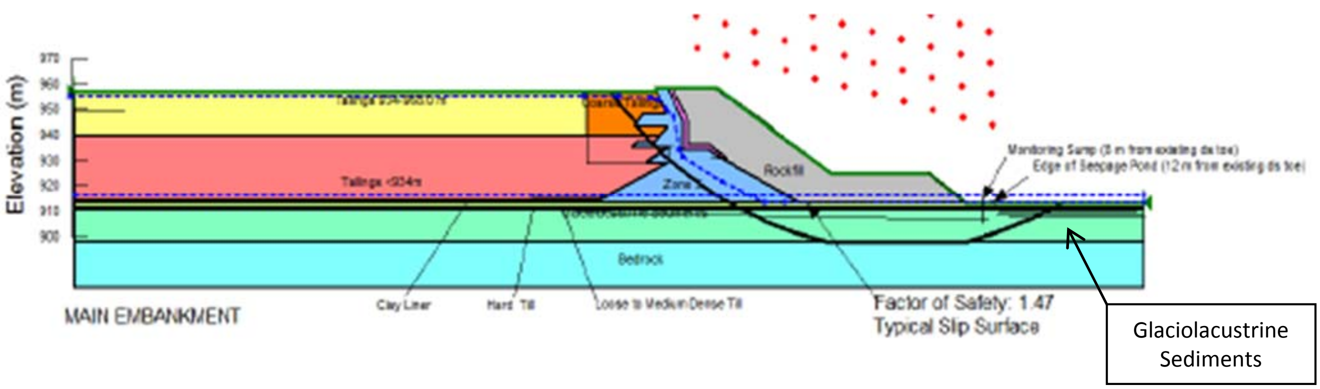


Figure 3.4 KP Geologic Model for the Main Embankment used for Stage 6 Report (KP 2014)

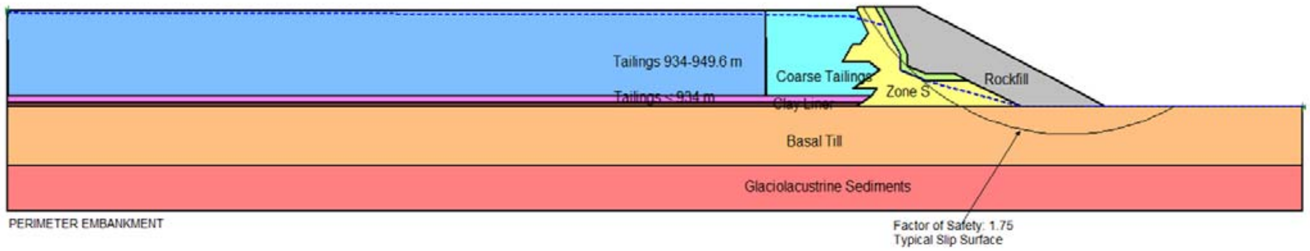


Figure 3.5 KP Geologic Model for the Perimeter Embankment used for Stage 6 Report (KP 2014)

3.4.3 AMEC

AMEC appeared to have adopted KP's generalized site stratigraphy for their embankment foundation conditions in the Stage 7 design report (note that the Stage 7 report was prepared prior to the AMEC site investigations that were carried out later in 2011). However, instead of using the consistency of the till to describe the near-surface till deposits below the ME, AMEC adopted the term "Ablation Till". Below the PE, AMEC assumed that a layer of glaciolacustrine/glaciofluvial sediments was present at surface, underlain by basal till. This is the reverse of what KP assumed below the PE. AMEC adopted the same strength parameters for the basal till and the glaciolacustrine/glaciofluvial sediments below the PE. Below the SE, basal till was assumed to directly overly bedrock (AMEC 2011, Appendix A).

For the Stage 8 stability analysis, AMEC updated their stratigraphy assumption below the PE and assumed that the embankment was directly underlain by basal till. Below the uppermost till layer, two layers of glaciolacustrine/glaciofluvial sediments were identified, separated by a thin layer of basal till (AMEC 2012c).

For the Stage 9 design, an additional ME stability section was introduced. The previous section was located at Sta. 2+060 and the new section was located at Sta. 1+850. The same general stratigraphic model was carried over for Sta. 2+060, with ablation till overlying glaciolacustrine/glaciofluvial sediments. However, at the new section the surficial till was described as basal (AMEC 2013b). It is noted that for both the Stage 8 and Stage 9 analyses, the lower relative strength that was applied to the ablation till in Stage 7 does not appear to have been used. Stability parameter summary tables indicate that the ablation till and the basal till were assigned the same strength values.

The geologic models used for the Stage 9 stability analyses are shown on Figure 3.6 and Figure 3.7 for the ME and PE, respectively.

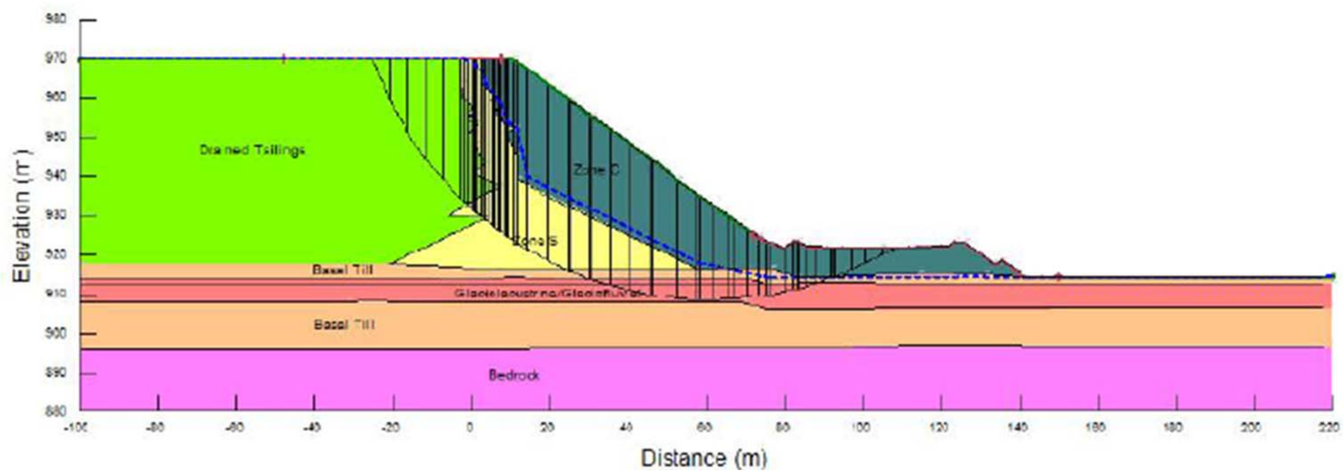


Figure 3.6 AMEC Geologic Model used for Stability Analysis of Stage 9 Main Embankment (1+850) (AMEC 2013b)

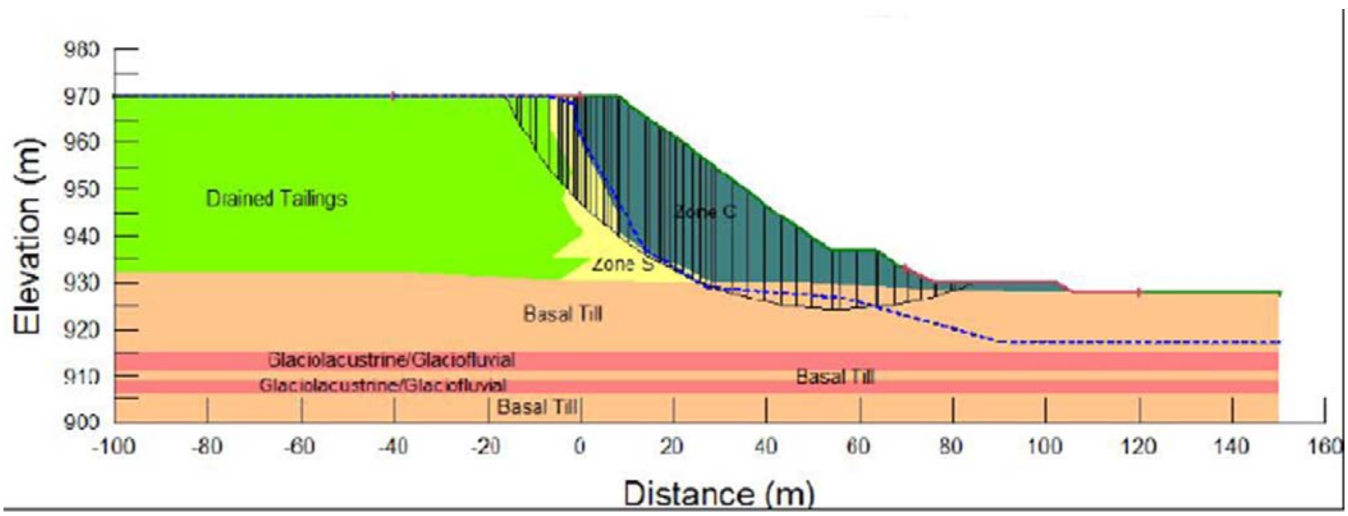


Figure 3.7 AMEC Geologic Model used for Stability Analysis of Stage 9 Perimeter Embankment (3+990, Section D) (AMEC 2013b)

3.4.4 BGC

BGC adopted the same general site geology model as KP and AMEC. However, BGC described the till to be basal and lodgement in origin, and no distinction was made between the properties of the till with respect to depth below ground surface or elevation (BGC 2014). Stage 10 stability sections with the foundation stratigraphy assumptions for the ME and PE are shown on Figure 3.8 and Figure 3.9.

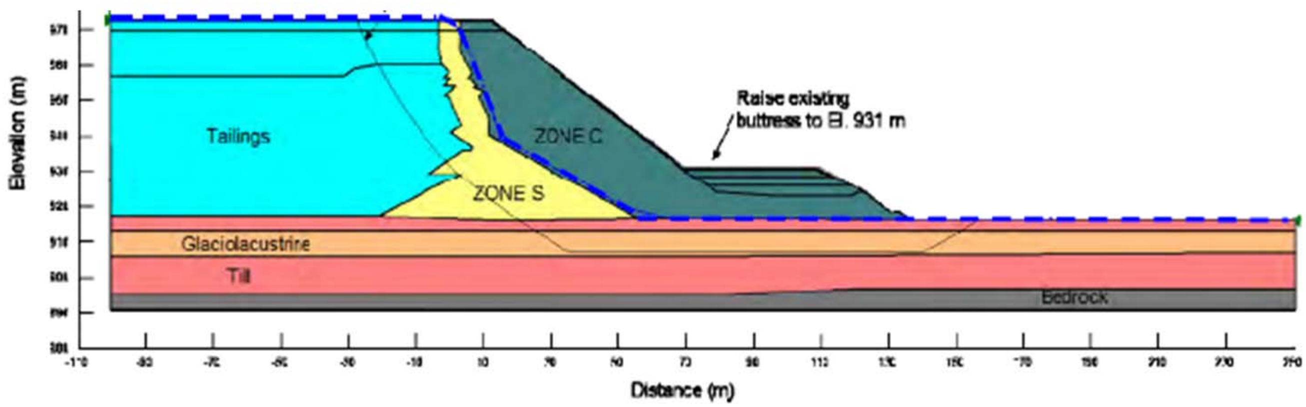


Figure 3.8 **BGC Geologic Model used for Stability Analysis of Stage 10 Main Embankment (1+900) (BGC 2014)**

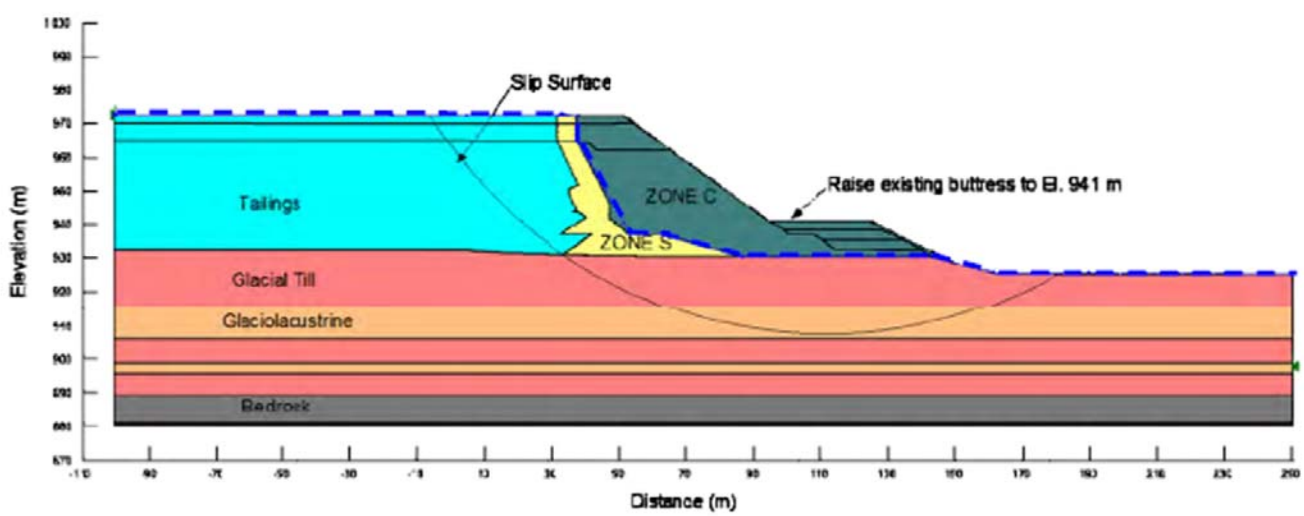


Figure 3.9 **BGC Geologic Model used for Stability Analysis of Stage 10 Perimeter Embankment (3+990, Section D) (BGC 2014)**

3.4.5 Summary

The generalized Quaternary geologic model proposed by KP was subsequently adopted by AMEC and BGC. Although different assumptions were often made regarding the origin and properties of the till units, the following general geologic mode was generally used:

- **Upper Tills**: A package of glacial till sediments that blanket the site. Thickness of the unit is variable and thins out within the main basin valley, which runs below the ME. The tills are basal in origin.

- **Glaciolacustrine/Glaciofluvial Units**: The tills are interlayered with glaciolacustrine and glaciofluvial sediments, which vary from a thick, continuous unit below the ME to thinner units interlayered with the upper tills. The presence of these thinner, discontinuous glaciolacustrine and glaciofluvial units within the tills below the PE was noted by both AMEC and KP (AMEC 2012a, pg. 24) (KP 1997b).
- **Lower Till**: The glaciolacustrine/glaciofluvial sediments are underlain by a package of dense basal till.

It is of particular note that although the glaciolacustrine/glaciofluvial units were understood to terminate at a top elevation of 928 m (KP 1997b) no stability analysis carried out by any consultant assumed that a glaciolacustrine/glaciofluvial unit existed in the embankment foundations above El. 915 m. The inherent assumption being that the upper glaciolacustrine/glaciofluvial units were not continuous enough to control stability and, therefore, could be ignored. There are two notable exceptions:

- In the Stage 6 design report KP assumed that the SE was underlain by a layer of basal till overlying a glaciolacustrine unit. Although elevations were not given on the analysis section, scaling of the model suggests that the top elevation of the glaciolacustrine sediments is above 915 m (KP 2014). This interpretation of the SE foundation stratigraphy appears inconsistent with what was assumed by KP previously. Previous analysis assumed that a thin layer of basal till overlay shallow bedrock (KP 2005a).
- Glaciolacustrine/glaciofluvial sediments, approximately 17 m thick, were assumed to be directly underlying the PE in the AMEC Stage 7 Design Report (AMEC 2011). The top elevation of this unit was approximately 932 m. Since no additional site investigation work was carried out in the period between when this analysis was done and the issue of KP's Stage 6 Design Report, the basis behind this significant reinterpretation of the PE foundation conditions is not clear. It is noted that in the Stage 8 design report, AMEC assumed that the top elevation of the glaciolacustrine/glaciofluvial unit below the PE was 915 m.

However, there is a significant amount of evidence to support the existence of glaciolacustrine sediments above El. 915 m. The distribution of glaciolacustrine soils is further discussed in Section 4.2 of this report.

3.5 Design Basis – MPMC Consultants

3.5.1 General

The TSF design criteria evolved as the facility was constructed in response to new information gathered during site investigation and construction and modifications to the mine plan. Table 3.5 provides a summary of the some of the main design criteria for the TSF. The evolution of design criteria is discussed in the following sections.

3.5.2 Knight Piésold

Consequence Classification

Consequence rating guidelines suggested by the Canadian Dam Safety Association (CDSA), later the Canadian Dam Association (CDA), were used to classify the hazard posed by the Mount Polley TSF and provide guidance in the selection of appropriate design criteria. Initially, the TSF was assigned a "low" consequence rating for operations and a "high" consequence rating for closure (KP 1995b, pg. 22; KP 1997b, pg. 26). The operations classification was subsequently upgraded to "high" in 2005 to account for the economic losses that would be incurred by Imperial Metals should the TSF embankments fail or become inoperable (KP 2005a, pg. 4). The hazard classification was reviewed as part of the DSR in 2006, and it was suggested that the "low" consequence rating be adopted since it was believed that mine owners' losses should not be considered in the consequence classification (AMEC 2006, pg. 18). These recommendations were adopted by KP in the Stage 6 design report (KP 2007b, Table 3.1).

The CDA updated their consequence classification rating system in 2007 and the newly created "significant" classification was considered most appropriate for Mt. Polley. The adoption of the "significant" rating did not require changes to the design criteria. (KP 2009a, pg. 3). However, it is noted by KCB that in the revised CDA guidelines, the 1 in 1,000 year earthquake was suggested for earthquake design ground motions corresponding to a "significant" consequence rating (CDA 2007). Previous releases of the guidelines suggested using between the 1 in 100 year and 1 in 1000 year ground motions for "low" consequence structures.

Hydrologic Design Criteria

The design flood criteria for the facility remained consistent throughout most of KP's tenure as design engineers. The facility was designed to store the 24-hour probable maximum precipitation (PMP) in addition to the assumed operating pond volume. Embankment crest elevations were set to achieve a minimum of 1.0 m of freeboard above the design flood volume. The TSF was also designed to have capacity to store the 10-day PMP, albeit with reduced freeboard. For these reasons an operations spillway was not deemed necessary. Closure spillways were designed to route the probably maximum flood (PMF), which is consistent with the CDA hazard classification of "high", adopted for closure.

The magnitude of the design flood event was modified in Stage 6 to correspond to the 72-hour PMP event. Concurrently, the wave run-up requirement was re-evaluated and reduced to 0.7 m. The additional depth resulting from revised design flood was effectively negated by the reduction in the wave run-up depth (KP 2007b, pg. 7).

Water Balance

Initial design reports assumed that the TSF would operate in a water balance deficit and that additional makeup water may be required (KP 1997b, pg. 60). Around the time of the mine re-start, KP noted that the results from their updated water balance model "indicate that the mine site is currently moving from a deficit to a surplus condition and the water must be stored on site to meet the current effluent permits" and they anticipated that the majority of the water would require storage in the TSF during operations unless the effluent permit could be amended to include the discharge of tailings supernatant (KP 2005a, pg. 12). MPMC was actively transferring water from the TSF to the Cariboo Pit near the end of KP's tenure as design engineers to remove some of the surplus pond water (KP 2011c).

Seismic Design Criteria

Probabilistic and deterministic seismic hazard assessments were carried out to estimate the magnitude of the design earthquake. The 1 in 475 year event was chosen as the design basis earthquake (DBE) based on CDSA guidelines (KP 1995b, pg. 9). The maximum design earthquake (MDE) was chosen for post-closure conditions and was equal to 50% of the maximum credible earthquake (MCE). KP stated that "due to the dense nature of the overconsolidated foundation soils at the project site, the amplification of seismic waves as they propagate from bedrock to the ground surface will not be significant" and that therefore "maximum bedrock ground motion parameters have been used for design" (KP 1995b, pg. 10).

In 2007, the design earthquake ground motions were updated to reflect the 2005 National Building Code Seismic Hazard Calculation by Natural Resources Canada (KP 2007b, Table 3.1). The design return period for the OBE was left unchanged but the magnitude of the ground motions was increased. For the MDE, the return period was reduced and the magnitude of the ground motions was increased.

Embankment Design

This section is a general summary of the as-built embankment design for the PE while KP was EOR. A discussion of the differences between the PE design and the ME and SE designs is beyond the scope of this report.

The PE is a zoned structure consisting of an upstream compacted till core (Zone S) and downstream rockfill shell (Zone C) separated by fine filter (Zone F) and transition (Zone T) zones. During the years that KP were the design engineers, the embankment was raised using a "modified centerline" raise geometry which resulted in the till core being inclined slightly upstream. A notable exception to the general design was the substitution of Zone C with cycloned sand during a cycloned sand placement trial in Stage 2C. The sand was placed between approximately Sta. 3+900 and 4+000 (KP 2001, Dwg. 11162-13-120).

Support for the till core is provided by an upstream zone (Zone U) that is partially constructed overtop of the tailings beach. The design and nomenclature used for the upstream support zone changed throughout operations. In Stages 1 and 2, it was constructed of compacted till (Zone B) underlain by a coarse rockfill bearing layer (Zone CBL). In Stage 3, cycloned sand was used (Zone CS). From Stage 4 onwards, the zone was constructed primarily of hydraulically placed cell sand which was often substituted by rockfill when sand cells could not be constructed. Zone CBL was placed below Zone U for the Stage 4 raise only. Subsequent to Stage 4 Zone CBL was no longer used.

The PE design contains features aimed at reducing the elevation of the phreatic surface within the embankment fill and foundation soils and aid in consolidation of the tailings beach. There are:

- 3 Outlet Drains (OD-4, OD-5 and OD-6) which convey water from downstream of the Zone S, through the Drain Monitoring Sump to the PE SCP. The drains were installed during Stage 2A construction. The drains consist of 150 mm diameter perforated corrugated plastic tubing conveyance pipes surrounded by drain gravel, Zone F material and filter fabric. (KP 1999b, pg. 19; Dwg. 1062-9-133).
- An Upstream Toe Drain constructed in Stage 5 within Zone U between Sta. 45+75 and 30+00 at an elevation of 946.3 m. The drain consists of a 250 mm diameter perforated pipe. It exits the embankment at Sta. 45+75 through a concrete encased steel pipe in the foundation. Water that collects in the Upstream Toe Drain reports to the PE SCP (KP 2008a, pg. 10; Dwg. 220).

The design for the PE specified an overall downstream slope of 2H:1V. As-built drawings show that 2H:1V downstream slopes were achieved in all construction stages up to and including Stage 4. In Stage 5, the design was modified to allow localized steepening of the upper portion of the slope to 1.4H:1V “to allow the embankments to be raised using the modified centerline method in the timeline required to maintain the storage and freeboard requirements of the TSF”. The steepened slope was intended to be an interim slope, to be expanded to 2H:1V once the Stage 5 elevation was reached. (KP 2006, pg. 9). As-built drawings issued for Stage 5 construction (KP 2008a) indicate 1.43:1V downstream slope for the PE. Review of the as-built construction record drawings for Stages 6a and 6b shows that the slope was flattened to approximately overall 1.8H:1V with a break point between a toe fill and a 1.7H:1V slope at approximately El. 938 m (KP 2009c, Dwg. 225 and KP 2011a, Dwg. 225). It is noted by KCB that the stability of the PE for the Stage 6 design report was analyzed assuming a downstream slope of 2H:1V (KP 2014).

Core Zone S –Width and Gradation

Zone S width within the PE was specified as 8 m from Stages 1 through 5, and was reduced to 5 m in Stage 6. The rationale for the reduction in core width was the positive effect of the upstream toe drain in reducing the hydraulic gradient across the core (KP 2007b).

Zone S gradation criteria remained unchanged throughout Stages 1 through 6 and are shown on Figure 3.10, with typical gradations of samples collected during construction.

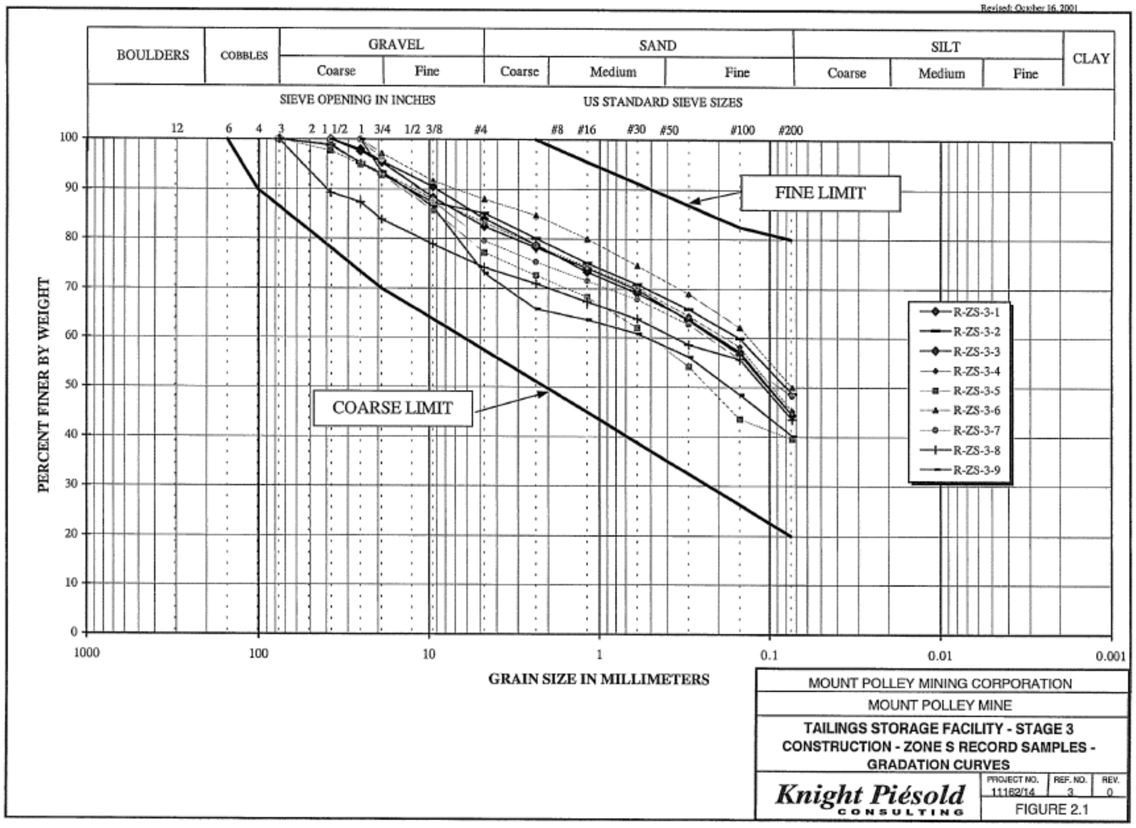


Figure 3.10 KP Zone S (Core) Specifications with Gradation Record Results (Stage 3) (KP 2001)

Filter Criteria

The gradation specification for the filter zone was modified three times during KP's tenure as design engineer. Different filter design was used for Stages 1, 2 and 3 through 6. Gradation envelopes are shown on Figure 3.11 along with typical gradation curves from samples of Zone F collected during construction. It is noted that placed material was often coarse of the design envelope.

An internal review of the filter criteria was undertaken by KP (2005c) which compared the design envelope to commonly used design guidelines. The analysis concluded that the filter design did not meet the filter criteria as it was too broadly graded and allowed a too high percentage of fine material. Broadly graded filters may be susceptible to internal instability and filters with high fines content may develop and sustain open cracks. Both of these issues can lead to loss of core material through the affected filter. It was also noted, however, that the reduction of the hydraulic gradient across the dam core considerably reduces risk of core erosion even with filters that do not meet recommended criteria. Recommendations from the study included: maintaining a low phreatic surface upstream of the core through toe drains, monitoring the phreatic surface within the embankments to ensure gradients are low, modifying the filter gradation specifications in areas where hydraulic gradients may be high and adopting other measures to reduce hydraulic gradients across the core. A comparison of filter specifications before and after this memorandum was issued indicates that the filter specifications were not changed.

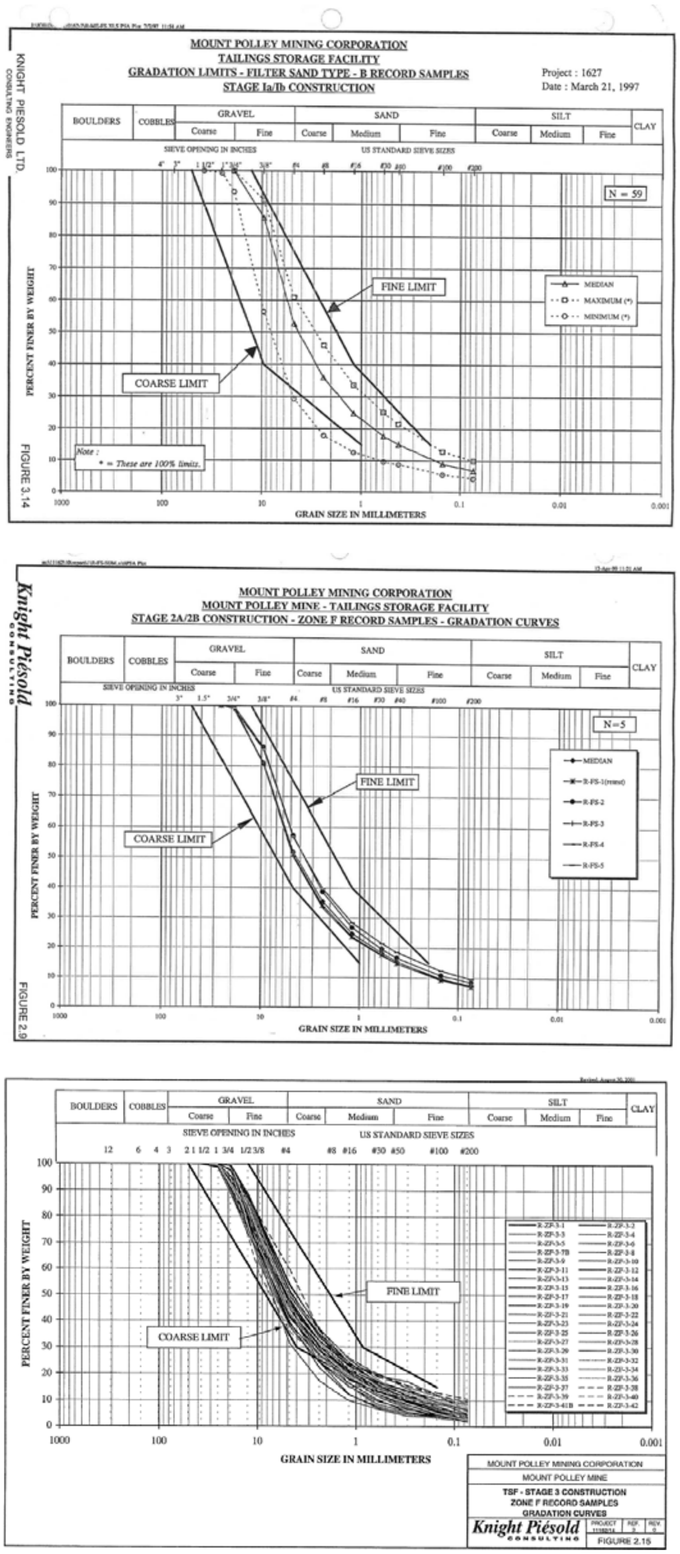


Figure 3.11 **KP Filter Zone Specifications (for Stage 1 (top) (KP 1997c), Stage 2 (middle) (KP 1999b) and Stage 3 (bottom) (KP 2001) with Gradation Record Results**

3.5.3 AMEC

The AMEC annual construction monitoring manuals essentially served as design reports for the Stage 7, Stage 8 and Stage 9 raises. Issued for construction drawings were typically issued with the monitoring manuals, along with stability analyses. Based on review of the documentation, it appears that AMEC generally accepted the design criteria adopted by KP. AMEC did not summarize their design criteria in a table or design basis memorandum.

Consequence Rating

AMEC adopted the same "significant" hazard consequence rating for the Stage 7, 8 and 9 TSF embankments as was assumed by KP previously(AMEC 2012b, pg. 1; BGC 2013, pg. i; AMEC 2014a, pg. 1).

Hydrologic Design Criteria

The 72-hour PMP event was maintained as the design flood by AMEC. They specified a minimum operating freeboard of 1.3 m below the dam crest to store the design flood and to provide for wave run-up (AMEC 2012b, pg. 8; AMEC 2014a, pg. 10). It is noted that in the 2012 dam safety inspection report, the design flood event is stated to be the 24-hour PMP, although the minimum freeboard requirement of 1.3 m is unchanged. It is unclear whether this is a mistake made in the report, or whether a change to the design criteria was made in 2012, and was then reverted back to the 72-hour PMP in 2013 (BGC 2013, pg. 7).

Seismic Design Criteria

AMEC does not explicitly state their seismic design criteria. It is noted though that stability analysis under seismic loading (pseudo-static conditions) was not analyzed. In the stability analysis included in the Stage 8 monitoring manual, AMEC states that "due to the negligible reduction in FoS under static loading conditions, it is reasonable to infer that the seismic stability situation would remain essentially unchanged relative to KP's 2007 analyses, which predicted earthquake-induced deformations, under the design earthquake loading, to be well within tolerable limits." (AMEC 2012c, Appendix A, pg. 4). From this statement it is assumed that AMEC adopted KP's design seismic criteria, at least for the Stage 8 design.

Embankment Design

For the Stage 7 raise, AMEC adopted the same general design for the PE as KP, with an upstream raised core and Zone U constructed variably with sand cells, waste rock or a combination of both (AMEC 2012b, pg. 10; AMEC 2013a, pg. 11). The design was revised during Stage 8, when the raising method changed from modified centerline to centerline above El. 963.5 m (AMEC 2012d).

As-built drawings for Stages 7 through 9 show that the downstream slope of the PE was constructed at approximately 1.3H:1V (AMEC 2012b, fig. 2011AB.05; AMEC 2013a, Dwg. 2012AB.04; AMEC 2014a, Dwg. 2013AB.04) despite a design slope of 2H:1V being specified in the Stage 7 and Stage 8 construction monitoring manuals. No explicit mention is made to the oversteepened slopes in the 2011 construction record report. In the 2012 as-built report, the steepened slopes are addressed but are accepted as temporary since the final dam slope will be flattened. Beginning in Stage 9, a 1.3H:1V downstream slope is specified on the issued for construction drawings, and no mention is made to achieving an overall design slope of 2H:1V (AMEC 2013, Dwg. 2013.05.01).

Core Width and Filter Criteria

AMEC specifications for the filter zone are shown on Figure 3.12. The gradation envelope is the same as was adopted by KP for Stages 3 through 6. As noted in the figure, the filter zone gradation was often coarser than specified, based on 2013 construction records.

The Zone S width specified in the AMEC design reports remained unchanged from KP's Stage 6 design at 5 m.

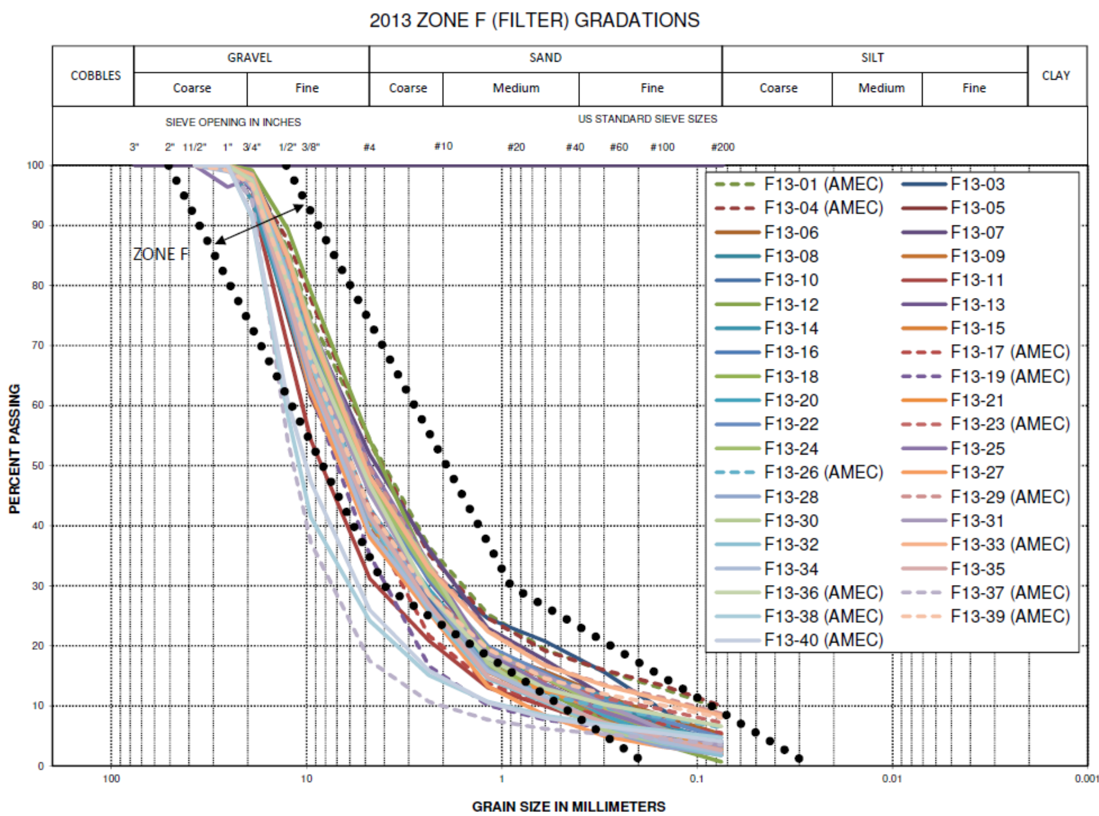


Figure 3.12 AMEC Filter Specifications and QC Data – 2013 (AMEC 2014a)

3.5.4 BGC

BGC's Stage 10 design report was issued in 2014. The scope of the report was for embankment construction above 970 m, which had not yet been reached when the PE was breached. Therefore, the following sections only summarize information within the design report and are not representative of as-built conditions.

Consequence Classification

BGC does not explicitly state that a change in the CDA consequence classification from “significant” to “high” is to be made for Stage 10, but they do say that the seismic and design flood criteria are consistent with the “high” classification (BGC 2014).

Hydrologic Design Criteria

The 10-day duration PMF was chosen as the appropriate inflow design flood (IDF). The longer duration flood event was deemed most critical to the TSF since the facility does not have an operations spillway and any excess flood water must be pumped down by the reclaim system. Freeboard above the flood level was set at 1.0 m (BGC 2014, pg. 11-12).

BGC notes that that “in recent years, the TSF has been operating with a significant annual water balance surplus, with the result that the volume of water stored within the TSF has increased on a year over year basis” and that in May, 2014 the pond volume was estimated to be between 8 Mm^3 and 9 Mm^3, significantly more than is required to maintain a viable water reclaim pond. (BGC 2014, pg. 22). The water surplus was taken into account in determining the 2014 crest elevation, along with MPMC’s plan for constructing and operating a water treatment plant capable of treating and discharging 3 Mm^3 per year, year round (BGC 2014, pg. 23). The following three scenarios were reviewed (BGC 2014, pg. 24):

1. Average-year hydrologic conditions with no water treatment and discharge. Required 2014 crest elevation 972.5 m.
2. Wet year (1 in 200 year) hydrologic conditions with water treatment and discharge beginning in January 2015, three months later than expected by MPMC. Required 2014 crest elevation 972.5 m.
3. Wet year (1 in 200 year) hydrologic conditions with water treatment and discharge beginning in July, 2015, nine months later than expected by MPMC. Required 2014 crest elevation 973.5 m.

The second scenario was chosen as the design basis for the Stage 10 crest elevation.

Seismic Design Criteria

More conservative seismic design criteria were adopted by BGC. The revised design earthquake magnitudes and ground motions were calculated using the NRCAN seismic hazard calculator (BGC 2014, pg. 14). The 1 in 5,000 year earthquake design ground motions are consistent with CDA guidelines for the “very high” consequence classification.

Embankment Design

The results of the Stage 10 stability analysis indicated that construction of a 30 m wide (min.) rockfill buttress was required along the toe of the embankment from Sta. 4+400 to the tie-in with the ME at Corner 2. A minimum buttress elevation of 940 m was specified between Sta. 4+400 and 3+300, rising from 940 m to 945 m between 3+300 and 3+100 and 945 m between Sta. 3+100 and Corner 2. It was understood that the crest raise to El. 972.5 m would precede the completion of the toe buttress construction, but that the buttress would be in place before raising the dam above El. 972.5 m (BGC 2014, pg. 50-51).

BGC specified the same fill zone dimensions as AMEC and downstream design slope of 1.3H:1V (BGC 2014, Dwg. MPMC-XD-03-02).

Core Width and Filter Criteria

BGC specifications for the filter zone are shown on Figure 3.13. BGC identified that previous filter criteria used did not meet the specification for control of segregation and, accordingly, increased the minimum sand size fraction from 30% to 45% (BGC 2014, pg. 60).

The same Zone S width and material specifications as adopted by AMEC were used by BGC.

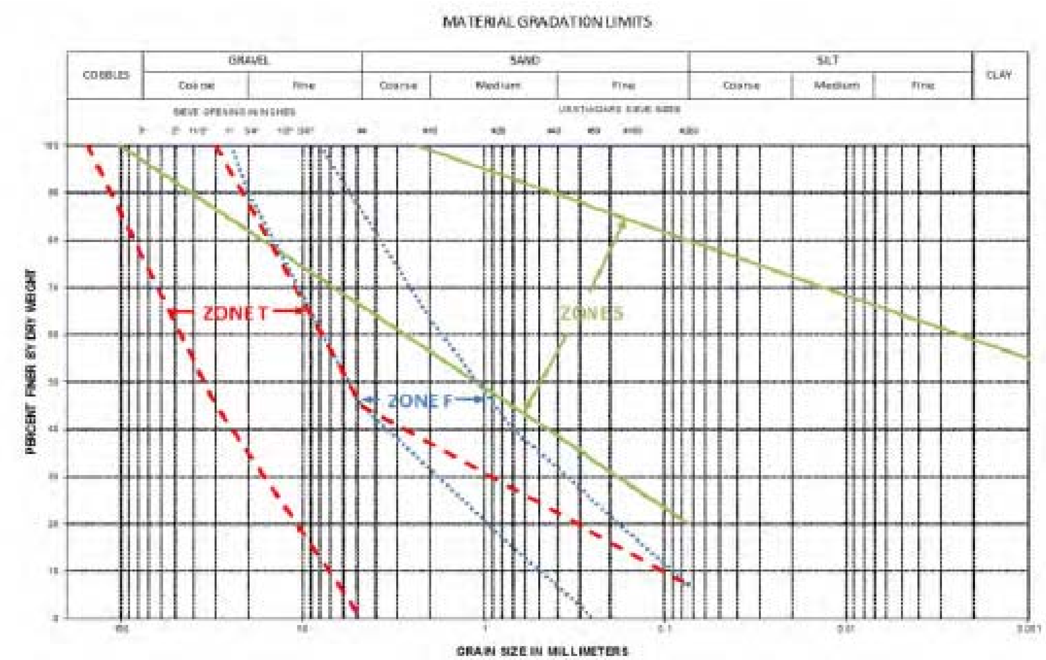


Figure 3.13 BGC Filter and Core Zone Specifications (BGC 2014)

Table 3.5 Summary of TSF Design Criteria from Key Reports

Criteria	1995 Design Report (KP 1995b, 1995c)	1997 Updated Design Report (KP 1997b)	Design of TSF to Ultimate Elevation (KP 2005a)	Stage 6 Design of TSF (KP 2007b)	TSF Report on 2008 Annual Inspection (KP 2009a)	Stage 7 2011 Construction Monitoring Manual (AMEC 2011)	2011 Construction As-Built and Annual Review Report (AMEC 2012b)	Stage 8 2012 Construction Monitoring Manual (AMEC 2012c)	Stage 8A TSF Raise (AMEC 2012d)	Tailings Storage Facility 2012 Annual Review (BGC 2013)	Stage 9 2013 Construction Monitoring Manual (AMEC 2013b)	2013 As-Built and Annual Review Report (AMEC 2014a)	Stage 10 Raise Design Report (BGC 2014)
Design Flood Event	24-hour PMP with ability to store 10-day PMP with reduced freeboard	24-hour PMP with ability to store 10-day PMP with reduced freeboard	24-hour PMP	72-hour PMP	-		72-hour PMP	-	-	24-hour PMP	-	72-hour PMP	10-day PMF
Raise Methodology	Modified Centerline	Modified Centerline	Modified Centerline	Modified Centerline	-	Modified Centerline	-	Modified Centerline	Centerline above 963.5 m	-	Centerline	-	Centerline
Permitted Elevation		934.0 m	951.0 m	958.0 m		960.5 m	960.5 m	963.5 m	965.0 m	965.0 m	970.0 m	970.0 m	972.5 m (submitted but not approved)
Ultimate Crest Elevation	960 m	965 m	965 m	970 m	-	Not Defined	-	Not Defined	Not Defined	-	Not Defined	-	Unknown – mine plan out to 2025 with flat beaches at 990 m
Operating Freeboard	24-hour PMP + 1.0 m Wave Run-up	24-hour PMP + 1.0 m Wave Run-up	24-hour PMP + 1.0 m Wave Run-up	72-hour PMP + 0.7 m Wave Run-up	-	-	1.3 m for storage of the PMP and wave run-up	-	-	1.3 m for storage of the PMP and Wave Run-Up	-	1.3 m for storage of the PMP and Wave Run-Up	10-day PMP +1.0 m Wave Run-up and Wind Set-up
Reclaim Pond Volume	2.0 Mm^3	2.5 Mm^3	-	-	-	-	-	-	-	-	-	-	Based on the actual pond volume in 2014.
Total Tailings Capacity	68.6 Mt	84.5 Mt	85.0 Mt	100.0 Mt	-	-	-	-	-	-	-	-	Unknown
Tailings Discharge Rate	13,425 tpd @ 35% solids	17,808 tpd at 35% solids	20,000 tpd at 35% solids	20,000 tpd at 35% solids	-	-	-	-	-	-	-	-	21,981 tpd @ 35% solids
Avg. Tailings Density for Staging	1.28 t/m^3	1.28 t/m^3	1.36 t/m^3	1.40 t/m^3	-	-	-	-			-		1.40 t/m^3
Tailings SG	2.78	2.78	2.70	2.65	-	-	-	-			-		-
Target FoS During Operations (Static)	1.3	1.3	1.3	1.3	-	1.3	-	1.3	1.3	-	1.3	-	1.5 1.1 (residual GLU strength – not design basis criteria)
Design Basis Earthquake	1 in 475 yr (0.037g)	1 in 475 yr (0.037g)	1 in 475 yr (0.037g)	1 in 475 yr (0.070g)	-	Stated to be comparable to KP Stage 6	-	Not stated. Post earthquake FOS >1.1	-	-	Not stated. Post earthquake FOS >1.1	-	1 in 5,000 yr (operations) (0.18g M=5.9)
Maximum Design Earthquake	1 in 2500 yr (0.065g) (50% of MCE) (Closure)	1 in 2500 yr (0.065g) (50% of MCE) (Closure)	50% of 1 in 2500 yr (0.065 g)	1 in 1000 yr (0.096g)	-	-	-	-	-	-	-	-	1 in 10,000 yr (closure) (0.24g M=6.2)
Consequence Classification (Operations)	LOW (CDSA)	LOW (CDSA)	HIGH (BCDSR)	LOW (CDA)	SIGNIFICANT (CDA)	-	SIGNIFICANT (CDA)	-	-	SIGNIFICANT (CDA)	-	SIGNIFICANT (CDA)	HIGH (CDA)
Consequence Classification (Closure)	HIGH (CDA)	HIGH (CDA)	HIGH (BCDSR)	LOW (CDA)	SIGNIFICANT (CDA)	-	-	-	-	-	-		HIGH (CDA)

Notes: 1. If the criteria is not stated in the Design Report one might assume that previous criteria was accepted and/or adopted.
CDSA – Canadian Dam Safety Association
CDA – Canadian Dam Association
BCDSR – BC Dam Safety Regulation

3.6 Geotechnical Parameters

3.6.1 General

This section presents KCB's review of the geotechnical parameters used in evaluating the stability of the PE. The GLU was identified as a unit of concern during most of the design stages and compared with the other soil units there is a relatively large amount of discussion in the record documents on its design strength. The following section discusses the evolution of the glaciolacustrine design strength. Material properties assumed for all soil types are summarized in Table 3.6 through Table 3.8.

3.6.2 Knight Piésold

For the initial design reports the glaciolacustrine unit's effective friction angle was assumed to be 33° which was based on the results of a consolidated undrained triaxial test on a sample of glaciolacustrine fine sand and silt collected from test pit TP95-39 near the southern end of the ME. The sample was remolded and compacted to 95% Standard Proctor density for testing (KP 1995a, pg. 11-12).

One of the recommendations from the 2006 DSR was to review the design criteria for the GLU's shear strength. In particular, the reviewer felt that the existence of pre-sheared planes in the foundation had not been adequately investigated. The review suggested that glaciolacustrine core from KP's 2006 site investigation program below the ME be thoroughly examined for pre-sheared surfaces. It was recommended that even if pre-sheared surfaces were not discovered, samples should be tested by direct shear, oriented along a plane of weakness, to assess whether the soil displayed a brittle response to shearing (AMEC 2006).

KP responded to this recommendation in the Stage 6 design report by adopting an effective residual friction angle of 24° for the glaciolacustrine unit below the ME. This angle was determined based on an empirical correlation between the drained residual strength, clay content, liquid limit and effective normal stress. In their assessment KP, assumed a clay content ranging from 25% to 50%, a liquid limit of 40%, and an effective normal stress of 700 kPa (KP 2007b, Appendix, pg. A3)

Subsequently, KP collected samples of "lacustrine material" below the ME for direct shear testing (refer to Section 3.3.2, 2007 investigation). The results of the direct shear testing were not available in time for the Stage 6 design report (KP 2007b, Appendix A, pg. A2 and A3). In a letter to MEM, the results of the testing were summarized as ranging from "21 to 25 degrees, with an average of 23 degrees" (KP 2007c, pg. 2). It is noted that the revised glaciolacustrine friction angle of 24° was only used for foundation soils beneath the ME for the Stage 6 design. The original friction angle of 33° was used below the PE.

Further stability analysis was carried out subsequent to the Stage 6 Design Report to assess the requirement for a toe buttress at the ME. In the first round of analysis it was noted by KP that the 24° friction angle was confirmed by direct shear testing and that strength was used again (KP 2008b). It was not stated by KP whether the 24° corresponded to a peak or residual strength.

The second round of analysis was performed in response to movements that were detected in a glaciolacustrine layer below the ME in slope inclinometer SI01-02. The movements were back analyzed using limit equilibrium analysis. It was determined that the mobilized friction angle along the plane of movement was between 16° and 17°. The design of the ME toe buttress was revised to achieve the design FoS with the back-analyzed foundation strength (KP 2009b).

Table 3.6 Summary of KP Assumed Material Properties

Materials	TSF Design Report (KP 1995b, 1995c)		TSF Updated Design Report (KP 1997b)		Design of TSF To Ultimate Elevation (KP 2005a)		Stage 6 Design Report (KP 2007b)		Letter – Buttress Requirement for Main Embankment (KP 2008b)		Letter – Buttress Requirement for Main Embankment (KP 2009b)	
	ϕ'	c'	ϕ'	c'	ϕ'	S_u/σ'_v	ϕ'	S_u/σ'_v	ϕ'	S_u/σ'_v	ϕ'	S_u/σ'_v
Tailings												
Partially Consolidated	-	10-55	-	10-55	-	-	-	-	-	-	-	-
Fully Consolidated	30	0	30	0	-	-	-	-	-	-	-	-
Residual Post-Liquefaction	-	-	-	10	-	0.1	-	?[1]	-	?[2]	-	?[2]
Coarse Tailings	-	-	-	-	30	-	30	-	30	-	30	-
Tailings < El. 934 m([3])	-	-	-	-	30	-	30	-	30	-	30	-
Tailings > El. 934 m([3])	-	-	-	-	-	0.3	?[4]	-	?[4]	-	?[4]	-
Fill Materials												
Zone A	35	0	35	0	-	-	-	-	-	-	-	-
Zone B	35	0	35	0	35	-	-	-	-	-	-	-
Zone C	35	0	35	0	40	-	40	-	40	-	shear/ normal[6]	-
Zone CS	32	0	32	0	-	-	-	-	-	-	-	-
Zone S	35	0	35	0	35	-	35	-	35	-	35	-
Drain	-	-	35	0	35	-	35	-	35	-	35	-
Clay Liner	-	-	-	-	40	-	?	?	-	-	-	-
Foundation Soils												
Ablation Till	-	-	-	-	-	-	-	-	-	-	-	-
Loose to Medium Dense Till (Short Term)	-	-	-	85	-	-	-	-	-	-	-	-
Loose to Medium Dense Till (Long Term)			26	0	26	-	26	-	26	-	26	-
Dense to Very Dense Till	-	-	26	0	26	-	26	-	26	-	26	-
Glaciolacustrine (Peak)	-	-	-	-	-	-	33	-	-	-	-	-
Glaciolacustrine (Residual)	-	-	-	-	-	-	24	-	24	-	16-17	-
Glaciolacustrine (Undrained)	-	-	-	-	-	-	-	0.25[5]	-	-	-	-
Glaciofluvial Sediments	-	-	-	-	33	-	-	-				
Glaciolacustrine Sediments	-	-	-	-	33	-	-	-	-	-	-	-
Glaciolacustrine/Glaciofluvial Sediments (Peak)	-	-	33	0	-	-	-	-	-	-	-	-

Materials	TSF Design Report (KP 1995b, 1995c)		TSF Updated Design Report (KP 1997b)		Design of TSF To Ultimate Elevation (KP 2005a)		Stage 6 Design Report (KP 2007b)		Letter – Buttress Requirement for Main Embankment (KP 2008b)		Letter – Buttress Requirement for Main Embankment (KP 2009b)	
Glaciolacustrine/Glaciofluvial Sediments (Residual)	-	-	-	-	-	-	-	-	-	-	-	-
Dense Till	-	-	-	-	-	-	-	-	-	-	-	-
Basal Till	-	-	-	-	33	-	33	-	-	-	-	-
Foundation Soils	33	0	-	-	-	-	-	-	-	-	-	-

Notes:

Used for the Main Embankment analyses only.

Used for the Perimeter Embankment analyses only.

Used for both the Main Embankment and Perimeter Embankment Analyses.

1. The design report states that a post-liquefaction case was analyzed where tailings were assigned a liquefied strength. The post-liquefaction analysis was not provided to KCB in the 2014 Slope/W model submission.
2. The letter states that the same analyses were performed as for the Stage 6 Design Report. The post-liquefaction analysis was not provided to KCB in the 2014 Slope/W model submission.
3. There are inconsistencies between the tailings elevations recorded in the Slope/W output material property tables and what was actually assumed in the analysis.
4. The friction angle for the tailings was omitted from the material property summary table included with the Slope/W analysis.
5. This strength was used for sensitivity analysis not presented in the Stage 6 Design Report. The strength value and stability analysis results were presented to MEM in a letter in support of the Stage 6 design (KP 2007c).
6. Shear strength/normal function not specified.

3.6.3 AMEC

AMEC adopted KP's Stage 6 Design shear strength assumptions for the GLU for the Stage 7 design. In the subsequent 2011 site investigation report, AMEC summarized the GLU's shear strength assumptions as follows:

> *In summary, the 2011 investigations, previous investigations, and the inclinometer records indicate that the GLU is over-consolidated, not extensively pre-sheared if it is at all, has not exhibited displacements of any concern in response to the loading imposed by the dam construction. For stability evaluation purposes, the GLU can be characterized and perhaps conservatively so, with drained strength parameters of c' = 0 and φ' = 28^{o}. Continued regular monitoring of inclinometer displacements is important going forward to validate this conclusion and re-evaluate it should increased rates of displacement be noted.* (AMEC 2012a, pg. 26).

The design friction angle of 28^{o} was carried forward for the Stage 8 and Stage 9 designs, although sensitivity analysis was also performed at the ME in Stage 8 for a range of glaciolacustrine strengths (AMEC 2012c, Appendix A, pg. 2). Beginning in Stage 8, the same shear strength parameters were applied to the GLU below both the ME and PE.

Table 3.7 Summary of AMEC Assumed Material Properties

Materials	Stage 7 2011 Construction Monitoring Manual (AMEC 2011)		Stage 8 2012 Construction Monitoring Manual (AMEC 2012c)		Stage 8A Drawings and Stability Analysis (AMEC 2012d)		Stage 9 2013 Construction Monitoring Manual (AMEC 2013b)	
	ϕ'	S_u/σ'_v	ϕ'	S_u/σ'_v	ϕ'	S_u/σ'_v	ϕ'	S_u/σ'_v
Tailings								
Partially Consolidated	-	-	-	-	-	-	-	-
Fully Consolidated	30	-	30	-	30	-	30	-
Residual Post-Liquefaction	-	0.1	-	0.1	-	0.1	-	0.1
Coarse Tailings	-	-	-	-	-	-	-	-
Tailings < El. 934 m(3)	-	-	-	-	-	-	-	-
Tailings > El. 934 m(3)	-	-	-	-	-	-	-	-
Fill Materials								
Zone A	-	-	-	-	-	-	-	-
Zone B	-	-	-	-	-	-	-	-
Zone C	shear/ normal[1]	-	shear/ normal[1]	-	shear/ normal[1]	-	shear/ normal[1]	-
Zone CS	-	-	-	-	-	-	-	-
Zone S	35	-	35	-	35	-	35	-
Drain	-	-	-	-	-	-	-	-
Clay Liner	-	-	-	-	-	-	-	-
Foundation Soils								
Ablation Till	26	-	-	-	-	-	-	-
Loose to Medium Dense Till (Short-Term)	-	-	-	-	-	-	-	-
Loose to Medium Dense Till (Long-Term)	-	-	-	-	-	-	-	-
Dense to Very Dense Till	-	-	-	-	-	-	-	-
Glaciolacustrine (Peak)	-	-	-	-	-	-	-	-
Glaciolacustrine (Residual)	-	-	-	-	-	-	-	-
Glaciolacustrine Sediments	-	-	-	-	-	-	-	-
Glaciofluvial Sediments	-	-	-	-	-	-	-	-
Glaciolacustrine/Glaciofluvial Sediments (Peak)	33	-	28	-	28	-	28	-
Glaciolacustrine/Glaciofluvial Sediments (Residual)	24	-	-	-	-	-	-	-
Dense Till	-	-	-	-	-	-	-	-
Basal Till	33	-	33	-	33	-	33	-
Foundation Soils	-	-	-	-	-	-	-	-

Notes:
Used for the Main Embankment analyses only.
Used for the Perimeter Embankment analyses only.
Used for both the Main Embankment and Perimeter Embankment Analyses
1. Shear strength/normal function from Leps (1970).
2. Note: only values of parameters used in the analyses are noted

3.6.4 BGC

BGC adopted the same peak glaciolacustrine shear strength as used by AMEC for the Stage 8 and 9 designs. However, to account for potential pre-shearing along clayey varves and/or the formation of sufficient foundation strain that could lead to strength loss, they also assigned a residual shear strength to the glaciolacustrine unit. The residual friction angle was approximated based on the Stark and Eid (1994) relationship using index properties determined during the AMEC 2011 site investigation (BGC 2014, pg. 44-48).

Table 3.8 Summary of BGC Assumed Material Properties

Materials	Tailings Storage Facility Stage 10 Raise Design Report (BGC 2014)	
	ϕ'	S_u/σ'_v
Tailings		
Partially Consolidated	-	-
Fully Consolidated	-	-
Residual Post-Liquefaction	-	0.1
Coarse Tailings	-	-
Tailings < El. 934 m(3)	-	-
Tailings > El. 934 m(3)	-	-
Fill Materials		
Zone A	-	-
Zone B	-	-
Zone C	shear/ normal[1]	-
Zone CS	-	-
Zone S	35	-
Drain	-	-
Clay Liner	-	-
Foundation Soils		
Ablation Till	-	-
Loose to Medium Dense Till	-	-
Loose to Medium Dense Till (Short-Term)	-	-
Loose to Medium Dense Till (Long-Term)	-	-
Glaciolacustrine (Peak)	28	-
Glaciolacustrine (Residual)	28 (inclined)[2] 18 (horizontal)	-
Glaciolacustrine Sediments	-	-
Glaciofluvial Sediments	-	-
Glaciolacustrine/Glaciofluvial Sediments (Peak)	-	-
Glaciolacustrine/Glaciofluvial Sediments (Residual)	-	-
Dense Till	-	-
Basal Till	33	-
Foundation Soils	-	-

Notes:
Used for both the Main Embankment and Perimeter Embankment Analyses
1. Shear strength/normal function from Leps (1970).
2. The "inclined" friction angle was applied to portions of the failure surface in the glaciolacustrine unit that were not horizontal.

3.7 Stability Analyses – Perimeter Embankment

3.7.1 General

Stability analyses of the PE were carried out by KP, AMEC and BGC, for various embankment elevations and with some variations in the assumed foundation geometry, phreatic surfaces, and foundation soils. The following sections summarize the latest results from the respective consultants, which include:

- KP Stage 6, Dam crest elevation 958 m (KP 2007b)
- AMEC Stage 9, dam crest elevation 970 m (AMEC 2013b) with an additional undrained analysis carried out for Stage 7.
- BGC Stage 10, dam crest elevation 972.5 m (BGC 2014).

Design Section D, located at Sta. 3+990 was chosen by KP, AMEC and BGC as a critical section for embankment stability. It was the only PE section analyzed by KP and AMEC and one of two sections analyzed by BGC, the other being at 3+550 near the PE Till Borrow Pit. BGC found Section D to be the more critical of the two in terms of stability.

A summary of the factors of safety, design downstream slopes and as-built downstream slopes for the PE are presented in Table 3.9.

Table 3.9 Summary of PE Slopes and Stability Analyses Results

Stage	Elevation	Design Slope	As-Built Slope	FoS - Static	FoS - Seismic	Reference
6	958.0	2H:1V		1.7	$A_{crit.}$ = 0.25	KP 2007c
	958.0		1.8H:1V	2.1	-	AMEC 2011
7	960.0	2H:1V	1.8H:1V	2.0	-	AMEC 2011
8	965.0	2H:1V (ultimate slope)		1.81	1.77[1]	AMEC 2012d
	965.0		1.3H:1V			AMEC 2013a
9	970.0	1.3H:1V	1.3H:1V	1.63	1.58[1]	AMEC 2014a
10	972.5	1.3H:1V, with buttress		1.5	0.9[2]	BGC 2014

1. Seismic case is for post-earthquake stability assuming residual undrained strength in the tailings.
2. Pseudo-static analysis – BGC note that deformation analysis will be carried out in the next stage of design to support the lower FoS.

Key assumptions for PE stability section and results from the stability analyses are summarized in the following sections.

3.7.2 Knight Piésold

3.7.2.1 Design Analysis

Stability analysis output files for the Stage 6 Design were not included in the Design Report, however they were provided subsequently to MEM as part of their inquiry (KP 2014). The following summarizes key assumptions used for the analysis:

- Foundation stratigraphy is comprised of basal till over glaciolacustrine sediments. Elevations are not shown on the model, but based on scaling from the Stage 6 crest elevation (958 m) to the ground surface (~930 m) the elevation of the top of the GLU is approximately 915 m.
- An overall downstream slope of 2H:1V.
- The phreatic surface is drawn down through Zone C (on the downstream side of Zone S) and is at the ground surface below Zone C and at the downstream toe.

A static FoS of 1.7 was calculated for the PE for this analysis, above the design FoS of 1.3.

3.7.2.2 Undrained Analysis

KP references an analysis in a letter to MEM in which an undrained strength was assigned to the GLU in support of the Stage 6 design for the ME. It is not clear whether this analysis was performed for the PE. KP describes the analysis as follows:

> *"Stability analyses were also completed using conservative residual undrained shear strength, (S_u/p') values to calculate the factor of safety for undrained conditions in the lacustrine unit under large strain conditions. The analyses were completed using typical tau/sigma values for soft clayey materials in the order of 0.25 to 0.3. The factor of safety for the Stage 6 configuration was approximately 1.1 for a tau/sigma value of 0.25, indicating that there is also sufficient undrained strength in the lacustrine unit for the embankment to remain stable."* (KP 2007c).

This is one of only two instances in the record of an undrained strength being applied to the GLU, the other being the AMEC analysis discussed in the next section.

3.7.3 AMEC

3.7.3.1 Design Analysis

AMEC's design basis for all of their issued design reports assumed that the GLU would undergo drained loading and that drained strength would be mobilized within the unit. This assumption is consistent with both KP and BGC. This section summarizes the design case stability analyses performed by AMEC.

Model Set-Up

AMEC analyzed stability of the PE to a maximum elevation of 970 m. Key assumptions used for the downstream stability of the embankment are summarized below:

- An overall downstream slope of 1.3H:1V.
- Sub-horizontal ground surface with El. 930 m below the embankment crest and approximately El. 928 m at the embankment toe.
- An existing berm (access road and catch berm) at the downstream toe.
- A phreatic surface elevation in the foundation soils based on VWP monitoring data (AMEC 2013b, pg. 3). On Section D, there were three functioning VWPs installed in the foundation when the Stage 9 stability was assessed. These were:
  - D1 – Tip El. 922.0 m, installed in glacial till in 1998 (KP 1999b, Dwg. 10162-9-152).
  - D6 – Tip El. 913.4 m, installed in glaciolacustrine sediments in drill hole VW11-10 (AMEC 2012a, pg. 8).
  - D7 – Tip El. 907.4 m, installed in glaciofluvial sediments in drill hole VW11-10 (AMEC 2012a, pg. 8).

  Historical monitoring data indicates that the elevation of the phreatic surface in D6 and D7 was approximately 917 m during the Stage 9 design. In D1 the phreatic surface was at approximately 927.5 m. Although not explicitly stated by AMEC, these elevations appear to be what were assumed in defining the location of the phreatic surface below the PE at Section D. The result is a phreatic surface that drops off to well below ground surface at the embankment toe.
- Foundation stratigraphy characterized by a basal till unit underlain by basal till interlayered with horizontal glaciolacustrine/glaciofluvial units. The top elevation of the top glaciolacustrine/glaciofluvial unit was assumed to be 915 m based on the drill hole VW11-10(DX4).

Material Parameters and Analysis

The material parameters used in the analysis are summarized in Table 3.7.

Static downstream stability was assessed for “drained” and “undrained” tailings strength. For the drained case, effective stress strength parameters were assumed. In the undrained case, an undrained strength relationship was used.

FoS is adequate for both the drained and undrained analysis scenarios, based on the design criteria summarized in Table 3.5. The FoS calculated by AMEC (AMEC 2013, MP00045_2013, App. A, pg. 4.) are summarized in Table 3.10.

Table 3.10 Summary of AMEC Calculated FoS – Section D

Stability Case	FoS[1]
Drained Tailings	1.63
Undrained Tailings	1.58

3.7.3.2 Undrained Analysis

AMEC provided MEM a copy of a sensitivity analysis in which the stability of the PE was assessed assuming undrained loading in the GLU (AMEC 2015), although it does not appear that this documentation was issued by AMEC to MPMC.

Model Set-Up

AMEC analyzed the stability of the PE in Stage 7 (crest El. 960.5 m) and at a crest elevation of 970 m (referred to as the “long term ultimate” embankment). Approximate downstream slopes for both of the analysis cross sections was 2H:1V. The phreatic surface for both analyses was assumed to draw down through the Zone C shell to the ground surface and transition to the elevation of the pond in the PE SCP at the downstream toe. The top of the GLU below the PE was assumed to be at El. 915 m.

Material Parameters and Analysis

For the undrained sensitivity analysis, the undrained strength ratio (S_u/σ'_v) was varied from 0.1 to 0.4. The effective slope for the analysis is approximately 2H:1V. FoS from the undrained analysis ranged from 0.95 to 1.68 for the Stage 7 case and 0.86 to 1.53 for the 970 m case. The results for the El. 970.0 m case are shown on Figure 3.14. Only the results are reported with no conclusions discussed or recommendations.

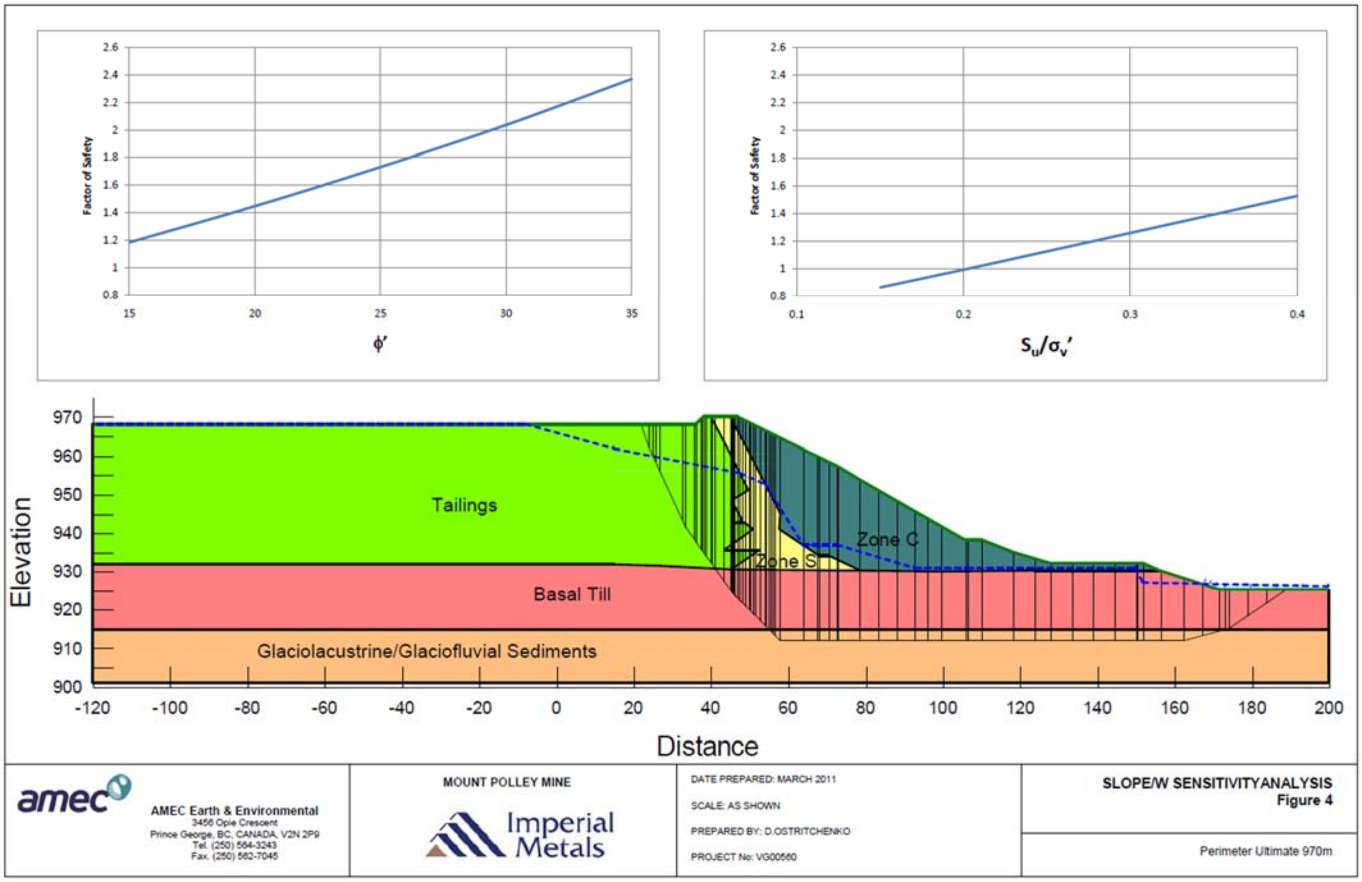


Figure 3.14 AMEC Undrained Strength Analysis – PE - El. 970.0 m

3.7.4 BGC

Model Set-Up

BGC analyzed stability of the PE at 970 m and 972.5 m, the design elevation of the Stage 10 raise. Key assumptions used for the downstream stability of the embankment are summarized below:

- An overall downstream slope of 1.3H:1V.
- Sub-horizontal ground surface below the embankment. The ground surface was approximately at El. 930 m below the embankment crest. Beyond the toe of the embankment the ground surface drops off to approximately El. 925 m to account for the PE SCP excavation.
- An existing buttress at the downstream toe.
- A phreatic surface at the fill/native ground contact below the embankment shell.
- Foundation stratigraphy characterized by a basal till unit underlain by basal till interlayered with horizontal glaciolacustrine units. The top elevation of the top glaciolacustrine unit was assumed to be 915 m. AMEC's drill logs from VW11-10 and SI11-04 were used to define the foundation stratigraphy (BGC 2014, pg. 40).

Material Parameters and Analysis

The material parameters used in the analysis are summarized in Table 3.8.

The three embankment geometries analyzed by BGC for downstream stability are listed below:

- crest elevation of 970 m without a buttress;
- crest elevation of 972.5 m without a buttress; and
- crest elevation of 972.5 m with a 30 m wide buttress raised to El. 941 m.

Each of the three cases above was analyzed for the peak and residual glaciolacustrine strengths. For all cases, the tailings were assigned an undrained strength.

BGC also analyzed a pseudo-static stability case assuming a pseudo-static acceleration of 0.09g, 50% of the bedrock peak ground acceleration. For this analysis, the friction angles assumed for Zone S and the foundation soils were reduced by 80% to account for cyclically induced pore pressures (BGC 2014, pg. 52).

FoS calculated for Section D by BGC are summarized in Table 3.11. It was determined that a buttress extension was required along the PE to achieve the minimum required FoS for the peak strength case. Details of the buttress requirement are provided in Section 3.5.4 of this report.

Table 3.11 Summary of BGC Calculated Static FoS for Downstream Stability – Section D

Embankment Geometry	Glaciolacustrine Strength Properties	FoS
Crest El. 970 m	Peak	1.47
	Residual	1.14
Crest El. 972.5 m; no buttress	Peak	1.40
	Residual	1.08
Crest El. 972.5 m; with buttress	Peak	1.55
	Residual	1.15

Hynes-Griffin and Franklin (1984) note that if a FoS greater than unity is achieved with a pseudo-static acceleration equal to 50% of the peak ground acceleration, displacements will be limited to less than 1 m. This criteria was not met in the BGC analysis, indicating that permanent displacements may exceed 1 m. BGC stated that “post-earthquake and seismic deformation analyses are the more appropriate means of evaluating the seismic stability of the dams, and such analyses will be undertaken in support of the next phase of the design” and “given the thin Zone S till core and filter/transition sequence, seismic deformation analyses may govern the ultimate design configurations for the dams” (BGC 2014, pg. 52).

4 KCB INTERPRETATION OF STATE OF KNOWLEDGE - PRIOR TO DAM BREACH

4.1 General

The purpose of this section is to present KCB's interpretation of the state of knowledge prior to the breach based on the site investigation and laboratory data that was available. The assessment includes the following:

- review of site investigation data (drilling, test pits, air photos, etc.) and assessment of Quaternary geology and development of a geologic model;
- review of laboratory and field test data and derivation of geotechnical parameters based on the available data;
- identification of design cases based on CDA Guidelines; and
- stability analyses, which includes: checks on previous consultant analyses; new analyses in the breach area; and, analysis using KCB geotechnical parameters.

The assessment is based on the dam design section that was constructed.

4.2 Summary of Pre-Breach Site Investigation Data

4.2.1 Site Investigation Overview

KCB used the general geological model discussed in Section 3.4 of this report to organize the data from the various investigations that were carried out within and around the TSF. KP assigned geologic descriptions to the soil units on their geologic cross sections (KP 1997b). In summarizing the index testing data, soils in the drill holes and test pits that appeared on the KP sections were classified as per the KP classification. Drill holes and test pits that were not included on the cross sections, or were drilled/excavated after the issue of KP's report, were classified by KCB based on the following:

- Proximity to the KP cross sections. Investigations that were close to the cross section locations, and had similar soil stratigraphy and soil descriptions to the soils shown on the sections, were given the same classification.
- Glaciolacustrine units below the ME were assumed to be part of the main glaciolacustrine/glaciofluvial unit (referred to herein as the "lower glaciolacustrine unit").
- Glaciolacustrine/glaciofluvial units above El. 917 m were classified as units within the "upper tills" (referred to herein as the "upper glaciolacustrine units").
- Till below the main glaciolacustrine/glaciofluvial unit was classified as "lower till".

KCB has divided the glaciolacustrine and glaciofluvial soils into two separate units: soils within the main glaciolacustrine/glaciofluvial unit below the ME and below El. 917 m, and soils interlayered within the upper tills. Data suggests that the glaciolacustrine and glaciofluvial layers within the upper tills are differentiated from the main glaciolacustrine/glaciofluvial unit by their variable consistency and wide distribution across site, both spatially and in elevation. Since glaciolacustrine sediments are deposited sub-aqueously, horizontally, and often contain high percentages of silt and clay, they typically represent weak units within the foundation and are therefore of particular geotechnical concern.

A simplified schematic illustrating the generalized soils units is presented on Figure 4.1.

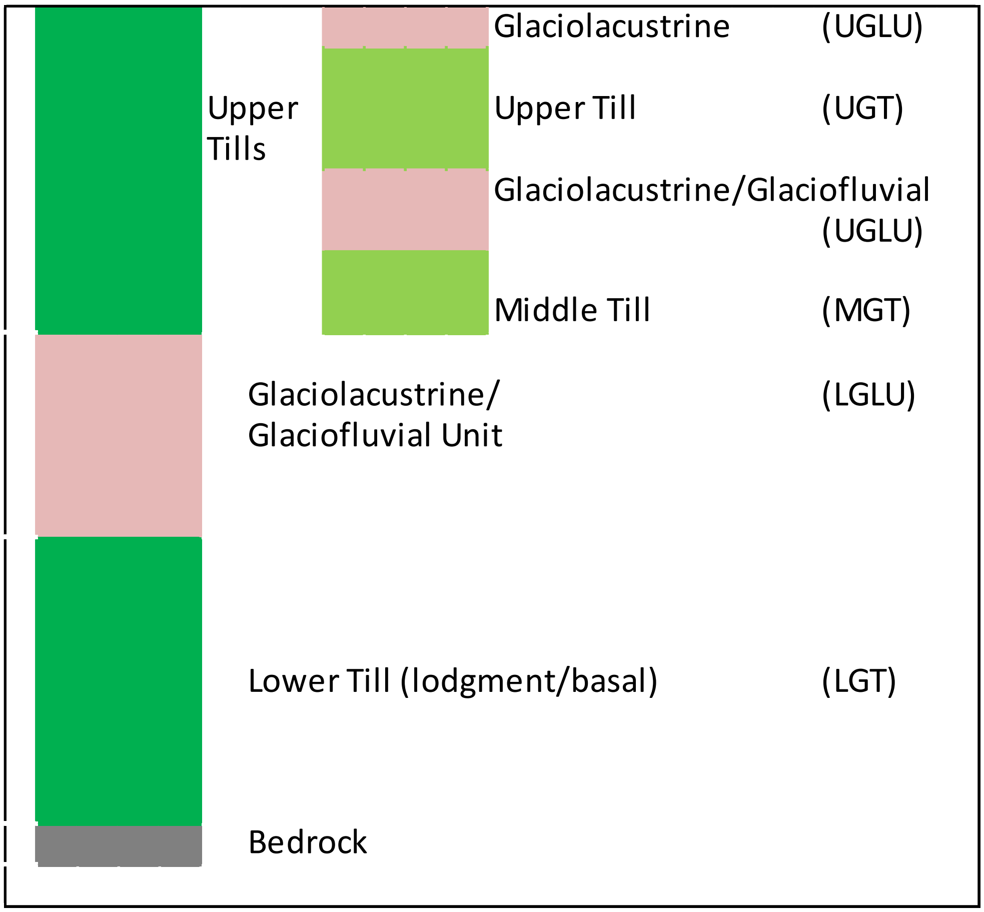


Figure 4.1 Schematic of Generalized Soil Units

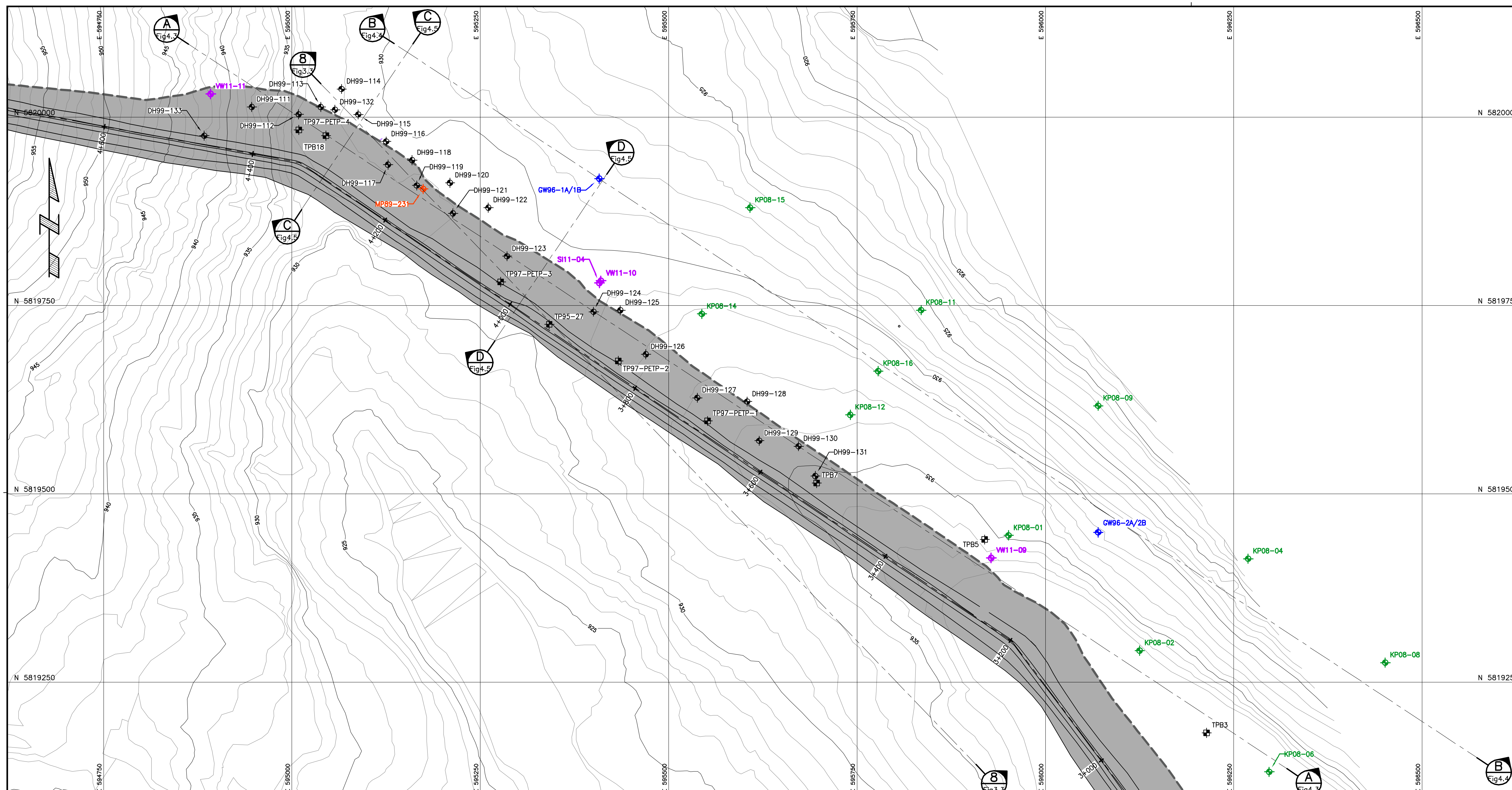
4.2.2 Geologic Model

The location of the PE site investigation drill holes and test pits is shown on Figure 4.2. Profiles along the length of the PE are shown on Figure 4.3 and Figure 4.4. Sections through the PE are shown on Figure 4.5.

The appropriateness of the geologic models developed by the consultants and presented in Section 3.4 of this report was reviewed within the context of the relatively complex geological setting and geological history described in the previous sections of this report. Some of the challenges and key observations that can be made in trying to support or modify the proposed geologic models include the following:

- The PE is approximately 2 km in length, with a current maximum dam height of approximately 40 m, which is a relatively long stretch of foundation conditions that need to be quantified.
- The majority of the site investigations within the PE footprint were shallow drill holes and test pits. Prior to 2011, only one drill hole extended to bedrock (MP89-231), and the tricone rotary drilling method only employed sampling of the drill cuttings as the hole progressed. This type of sampling is not adequate to quantify the soil properties or to identify thin layers.
- The 2011 drilling program included three locations near the downstream toe of the PE. However, the spacing on the drilling was approximately 500 m apart, which again does not provide adequate coverage.
- While there is ample evidence of upper glaciolacustrine units downstream of the PE. There was no quantitative data that they were present beneath the dam footprint. The glaciolacustrine units identified near surface in the breach area were reported to have been excavated as part of the dam foundation preparation.

The following assessment of material properties and stability analyses utilize the geologic model that has the Upper Till units as <u>not</u> including any upper glaciolacustrine soils. An update of the geologic model, using data collected from the breach assessment, is presented in Section 5.3 of this report.



PLAN VIEW

SCALE 0 100 m

LEGEND:

— · — · —	SET OUT LINE FOR TAILINGS EMBANKMENT
— — —	DAM TOE LINE PRE-FAILURE (INFERRED)
M89-231	KP – TRICONE ROTARY DRILLHOLE
GW96-1A	KP – GROUNDWATER MONITORING WELL WITH SPT'S
KP08-01	KP – SHALLOW SONIC DRILLHOLE
DH99-111	KP – SHALLOW DRILLHOLE
VW11-09 SI11-04	AMEC – SONIC DRILLHOLE TO BEDROCK (VIBRATING WIRE PIEZOMETERS AND SLOPE INCLINOMETERS)
TPB3 TP97-PETP1	KP – TEST PIT

NOTES:

1. CONTOURS DIGITIZED FROM TAILINGS STORAGE FACILITY FOUNDATION PREPARATION LINES DWG 1625.101 Rev.1 FROM KNIGHT PIESOLD MAY 26, 1995 (MP0001).
2. DATUM: UTM NAD 83 ZONE 10.

CLIENT
BC MINISTRY OF ENERGY AND MINES

Klohn Crippen Berger

PROJECT
MT. POLLEY DAM FAILURE

TITLE
PREVIOUS SITE INVESTIGATION LOCATIONS WITH ORIGINAL GROUND CONTOURS

PROJECT No. M09954 A01

FIG. No. 4.2

KCB-R-MLD

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Date: 15/07/2015
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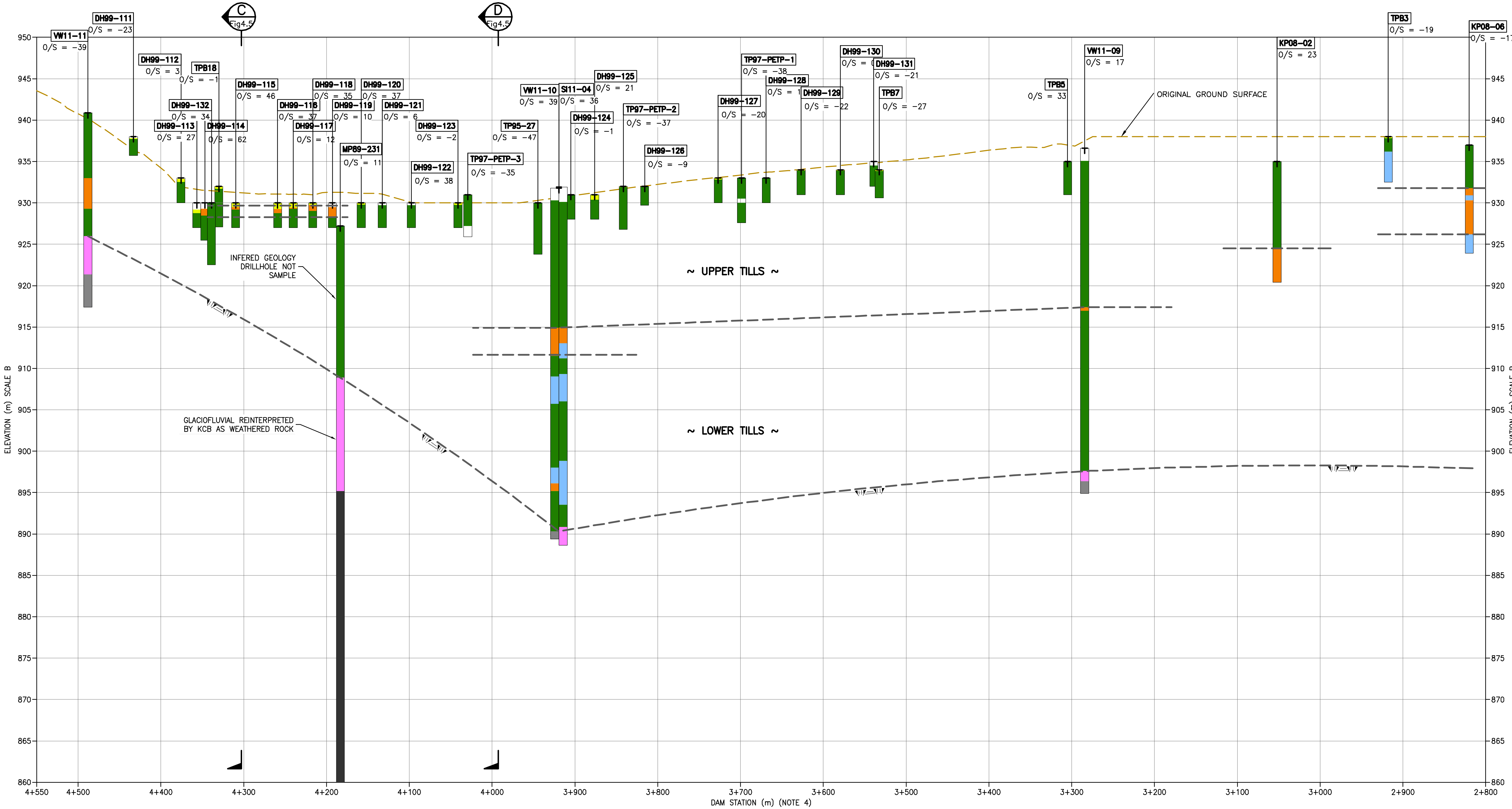
Drawing File: \\VCR-FS-V\Projects\M\M09954A01-MEM-Confidential Project\400 Drawings\Cad\Plan&Sections-WithAsBuilt\Figures-Report\MEM Inquiry Report\Fig4.2-4.3-4.4-4.5 – OriginalSiteInvest&Sections.dwg (persyem)
Xrefs: BM-Tulsequah_Apr08



SECTION A
SCALE A
Fig4.2

SCALE A 0 100 m
SCALE B 0 10 m

LEGEND:

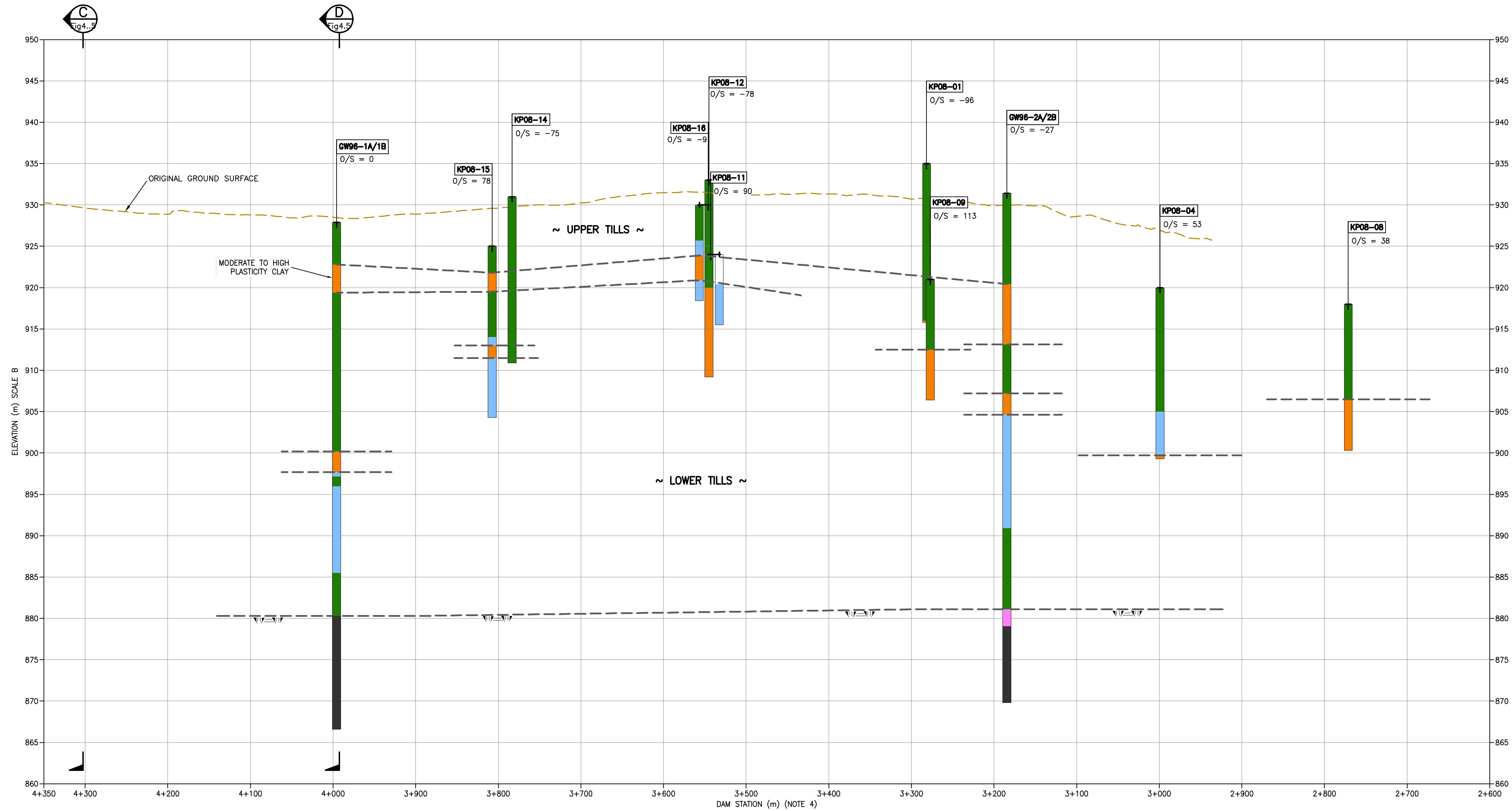
- FILL
- TOPSOIL
- TILL
- GLACIOLACUSTRINE SEDIMENTS
- GLACIOFLUVIAL SEDIMENTS
- WEATHERED BEDROCK
- UNSPECIFIED BEDROCK
- VOLCANIC CONGLOMERATE
- NOT CLASSIFIED
- INFERED GLACIOLACUSTRINE LAYER
- INFERED BEDROCK SURFACE

NOTES:

1. FOR GENERAL NOTES SEE FIGURE 4.2.
2. THIS SECTION INCLUDES INVESTIGATIONS LOCATED WITHIN 50 m EITHER SIDE OF THE SECTION LINE.
3. INVESTIGATIONS WITH NEGATIVE OFFSET ARE LOCATED LEFT OF THE SECTION LINE.
4. STATIONS WERE PROJECTED FROM THE SET OUT LINE ONTO THE SECTION.

CLIENT	BC MINISTRY OF ENERGY AND MINES
	Klohn Crippen Berger
PROJECT	MT. POLLEY DAM FAILURE
TITLE	SECTION A
PROJECT No.	M09954 A01
FIG. No.	4.3

KCB-R-MLD



SECTION B
SCALE A
Fig4.2

SCALE A 0 100 m

SCALE B 0 10 m

LEGEND:

- FILL
- TOPSOIL
- TILL
- GLACIOLACUSTRINE SEDIMENTS
- GLACIOFLUVIAL SEDIMENTS
- WEATHERED BEDROCK
- UNSPECIFIED BEDROCK
- VOLCANIC CONGLOMERATE
- NOT CLASSIFIED
- INFERED GLACIOLACUSTRINE LAYER
- INFERED BEDROCK SURFACE

NOTES:

1. FOR GENERAL NOTES SEE FIGURE 4.2.
2. THIS SECTION INCLUDES INVESTIGATIONS LOCATED GENERALLY 100 m EITHER SIDE OF THE SECTION LINE.
3. INVESTIGATIONS WITH NEGATIVE OFFSET ARE LOCATED LEFT OF THE SECTION LINE.
4. STATIONS WERE PROJECTED FROM THE SET OUT LINE ONTO THE SECTION.

CLIENT	BC MINISTRY OF ENERGY AND MINES
	Klohn Crippen Berger
PROJECT	MT. POLLEY DAM FAILURE
TITLE	SECTION B
PROJECT No.	M09954 A01
FIG. No.	4.4

KCB-R-MLD

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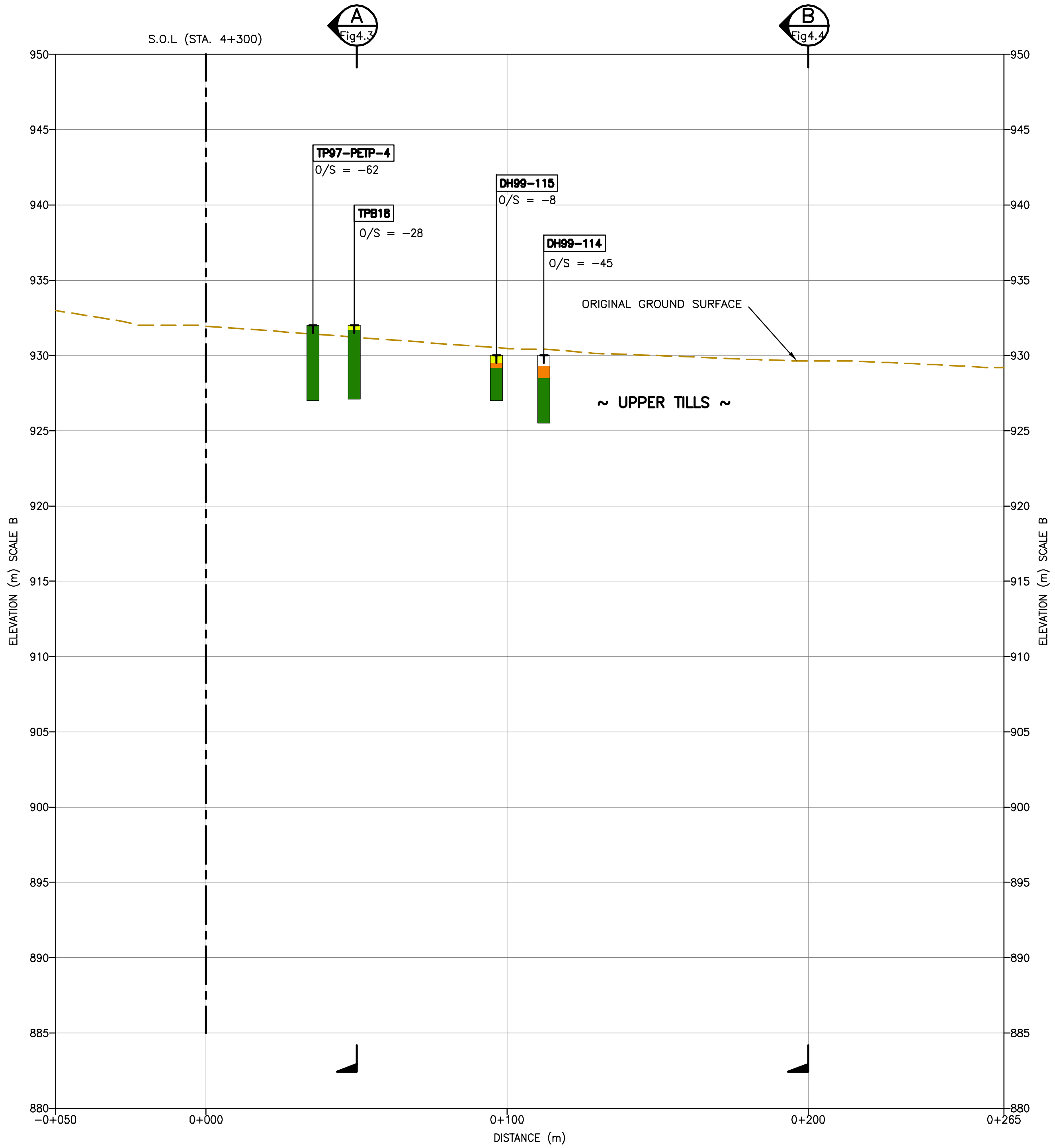
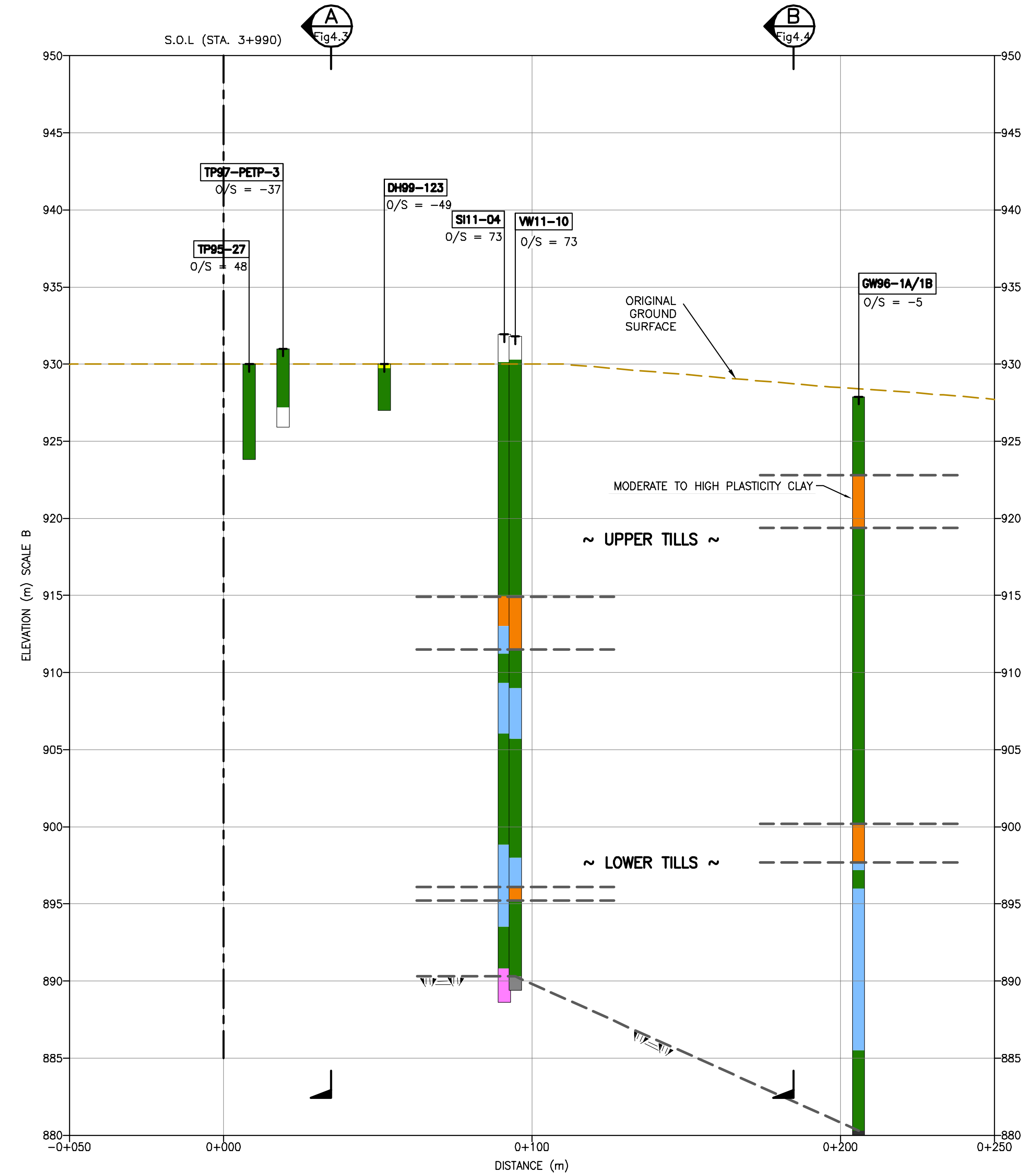
Xrefs: BM-Tulleequah_Apr08



SECTION C
SCALE A
Fig4.2



SECTION D
SCALE A
Fig4.2

SCALE A 0 50 m

SCALE B 0 10 m

LEGEND:

- FILL
- TOPSOIL
- TILL
- GLACIOLACUSTRINE SEDIMENTS
- GLACIOFLUVIAL SEDIMENTS
- WEATHERED BEDROCK
- UNSPECIFIED BEDROCK
- VOLCANIC CONGLOMERATE
- NOT CLASSIFIED
- INFERED GLACIOLACUSTRINE LAYER
- INFERED BEDROCK SURFACE

NOTES:

1. FOR GENERAL NOTES SEE FIGURE 4.2.
2. THIS SECTION INCLUDES INVESTIGATIONS LOCATED WITHIN 100 m EITHER SIDE OF THE SECTION LINE.
3. INVESTIGATIONS WITH NEGATIVE OFFSET ARE LOCATED LEFT OF THE SECTION LINE.

CLIENT	BC MINISTRY OF ENERGY AND MINES
	Klohn Crippen Berger
PROJECT	MT. POLLEY DAM FAILURE
TITLE	SECTION C AND SECTION D
PROJECT No.	M09954 A01
FIG. No.	4.5

KCB-R-MLD

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Date: 15/07/2015
Scale: 1:2(PS)
Drawing File: \\VCR-FS-V\Projects\M\M09954A01-MEM-Confidential Project\400 Drawings\Cad\Plan&Sections-WithAsBuilt\Figures-Report\MEM Inquiry Report\Fig4.2-4.3-4.4-4.5 - OriginalSiteInvest&Sections.dwg (persyem)
Xrefs: BM-Tulsequah_Apr08

4.2.3 Geotechnical Soil Units

Upper Tills

Shallow investigations that encountered glaciolacustrine soils in the upper tills are discussed below:

- Borrow Area No. 1: Glaciolacustrine top elevations ranged from 923 m to 920 m and units were variably described as sand and silt to laminated silt and clay. Soil consistency, when described, ranged from firm to very stiff.
- Borrow Area No. 2: Descriptions of these units were not available in the site investigation reports. Top elevation of glaciolacustrine sediments across Borrow Area No. 2 were highly variable, ranging from approximately 925 m to 934 m. The soils typically appeared at the bottom of the drill holes so typical unit thicknesses are unknown.
- Borrow Area No. 3: Glaciolacustrine units were exposed near-surface on a ridge. Descriptions of the units ranged from fine sand to silt and clay. Top elevations of these units were variable but all were above 930 m.
- Borrow Area No. 4: A layer of varved glaciolacustrine fine sand, silt, silty sand, clayey silt and silty clay was encountered at El. 933 m in test pit TP97-BA4-I6. The sediments were described as soft and wet. The unit extended below the final depth of the test pit.
- Slightly downstream of the PE between Sta. 4+350 and 4+175: A GLU was identified at surface during the 1999 shallow drilling program. It was typically described as silt to clay, soft to firm and intermediate to high plasticity. Top elevation of the unit was assumed to be approximately 930 m, based on the ground contours in the area. The unit ranged in thickness from 0.1 m to 1.7 m.
- Upstream of the ME within the TSF basin: Several of the 1996 natural basin liner investigation test pits intercepted upper GLUs. These soils had top elevations around 920 m and were typically described as laminated sands and silts with variable clay content. The test pits typically terminated within these GLUs so their thickness is unknown.
- In the vicinity of the SE abutment in the southwest corner of the TSF: GLU was encountered in shallow drill holes DH99-22 through 27, with the exception of DH99-25. It was also found in other investigation sites in the vicinity: DH98-BL-8, DH98-BL-03, TP95-33, DH99-18, DH99-19 and DH99-43. Top elevation of the GLU in these pits ranged from 936 m to 929 m and material was described variably as silt and sand to silt and clay.
- In some shallow test pits near the center of the basin.

Drill holes of note where the upper glaciolacustrine units were described are summarized in Table 4.1.

Table 4.1 Summary of UGLU Soil Observations within the Upper Tills in "Deep" Drill Holes

Drill Hole Identifier	Approximate Area	Top Elevation of Soil Unit (m)	Thickness (m)	Simplified Description
GW96-1	Downstream of PE, Sta. 4+020	923	3.4	laminated clay and silt, SPT 'N' value of 6
GW96-2	Downstream of PE, Sta. 3+200	920	7.3	laminated sand and silt or silt and clay, SPT 'N' values 58 and 66
KP08-02	Downstream of PE, Sta. 3+100	925	>4.1[1]	poorly graded massive sand with some clay
KP08-06	Downstream of PE, Sta. 2+850	932 (Unit 1) 930 (Unit 2)	0.9 (Unit 1) 4.1 (Unit 2)	Unit 1: silt and clay Unit 2: sand and silt
KP08-12	Downstream of PE, Sta. 3+550	920	>10.8[1]	silty sand with some gravel and trace cobbles, variable clay content
KP08-15	Downstream of PE, Sta. 3+800	922	2.3	clayey, massive silt with some gravel
KP08-16	Downstream of PE, Sta. 3+550	924	3.0	moderate to highly plastic clay, silt and sand
VW11-02	Downstream Toe of SE, Sta. 1+100	935	0.6	hard, low plasticity clayey silt,
VW11-11	Downstream Toe of PE, Sta. 4+450	933	3.7	silt, some gravel, trace clay, trace sand

Notes: 1. Unit was encountered at the bottom the drill hole.

Lower Glaciolacustrine/Glaciofluvial Unit

A relatively thick deposit of glaciolacustrine and glaciofluvial sediments exists below the ME. Additionally, glaciolacustrine deposits have also been noted at a similar elevation below and downstream of the PE in the drill holes summarized in Table 4.2. Consistency descriptions and SPT blow counts, when recorded, indicate that the soils are very stiff to hard or very dense. KP described the glaciolacustrine sediments as "often highly overconsolidated and very stiff to hard" (KP 1997b, pg. 22). AMEC also described the condition of the glaciolacustrine sediments below the ME as overconsolidated (AMEC 2012a, pg. 16).

Table 4.2 Summary of LGLU Deposits below the Perimeter Embankment

Drill Hole or Test Pit Name(s)	Approximate Area	Top Elevation of Soil Unit (m)	Thickness (m)	Simplified Description
KP08-01	Downstream, Sta. 3+300	916	>0.3[1]	clayey silt, moderate plasticity, very dense, massive
KP08-04	Downstream, Sta. 3+100	900	>0.4[1]	fine sand, low plasticity, poorly sorted, very dense, massive
KP08-08	Downstream, Sta. 2+850	907	>6.2[1]	silt, trace clay, low plasticity, poorly sorted, very dense, massive, grain size increases with depth into sand with some gravel, SPT 'N' value of 50
KP08-09	Downstream, Sta. 3+300	913	>6.1[1]	clayey silt, trace coarse sand, low to moderate plasticity, very dense, poorly sorted, massive, SPT 'N' value of 50
KP08-15	Downstream, Sta. 3+800	913	1.5	fine sand, silt and clay, trace gravel and cobbles, low plasticity, poorly sorted, massive
GW96-1	Downstream, Sta. 4+000	900	3.4	silt, trace sand, trace clay, non-plastic, very stiff, massive?, may be gravel and better graded layers
GW96-2	Downstream, Sta. 3+200	907	2.6	fine sand and silt, very stiff, laminated
SI11-04	Downstream Toe, Sta. 3+900	915	1.9	clayey silt, trace sand, medium plasticity, very stiff to hard, varved
VW11-09	Downstream Toe, Sta. 3+300	917	0.4	clayey silt, trace sand, low plasticity
VW11-10	Downstream Toe, Sta. 3+900	915 (Unit 1) 896 (Unit 2)	3.4 (Unit 1) 0.9 (Unit 2)	Unit 1: silty clay, trace to some sand, medium plasticity, hard, varved Unit 2: silt, some clay, low plasticity, hard, varved

1. Unit was encountered at the bottom of the drill hole

Lower Till Unit

There is little data on the lower till unit. KP described the soil as a "very dense, well graded silt and sand glacial till" and massive, highly consolidated" with some gravel and trace to some clay (KP 1997b, pg. 23-24). The lower till moisture content is comparable to the upper tills, ranging from 6% to 30%, with a median value of 10%. SPT 'N' values within the lower till are very high, indicating that the unit is very dense/hard and well consolidated.

Bedrock

The drill holes in which bedrock was encountered and the geology was described are summarized in Table 4.3. Bedrock was encountered during the 2011 AMEC instrumentation installation and site investigation program, however geologic descriptions of the rock were not made.

Table 4.3 Summary of Bedrock Descriptions

Drill Hole ID	Top of Bedrock Elevation	Bedrock Description
MP89-229	906	Volcanic Conglomerate
	850	Basalt
MP89-230	909	Volcanic Conglomerate
MP89-231	895	Volcanic Conglomerate
MP89-232	893	Volcanic Conglomerate
	858	Mudstone
	842	Sandstone
	829	Volcanic Conglomerate
MP89-233	897	Volcanic Conglomerate
MP89-234	880	Volcanic Conglomerate
MP89-235	942	Volcanic Conglomerate
	846	Basalt
MP89-236	937	Syenite
GW96-1	895	Volcanic Conglomerate
GW96-2	881	Volcanic Conglomerate
GW96-3	877	Volcanic Conglomerate
GW96-4	931	Volcanic Conglomerate
GW96-5	963	Syenite
SI01-02	889	Volcanic Conglomerate
SI06-01	885	Volcanic Conglomerate
SI06-02	890	Volcanic Conglomerate
SI06-03	881	Volcanic Conglomerate

4.3 Quaternary Geology and Soil Distribution

In characterizing the foundation conditions of the site it is important that the geologic history be understood to appreciate the level of complexity that may exist in the distribution of the soil units. Site investigations and interpretation of data can then be appropriately scoped to address the complexity. The following section describes a literature review carried out by KCB, to understand the Quaternary geology of the Mount Polley site. The results of the review were used to support the proposed geological model beneath the TSF.

British Columbia is located within the Canadian Cordillera, a physiographic region that describes the complex of mountain chains that cross the western part of Canada. During the Quaternary Period (2.6 M years ago to present), an amalgamation of glaciers formed a more or less continuous body of ice across BC, known as the Cordilleran Ice Sheet. A general framework for the formation of the Cordilleran Ice Sheet was proposed by Fulton (1991):

- ice caps developed from alpine glaciers on mountain tops at high elevation;
- piedmont lobes developed and coalesced at the mountain fronts; and
- the piedmont lobes developed into an ice sheet with its accumulation area centered over the interior of the Cordillera.

A schematic showing the formation of the Cordilleran Ice Sheet is shown on Figure 4.6. At its maximum extent the top of the ice was approximately El. 2500 m.

Mount Polley is located in a portion of the Province referred to as the Interior Plateau, an area of relatively subdued topographic relief and lower elevation than the Coast and Cariboo Mountains that bracket it to the east and west, respectively. During glaciation, glaciers flowed from the mountains to merge over the Interior Plateau (Claque 1988, Claque 1991).

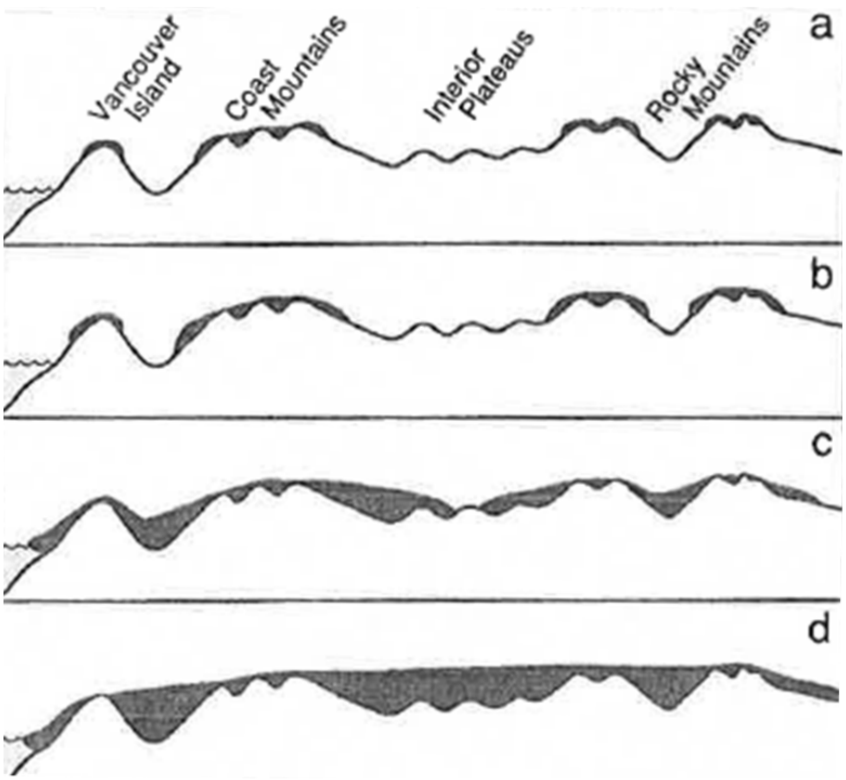


Figure 4.6 Development of the Cordilleran Ice Sheet (from Fulton, 1991)

Several studies have been undertaken to understand the glacial history of the Interior Plateau. Of particular note are those that have focused on the glacial history of the Quesnel and Cariboo River Valleys, due to their proximity to Mt. Polley. The study area locations are shown on Figure 4.7. The glacial history of the Bullion Pit area (Clague, Hebda and Mathewes, 1990) was studied and a stratigraphic section is shown on Figure 4.8.

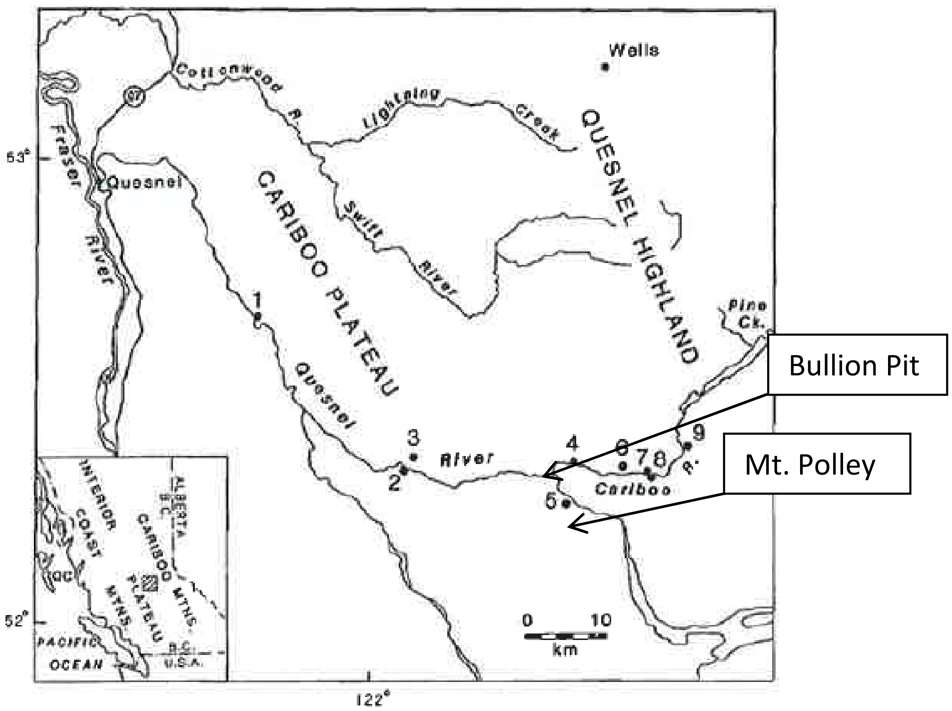


Figure 4.7 Study Locations (Shown as Numbers) in the Quesnel and Fraser Valleys (Clague, Hebda and Mathewes, 1990)

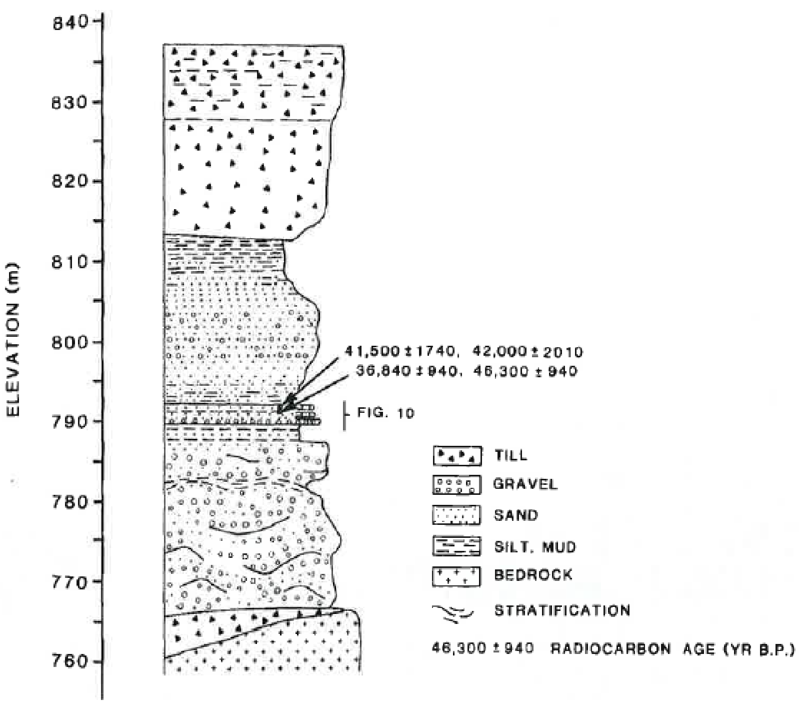


Figure 4.8 Stratigraphic Section at Bullion Pit (Clague, Hebda and Mathewes, 1990)

There is evidence to suggest that two glaciations occurred within these valleys; the penultimate glaciation (>35,000 bp) and the Fraser Glaciation (10,000 bp to 25,000 bp). Glaciolacustrine and glaciofluvial sediments were deposited in the valleys during glacial advance and retreat. The glaciolacustrine deposits associated with the penultimate glacial retreat are described as mainly "massive and stratified silt, sand, gravel, and minor diamicton" (well graded soil) (Clague 1991). Structures within these sediments include folds, faults and sedimentary dykes that are both syndepositional (occurred during deposition) and post-depositional (Clague 1991). The second package of glaciolacustrine sediments were deposited at the end of the Fraser Glaciation, and consist of laminated sand, silt and mud. This unit resembles the sediments deposited after the penultimate glaciation but "lack the pervasive deformation and complex intertonguing of lithologies typical of the latter" (Clague 1991).

A surficial geology map of the area surrounding Mount Polley was produced by Bichler and Bobrowsky (2003). The following is a summary of their findings:

- Surficial material in the map area is attributed to the Fraser Glaciation that occurred during the Late Wisconsin.
- Till typically occurs as a thick blanket (>1 m) over the area and is classified as lodgement till in origin. Some less compact, boulder rich tills with a sandy matrix were found north of the Quesnel River and were classified as ablation till.
- Mapping of streamlined subglacial bedforms such as flutings, drumlins, drumlinoid ridges, and crag and tail features indicates that the orientation of glacial movement near the Mt. Polley TSF was approximately 300°. These features are records of glacial movement during the most recent, Fraser Glaciation.
- Glaciofluvial sediments (typically moderately sorted, weak to moderately compacted, cobble or boulder cobble with a sand matrix) cover 6% of the map area and were found in major valleys as terraces, hummocky blankets and fans.
- Glaciolacustrine deposits, organics, colluvium, modern fluvial deposits and land disturbed by human activity (anthropogenically) covered 8% of the map area. It was noted that glaciolacustrine sediments were found to underlie virtually all terraces within major river valleys.

Based on review of site investigation data available before the breach, literature review, and discussions with MEM Quaternary geologist Travis Ferbey (pers. comm.), KCB has developed the following interpretation of the Quaternary geologic history:

- The Lower Till unit, with its high strength and consolidation, was deposited subglacially during the penultimate (first) glaciation. The till is basal in origin and could also be classified as a lodgement till as it appears to blanket the bedrock.
- The Lower Glaciolacustrine/Glaciofluvial Unit was deposited during glacial retreat at the end of the penultimate glaciation, and during advance of the Fraser Glaciation when ice dammed

lakes were formed. Some of these sediments may have also been deposited much later, as glacial advance lakes ahead of the second glaciation.

- A second glaciation occurred during the Late Wisconsin, the Fraser Glaciation. The advance of these glaciers laid down the Upper Tills and glaciolacustrine soils, which are basal in origin. Evidence of interlayering of till with glaciolacustrine and glaciofluvial sediments supports the idea that there were multiple periods of glacial advance and retreat over the TSF site during the Fraser Glaciation. That these deposits occur over a wide range of elevation across the site, is evidence to suggest that the glaciolacustrine and glaciofluvial sediments were deposited within and on the margins of localized ice proximal lakes. Variability in the degree of consolidation of the till and glaciolacustrine deposits within the Upper Tills, compared to the Lower Glaciolacustrine/Glaciofluvial Unit and Lower Till, suggests that the degree of ice loading (ice thickness) was variable during the recent glaciation.
- It is believed that the diamicton within the Upper Tills is basal till due to the presence of subglacial bedforms that were mapped in the vicinity of the TSF, and the gradation and visual descriptions of the soils.

4.4 Geotechnical Parameters

4.4.1 General

KCB reviewed the soil parameters used by the design consultants and assessed their reasonableness based on the state-of-knowledge and practice. The assessment was based on the soil descriptions, index properties, empirical relationships, and, where available, actual strength test data. The assessment is broken down by unit in the following sections.

4.4.2 Upper Tills and Upper Glaciolacustrine Unit

Upper Tills

A till friction angle of 33° was used for the till units below the PE in all previous analyses. Consolidated-undrained triaxial testing performed by KP on reconstituted and compacted till samples collected in test pits TP95-27 and TP95-37 gave effective stress parameters of $\phi' = 35°$ and $c' = 0$ kPa (KP 1995a, pg. 10 and 11). Since the soil was reconstituted, the test is not representative of the in situ conditions.

Index properties for the upper tills are summarized on Figure 4.11, which shows plots of the SPT values and liquidity indices for the available upper till data. Fines content is generally above 50% and is classified as low plasticity clay. Water contents between 10% and 20% are typical, with the majority of results falling between 10% and 15%.

Soil consistency based on SPTs is variable and ranges from 10 to 90 blows per foot (“firm” to “hard”). The median field ‘N’ value is 30 blows per foot, which would correspond to a field soil description of “very stiff to hard”, and correlates to an undrained shear strength of 200 kPa (Terzaghi and Peck 1948).

Liquidity index is a commonly used as an indicator of soil consistency and the degree of overconsolidation. It is defined as follows:

$$LI = \frac{NMC - PL}{LL - PL}$$

where: LL = Liquid Limit
PL = Plastic Limit
NMC = Natural Moisture Content

Soils that have undergone past loading and consolidation, such as basal tills, typically have a negative liquidity index. Sladen and Wrigley (1983) note that lodgement tills, a type of basal till, typically have liquidity indices in the range of -0.35 to -0.1. Of the 15 liquidity index results that could be calculated for the upper tills, 7 fell within this range, and 11 fell in the range of -0.5 to 0. The wide variation in SPT values and the liquidity index indicate that the till units have variable degrees of consolidation.

KCB feels that an effective friction angle of 33° with no cohesion, in the absence of laboratory strength data, is a reasonable assumption for till strength. However, the potential for a lower strength and/or pore pressure generation due to loading should be further examined. Given the importance of the till unit above the glaciolacustrine sediments for embankment stability at the PE, the collection and testing of undisturbed till samples would have been prudent.

Upper Glaciolacustrine Units

There is very little index testing data on the UGLU within the upper tills. Data that was gathered is summarized on Figure 4.12. Water content testing was mostly performed on near-surface soil within the borrow areas, inside and outside of the TSF basin. Results ranged from approximately 6% to 46%, with a median value of 20%. Soil log descriptions from drill hole GW96-1A indicate a softer material with a SPT value of 6 (undrained shear strength on the order of 40 kPa).

4.4.3 Lower Till and Lower Glaciolacustrine/Glaciofluvial Units

Lower Glaciolacustrine Unit (LGLU)

A summary of the site investigation index testing data on LGLU is shown on Figure 4.13. Review of the data indicates the following:

- Water content ranges from 15% to 40% with the majority of the data falling within the 20% to 30% range. The median value is 26%.
- Soil plasticity covers a wide range from non-plastic to high plasticity. However the majority of the data falls within the intermediate plasticity range. The median liquid limit is 37%.
- Soil consistency measured through SPTs is also variable and does not show any particular trend with respect to elevation. However there is a tight grouping of data between SPT 'N' values of 10 and 30 which correspond to undrained shear strengths in the range of 50 kPa to 200 kPa (Terzaghi and Peck 1948).

- Clay content ranging from 10% to 40%, with a median of 27%. Plotting the clay activity with the correlation suggested by Holtz and Kovacs (1981) indicates that the clay mineralogy is primarily illite and kaolinite.

The only laboratory strength testing on undisturbed glaciolacustrine sediments were direct shear tests performed by KP in 2007. The samples were sheared at 3 normal stress levels, 200 kPa, 400 kPa and 800 kPa to maximum 20% strain to determine the peak and ultimate-strain failure envelopes for the soil. Results from the tests as reported by KP are summarized below (KP 2014). Stresses were originally reported in imperial units by KP but have been converted by KCB to SI for consistency.

Sample 03-1:
Peak: $\phi = 21^o$ c = 74 kPa
20% Strain: $\phi = 22^o$ c = 25 kPa

Sample 03-2:
Peak: $\phi = 25^o$ c = 36 kPa
20% Strain: $\phi = 22^o$ c = 22 kPa

KCB interpreted average peak and 20% strain friction angles of approximately 26^o and 23^o from these results, without cohesion. Since the failure envelopes are non-linear, and are functions of normal stress, these values are based off the largest test stress level, 800 kPa. This most accurately represents the stresses below the PE downstream slope.

KCB reviewed the empirical correlations published by Terzaghi et al. (1996) for peak friction angle (ϕ'_p) of cohesive soils (Figure 4.9) and by Stark and Hussain (2013) for residual (ϕ'_r) friction angle (Figure 4.10). These correlations relate frictional strength to a soil's index properties. Index properties from the glaciolacustrine testing were used in estimating design parameters and no attempt was made to try to separate out the limited data from the upper glaciolacustrine units.

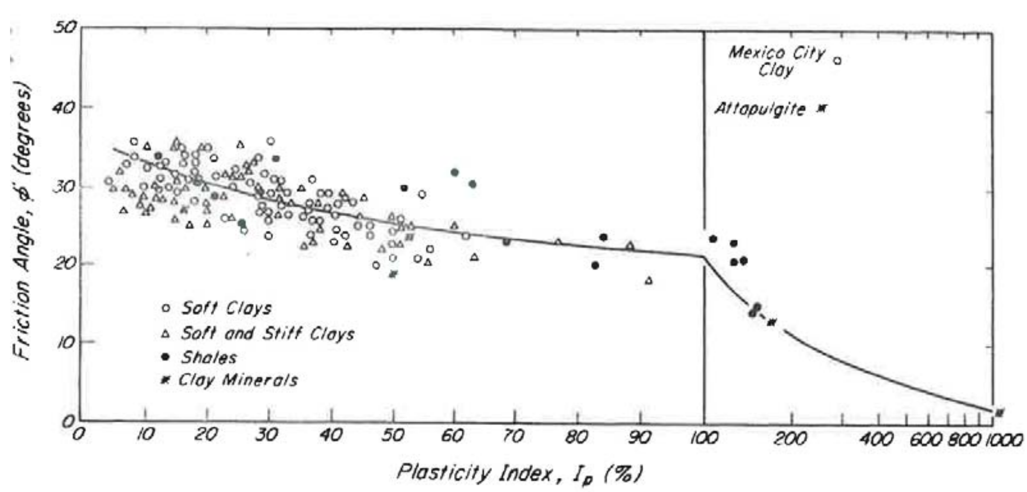


Figure 4.9 Correlation between Peak Friction Angle and Plasticity Index (from Terzaghi et al. 1996)

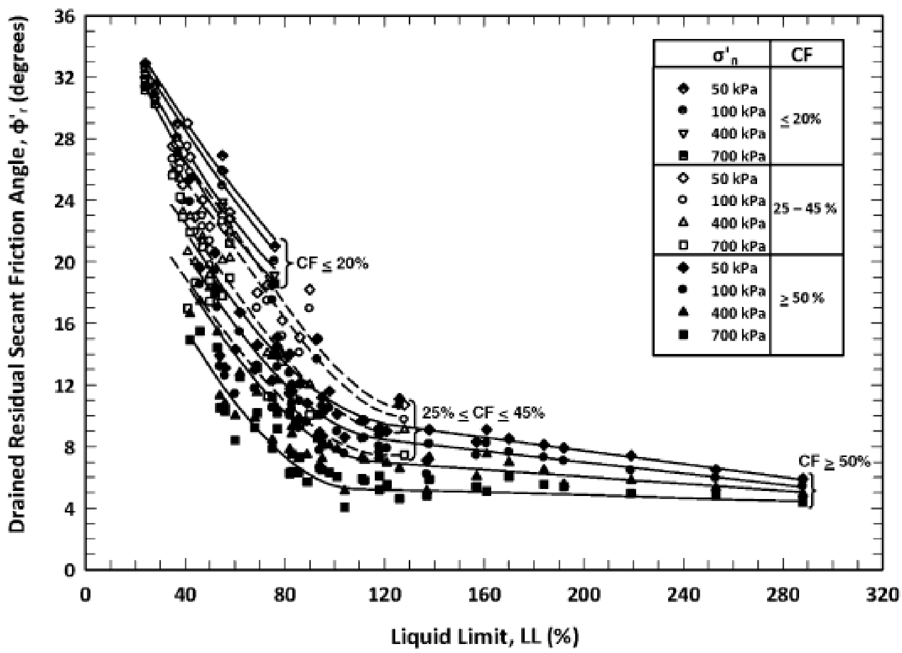


Figure 4.10 Correlation between Residual Friction Angle and Liquid Limit (from Stark and Hussain 2013)

Two sets of strength criteria for the glaciolacustrine unit were developed. One set of data was used to represent the strength in a “homogeneous” soil where the shearing resistance is not affected by the orientation or location of the failure plane. The second set of criteria represents a “layered” soil where the strength varies depending on whether the failure surface is parallel to bedding structure or cutting across bedding structure.

In developing the strength parameters for the “homogeneous” model, KCB used the “upper 2/3” value from the index property data set, in this case plasticity index, liquid limit and clay content. Statistically this means that 2/3 of the data points are less than the chosen design value and 1/3 of the data points are above. When compared to one possible alternative approach of using the mean or median value, this method accounts more for uncertainty in the index properties and is not felt to be overly conservative. The upper 2/3 liquid limit, plasticity index and clay fraction from glaciolacustrine testing is approximately 39%, 16% and 29%, respectively.

For the “layered” model, KCB was guided by the 90th percentile and mean index property values for the horizontal and cross-bedding strengths, respectively. The rationale behind choosing the 90th percentile value is due to uncertainty in how the laboratory samples were collected in the field. It is not stated in the site investigation reports whether discrete clay layers, which represent the weak planes in a layered geology model, were selectively sampled, or whether the varves were mixed during sample handling. The addition of silt to the mixture may lower the liquid limit and plasticity index of the tested specimens. A reasonable upper bound value was therefore felt to be representative of the weakest layer in the foundation. The 90th percentile liquid limit, plasticity index and clay fraction are 47%, 23% and 37%, respectively.

When shearing across bedding, the friction angle of the mixed material will be mobilized. Therefore the mean index properties were deemed appropriate. The mean liquid limit, plasticity index and clay fraction are 36%, 13% and 27%, respectively.

A soil's failure envelope (the expression that relates shear resistance to effective stress) is often approximated as a straight line. In reality however, the failure envelope is generally curved (Terzaghi et al. 1996). Therefore, the friction angle of the soil is stress dependent and was taken into account in developing the empirical correlations. The effective stress below the PE was calculated at the surface of the glaciolacustrine unit below the PE, midway between the embankment crest and the embankment toe. The relatively high stresses below the embankment slope are near the upper limit of the testing data used to develop the residual friction angle correlation. For peak friction angle, a lower bound line was drawn through the lowermost data points to account for the relatively high stresses.

The peak friction angle estimated from the empirical correlation agrees well with the average peak friction angle from direct shear testing. Therefore a design peak friction angle of 26^o was selected. The residual friction angle of 18° is assumed to be mobilized only along a weak, horizontal plane. Selected friction angles are summarized in Table 4.4.

Table 4.4 Summary of KCB Empirically Derived LGLU Strength Criteria

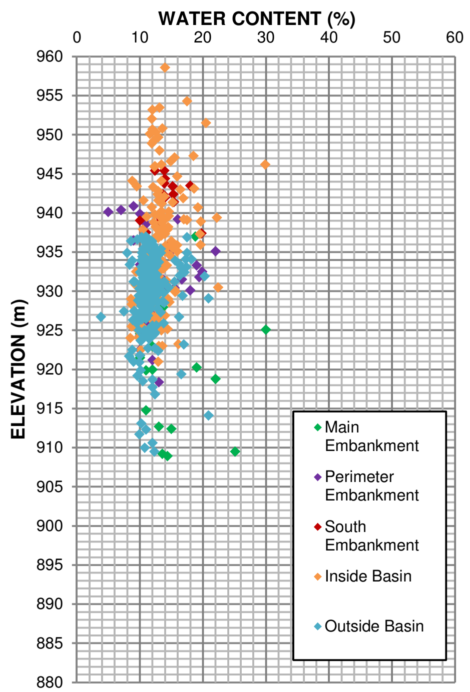
Geology Model	Shearing Direction	ϕ'_p	ϕ'_r
Homogeneous	-	26	-
Layered	Across Bedding	26	-
	Parallel to Bedding	25	18

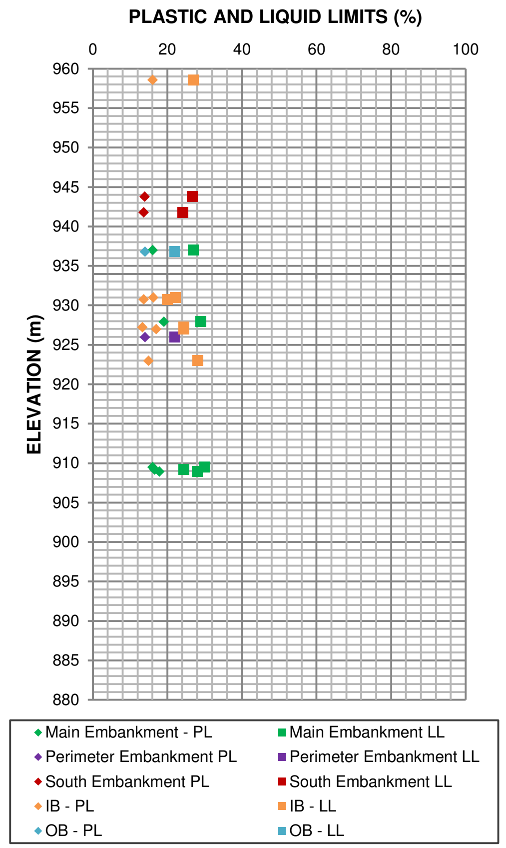
Lower Till

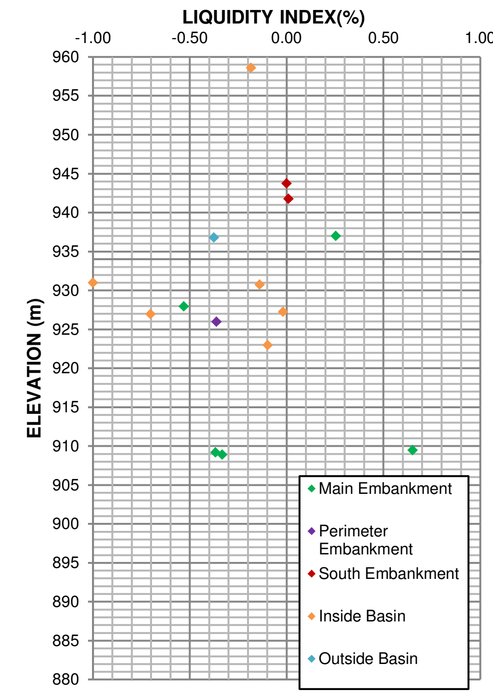
The lower tills have been heavily overconsolidated, likely due to the early Wisconsin glaciation in which an ice thickness of on the order of 1000 m would have been present near Mt. Polley. The index properties of the available data are summarized on Figure 4.14. Very limited data has been collected on the index properties and the main data collections include:

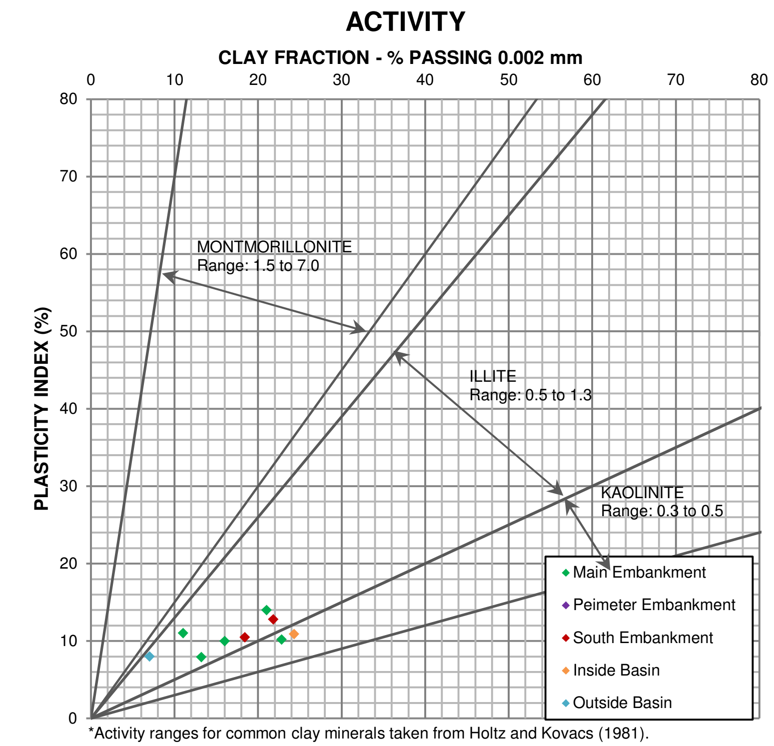
- Natural moisture contents are typically 12% and range from 6% to 30%
- Standard penetration tests are typically 75 blows/ft and range from 50 blows/ft to >100 blows/ft.

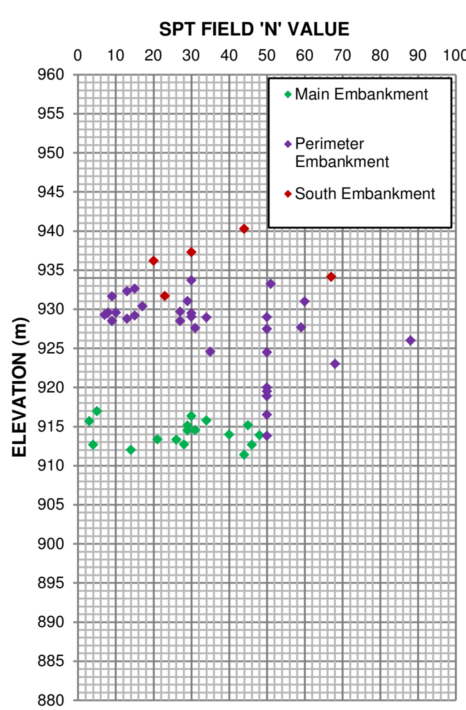
Based on SPT correlations, a peak friction angle of ϕ' = 36°, was selected.

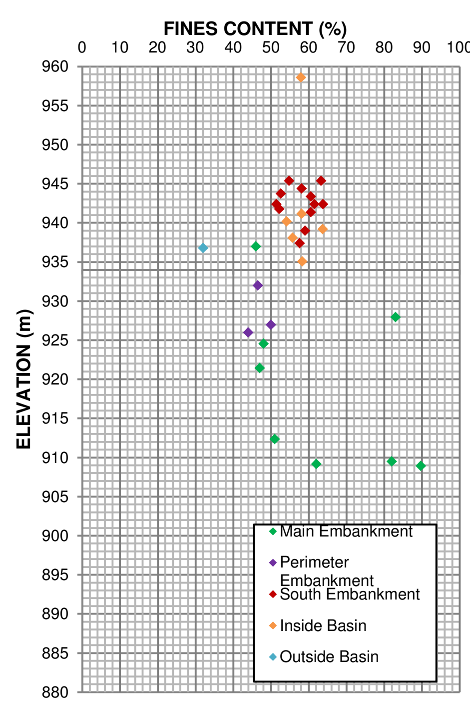


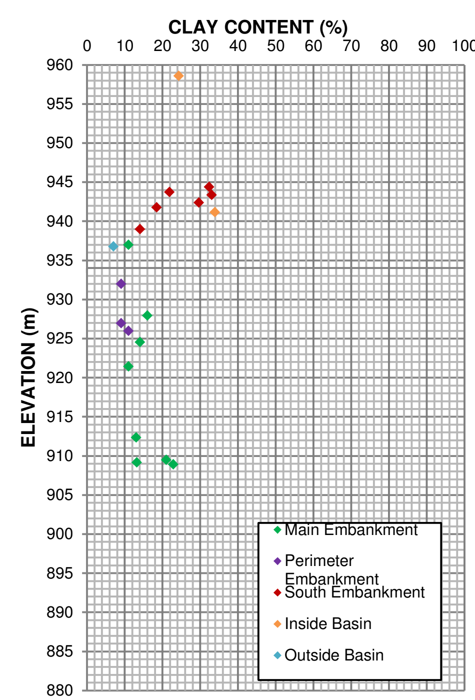


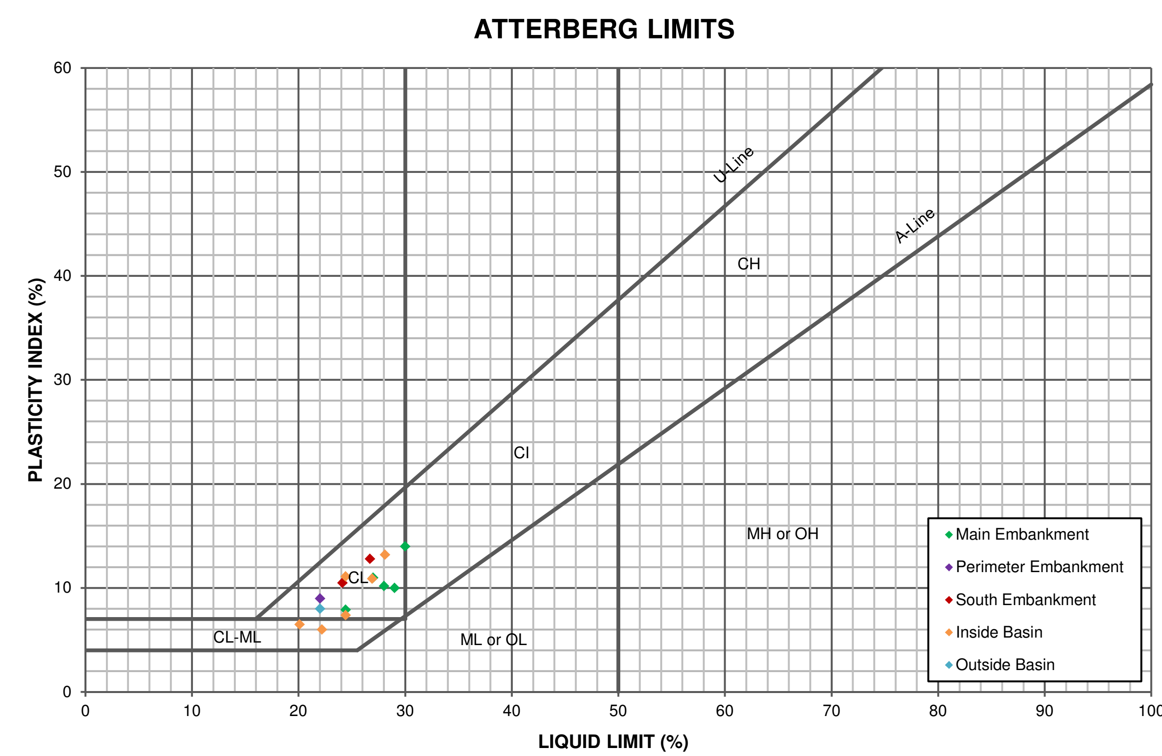




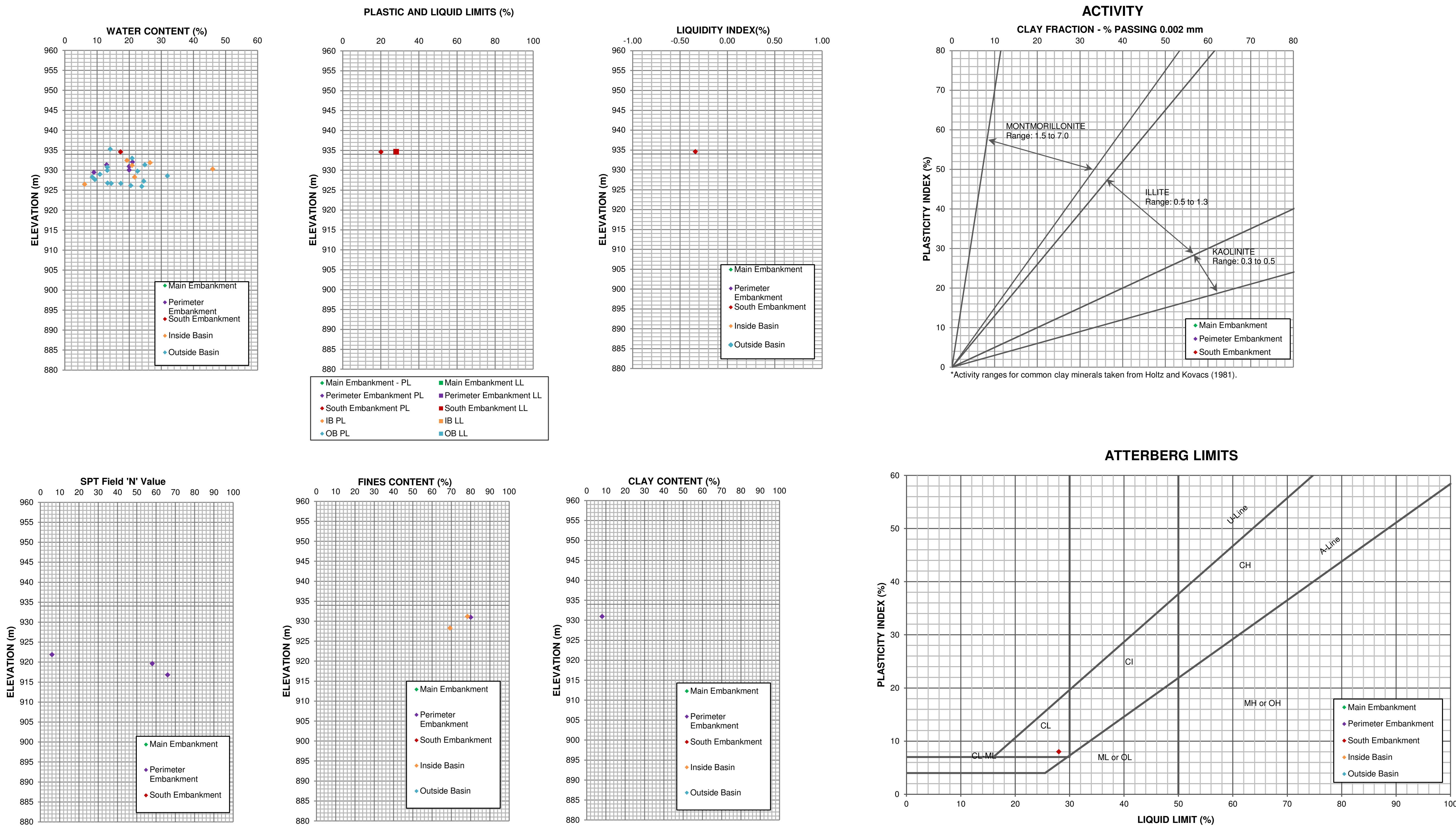




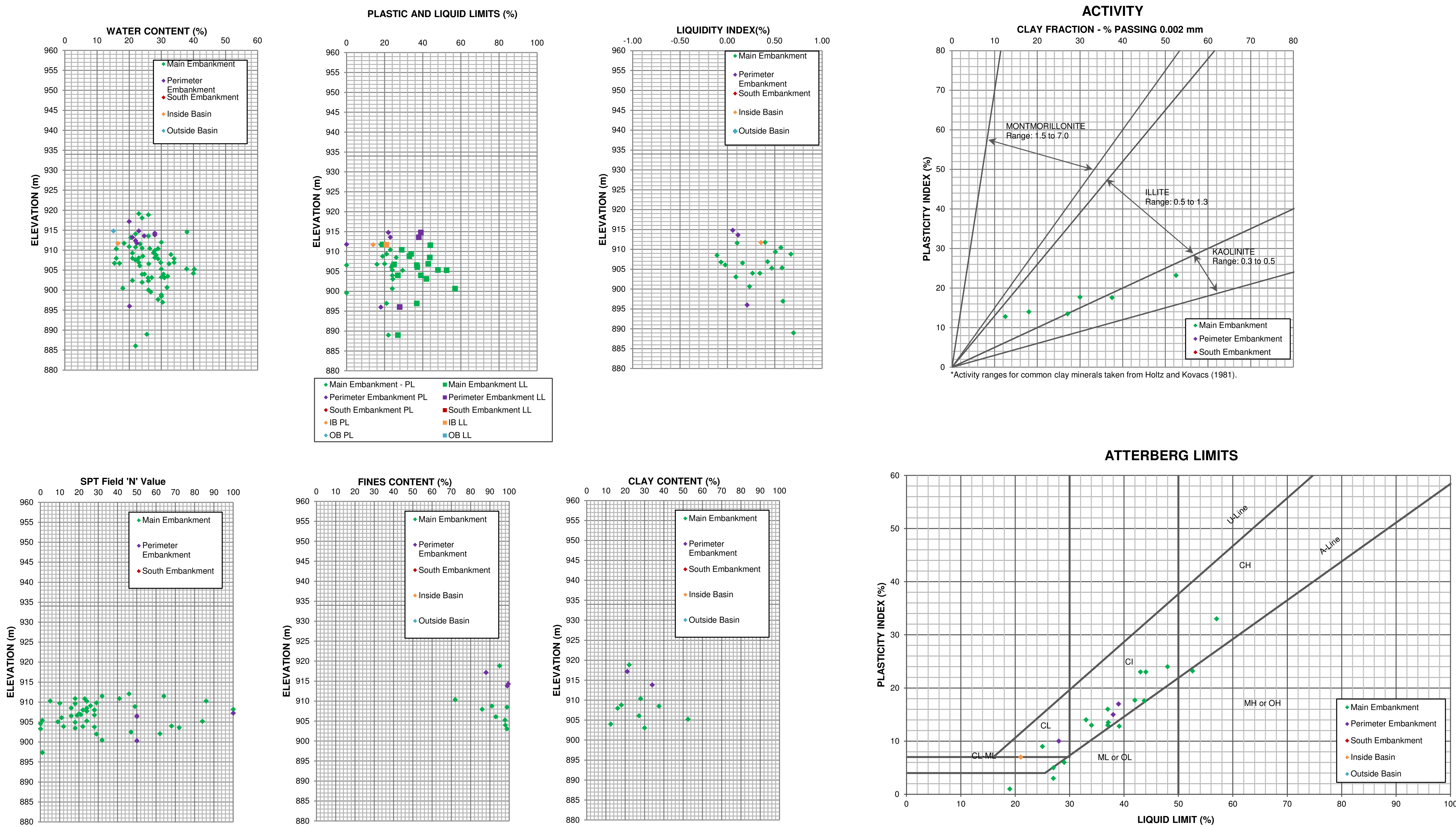




CLIENT	BC MINISTRY OF ENERGY AND MINES
	Klohn Crippen Berger
PROJECT	MOUNT POLLEY DAM FAILURE
TITLE	UPPER TILL
PROJECT No.	M09954 A01
FIG No.	4.11



CLIENT	BC MINISTRY OF ENERGY AND MINES
	Klohn Crippen Berger
PROJECT	MOUNT POLLEY DAM FAILURE
TITLE	GLACIOLACUSTRINE UNITS WITHIN THE UPPER TILLS
PROJECT No.	M09954 A01
FIG No.	4.12



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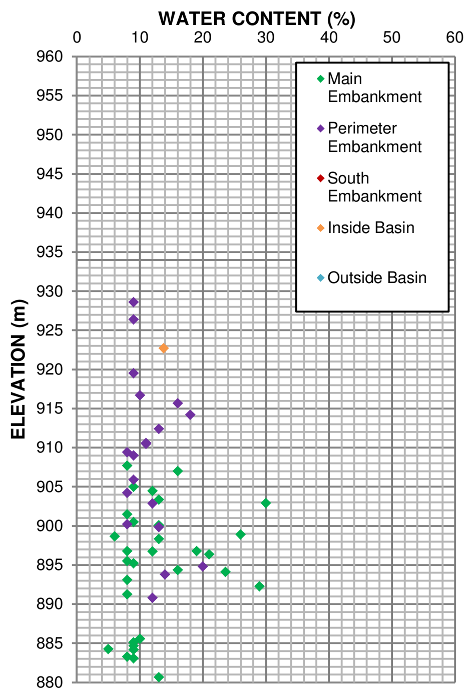
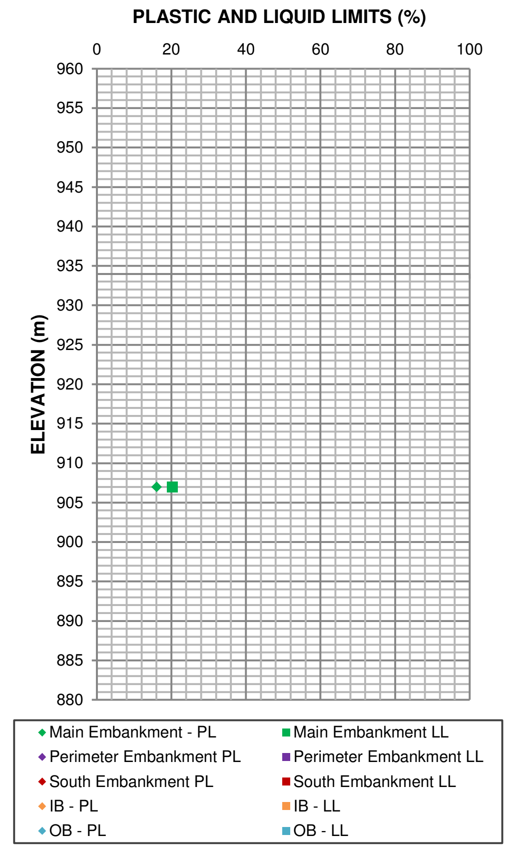
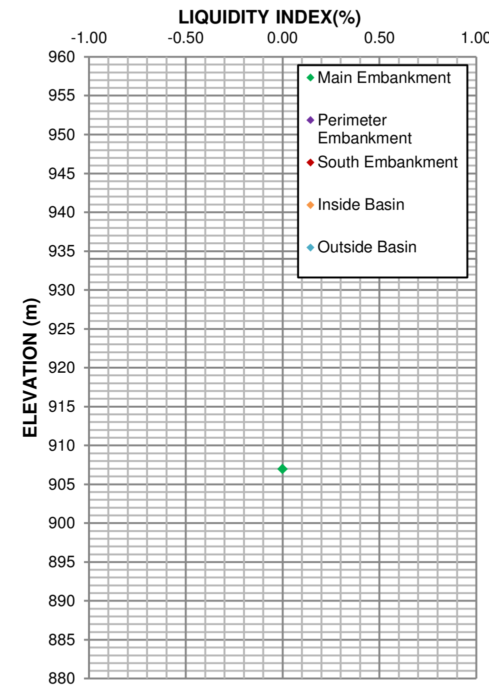
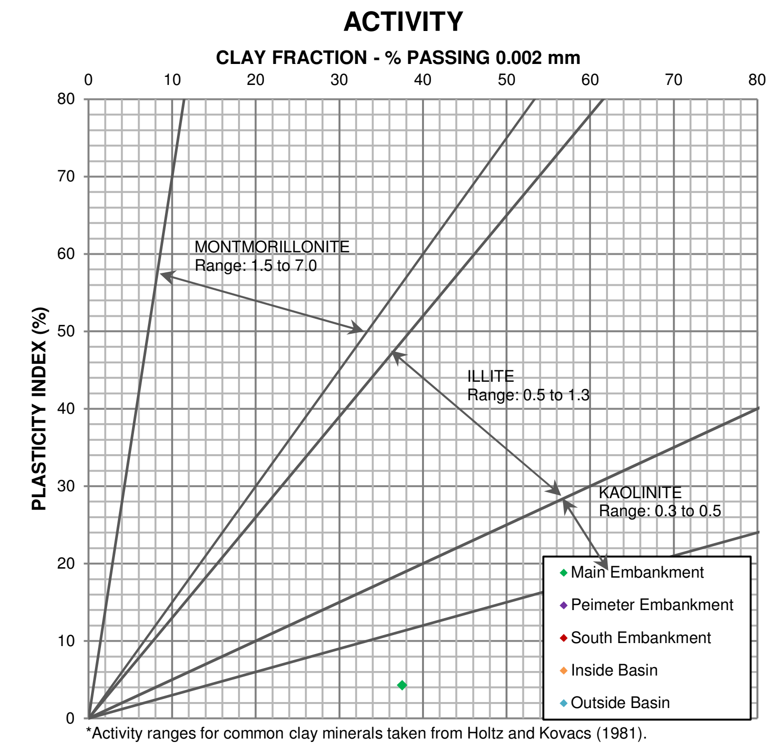
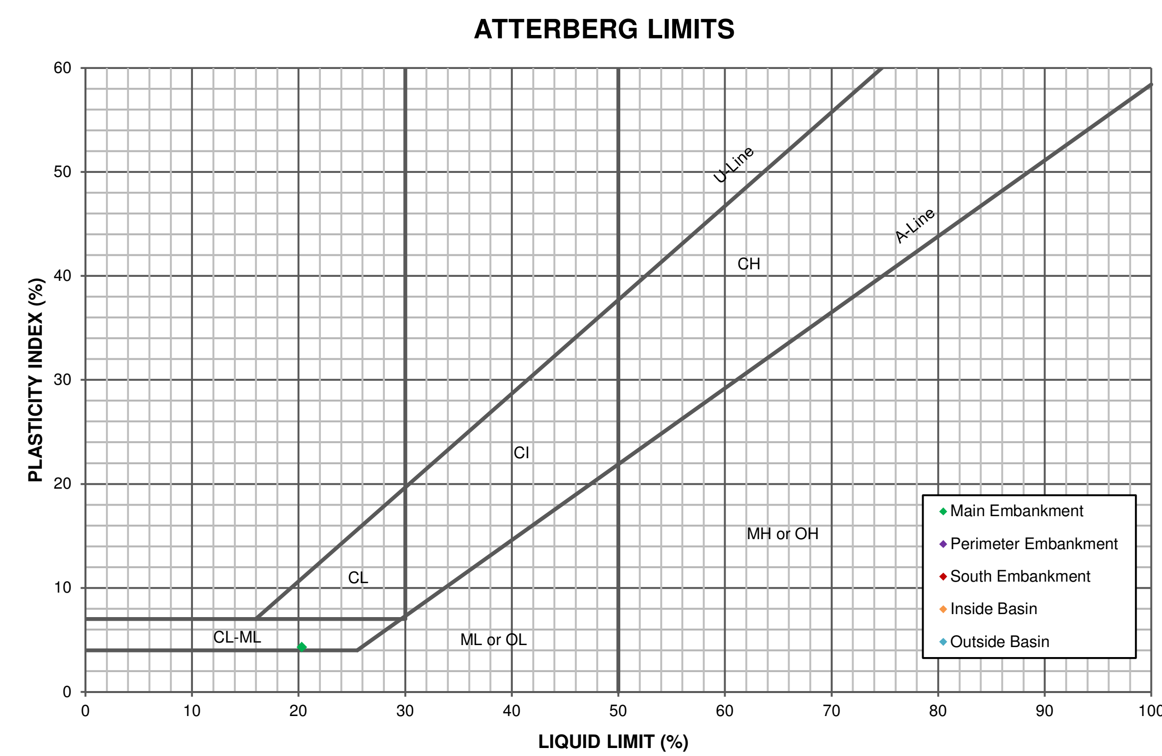
CLIENT	PROJECT
BC MINISTRY OF ENERGY AND MINES	MOUNT POLLEY DAM FAILURE
Klohn Crippen Berger	TITLE: LOWER GLACIOLACUSTRINE UNIT
	PROJECT No. M09954 A01 — FIG No. 4.13

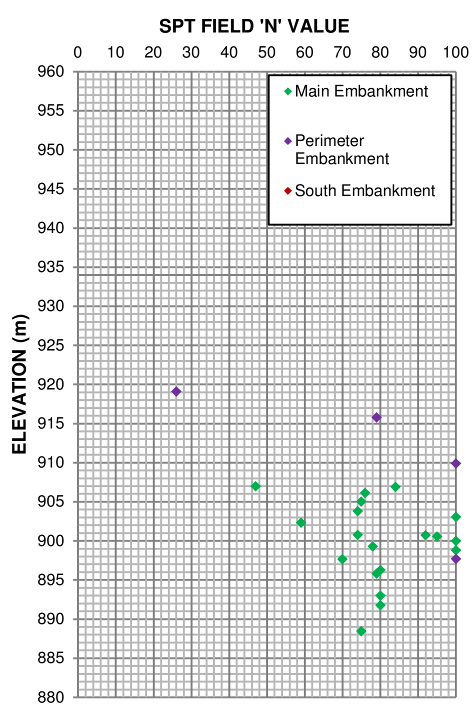


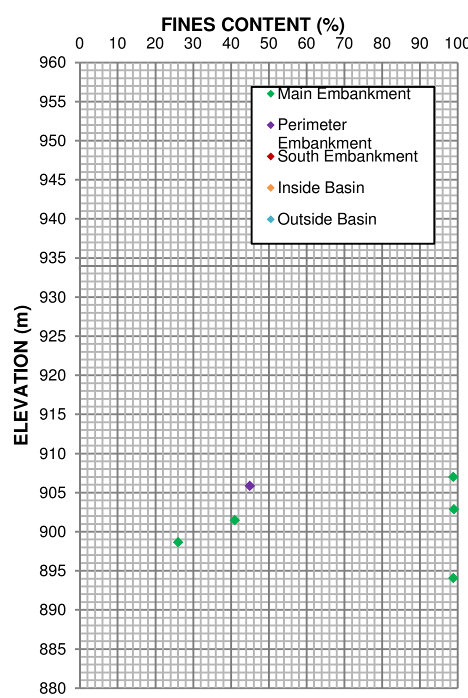


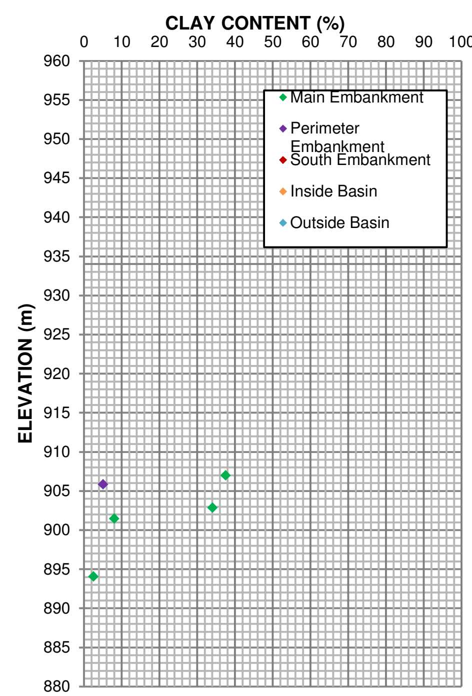












CLIENT
BC MINISTRY OF ENERGY AND MINES

Klohn Crippen Berger

PROJECT
MOUNT POLLEY DAM FAILURE

TITLE
LOWER TILL

PROJECT No.
M09954 A01

FIG No.
4.14

4.4.4 Embankment Fill Zones and Tailings

Embankment Fills

Based on the results of triaxial testing on reconstituted and compacted till samples, a friction angle of 35° for the compacted till core zone is considered appropriate. The strength-stress relationship presented by Leps (1970) for "average rockfill" is appropriate for the angular waste rock placed in Zone C. The relationship is shown graphically in Figure 4.10.

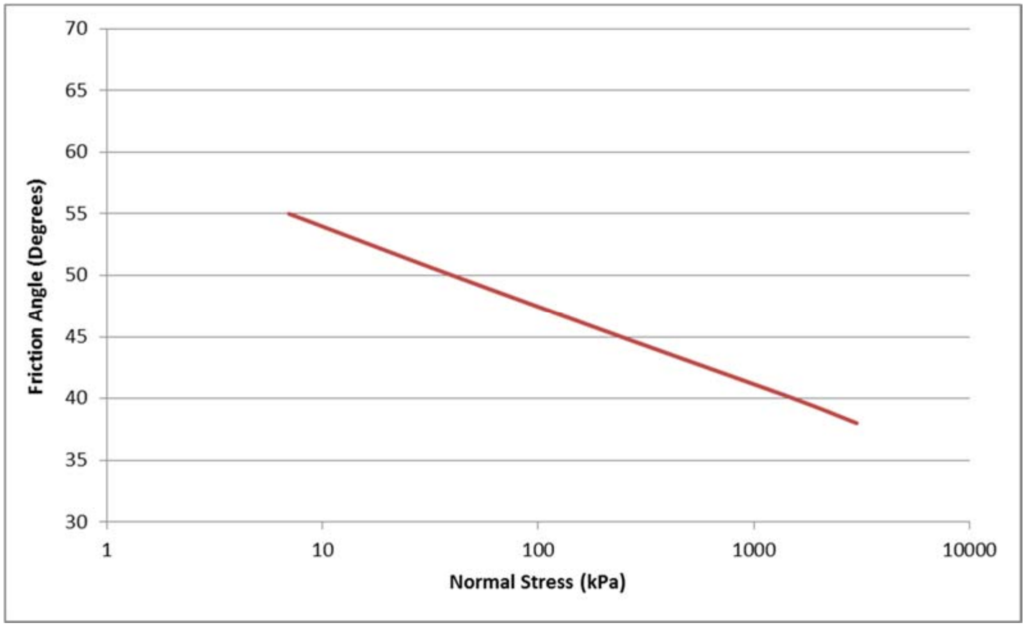


Figure 4.15 Relationship between Friction Angle and Normal Stress for "Average" Rockfill (adapted from Leps 1970)

Tailings

In KCB's experience, the friction angle for porphyry-copper tailings is often measured to exceed 30°. In the absence of laboratory testing, adopting this friction angle seems appropriate. The adoption of an undrained shear strength/effective stress ratio for post-liquefied strength is common. Based on experience on similar projects, a S_u/σ'_v ratio of 0.1 is appropriate for tailings post-liquefied strength.

4.5 Design Sections and Stability Analysis

4.5.1 Calculation Checks on AMEC and BGC Stability Analyses

KCB reviewed the stability analyses carried out by AMEC (2013b) and BGC (2014), as these best represent the geometry and elevation of the PE, and the state of knowledge, close to the time of failure. To carry this out the AMEC and BGC stability models were "rebuilt" by KCB using the same geometry, foundation conditions, phreatic surface assumptions and material properties, and the models were re-run to confirm the reported FoS.

Design Section D, located at Sta. 3+990 was chosen by AMEC and BGC as a critical section for embankment stability.

KCB reconstructed the AMEC model using the limit-equilibrium slope stability analysis software, Slope/W (GeoStudio 2012, December 2014 Release, version 8.14.1.10087). The Morgenstern-Price Method of Slices was used to calculate the global FoS.

The model was run using the same assumptions as AMEC and a comparison between the AMEC and KCB calculated FoS are shown in Table 4.5. Results show that the KCB calculated FoS are approximately 3% lower than AMEC's for both the drained and the undrained tailings cases. This may be a result of the slip surface search method that was used. KCB reviewed a number of different slip surface geometries to find the lowest FoS.

Table 4.5 Comparison of AMEC and KCB Calculated FoS – Section D

Stability Case	AMEC FoS	KCB FoS	Percent Difference
Drained Tailings	1.63	1.59	-3%
Undrained Tailings	1.58	1.53	-3%

The BGC model at Section D was reconstructed by KCB to confirm the calculated FoS. The same analysis software and calculation methods were used as for the AMEC analysis. Results from the comparison are presented in Table 4.6.

Table 4.6 Comparison of BGC and KCB Calculated FoS – Section D

Embankment Geometry	Glaciolacustrine Strength Properties	BGC FoS	KCB FoS	Percent Difference
Crest El. 970 m	Peak	1.47	1.40	-5%
	Residual	1.14	1.11	-3%
Crest El. 972.5 m; no buttress	Peak	1.40	1.34	-4%
	Residual	1.08	1.05	-3%
Crest El. 972.5 m; with buttress	Peak	1.55	1.46	-5%
	Residual	1.15	1.13	-2%

The stability calculation checks indicate slightly lower factors of safety with the KCB analysis than both AMEC and BGC. The differences are not considered significant and are within the normal variation of analyses.

4.5.2 AMEC and BGC Stability Analyses Transferred to the Breach Section

The geometry of the breach area is different than the geometry at Section D and, accordingly, KCB carried out assessment of the stability of the PE using the as-constructed conditions at the location of the PE breach (Station 4+300).

Assessment of Breach Section using AMEC Analysis Basis

The Breach Section was prepared with the following assumptions:

- Foundation soil stratigraphy is unchanged from Section D.
- Material properties are unchanged from Section D.
- The elevation of the ground surface below the embankment is based on the ground surface profile shown on KP's Instrumentation Section G, which was located at Sta. 4+300 (KP 2011a, Dwg. 258).
- The ground surface elevation at the toe and immediately downstream of the embankment is based on the as-built drawings in the Stage 9 construction report (AMEC 2013b, Dwg. 2013.03).
- The elevation of the phreatic surface is unchanged from AMEC's assumptions at Section D. This assumption was made because there were no operating piezometers in the foundation along the breach section.
- The overall downstream slope of the embankment is 1.3H:1V, the same as was assumed at Section D.
- The existing buttress that is present at Section D is not present at the Breach Section.

The same two stability cases were reviewed as for Section D. A comparison between the KCB calculated FoS at Section D and the Breach Section is provided in Table 4.7.

Table 4.7 Comparison of Stability Results at Section D and the Breach Section - AMEC

Stability Case – Crest El. 970.0 m	Section D FoS	Breach Section FoS	Percent Difference
Drained Tailings	1.59	1.37	-14%
Undrained Tailings	1.53	1.33	-13%

Results show that the location of the analysis section has a significant impact of the calculated FoS with FoS at the Breach Section being approximately 14% to 13% lower for the same analysis assumptions. This is attributed to the effect of the existing toe buttress at Section D. The buttress increases the effective stress at the embankment toe, thereby increasing the passive resistance.

It is noted that the slip surface did not pass through the glaciolacustrine/glaciofluvial unit for the AMEC analysis cases, both at Section D and the Breach Section. Even if the effective frictional angle in the glaciolacustrine unit is reduced to 24°, the critical slip surface remains in the upper till. Therefore, had AMEC adopted the 24° friction angle that they used for the ME stability analysis in the Stage 7 design, the calculated FoS would have been unchanged.

Assessment of Breach Section using BGC Analysis Basis

As for the AMEC analysis, the sensitivity of BGC's results to the section location was assessed. A new section G was constructed by KCB with the following assumptions:

- Foundation stratigraphy is unchanged from Section D.
- Material properties are unchanged from Section D.
- The elevation of the ground surface below the embankment is based on the ground surface profile shown on KP's Instrumentation Section G, which was located at Sta. 4+300 (KP 2011a, Dwg. 258).
- The ground surface elevation at the toe and immediately downstream of the embankment is based on the drawings provided in the Stage 9 2013 As-Built and Annual Review Report (AMEC 2014a, Dwg. 2013AB.02). The ground surface does not drop off downstream of the toe since the section does not intersect the SCP.
- The elevation of the phreatic surface is unchanged from BGC's assumptions at Section D.
- The overall downstream slope of the embankment is 1.3H:1V, the same as was assumed at Section D.
- The existing buttress that is present at Section D is not present at the Breach Section.

The same stability cases were run for the Breach Section as Section D. A comparison between the KCB calculated FoS is provided in Table 4.8.

Table 4.8 Comparison of Stability Results at Section D and the Breach Section - BGC

Embankment Geometry	Glaciolacustrine Strength Properties	Section D FoS	Breach Section FoS	Percent Difference
Crest El. 970 m	Peak	1.40	1.38	-1.4%
	Residual	1.11	1.18	6%
Crest El. 972.5 m; no buttress	Peak	1.34	1.32	-1.5%
	Residual	1.05	1.12	6.7%
Crest El. 972.5 m; with buttress	Peak	1.46	1.56	7%
	Residual	1.13	1.22	8%

With the exception of two cases, the FoS at Section D were found to be lower than those at the Breach Section. This is attributed to the drop in ground elevation that BGC assumed at the toe of the embankment at Section D which reduces the passive resistance. Generally this appears to have a greater effect on FoS that than the presence of the existing toe buttress.

4.5.3 KCB Analysis – Section D

KCB carried out stability analyses for the PE based on our review of the assumptions made by AMEC and BGC with respect to the state-of-knowledge at the time and the critical geometry sections. The geology model adopted by AMEC and BGC was adopted and, therefore, did not include any upper glaciolacustrine units in the dam foundation.

Model Set-up and Parameters

The elevation of the natural ground surface, and the piezometric conditions that BGC assumed at Section D were adopted by KCB as they represent the critical case as discussed in Section 4.5.2. The stability models are shown on Figure 4.16 to Figure 4.18.

KCB assigned material properties based on the rationale discussed in Section 4.4. Parameters are summarized in Table 4.9. None of the materials in the stability model were assumed to have cohesion.

Table 4.9 KCB Interpreted Material Properties for Stability Analysis

<table>
<tr><th>Material Type</th><th>Bulk Unit Weight (kN/m^3)</th><th>Friction Angle (degrees)</th></tr>
<tr><td>Glacial Till</td><td>21</td><td>33</td></tr>
<tr><td>Glaciolacustrine Sediments (Peak and Across Bedding)</td><td rowspan="2">20</td><td>26</td></tr>
<tr><td>Glaciolacustrine Sediments (Residual)</td><td>18</td></tr>
<tr><td>Zone S</td><td>20.5</td><td>33</td></tr>
<tr><td>Zone C</td><td>22</td><td>Leps (1970) relationship</td></tr>
<tr><td>Tailings (Drained)</td><td rowspan="2">18</td><td>30</td></tr>
<tr><td>Tailings (Post-Liquefaction)</td><td>$S_u/\sigma'_v = 0.1$</td></tr>
</table>

Analysis Cases and Design Criteria

Static, pseudo-static and post-earthquake stability cases were assessed. Sensitivity to the embankment crest elevation (970 m or 972.5 m) and presence of a toe buttress (for the 972.5 m case only) was checked. The buttress was assumed to be constructed to an elevation of 941 m, as per the BGC stability assumptions. The analysis scenarios are summarized below.

Case 1: Static – Peak Strength: Tailings are assumed to be drained and peak strength is mobilized in the glaciolacustrine unit. The homogeneous soil model is applied to the glaciolacustrine unit.

Case 2: Static – Residual Strength: As for Case 1, except residual strength is mobilized along a horizontal layer within the glaciolacustrine unit. Peak strength is mobilized across bedding planes. A "wedge-type" failure geometry is specified for this scenario to force the failure surface horizontally through the glaciolacustrine unit.

Case 3: Pseudo-Static – Reduced Strength: A horizontal pseudo-static acceleration of 0.09 g is applied, which is equal to half of the OBE peak ground acceleration. Mobilized shear strength in the foundation soils and Zone S is equal to 80% of the peak strength due to generation of cyclically induced pore water pressure. The tailings liquefy.

Case 4: Post-Earthquake – Reduced Strength: As Case 3, except without pseudo-static loading.

The factor of safety uses the CDA 2007 guidelines, and are included in Table 4.10.

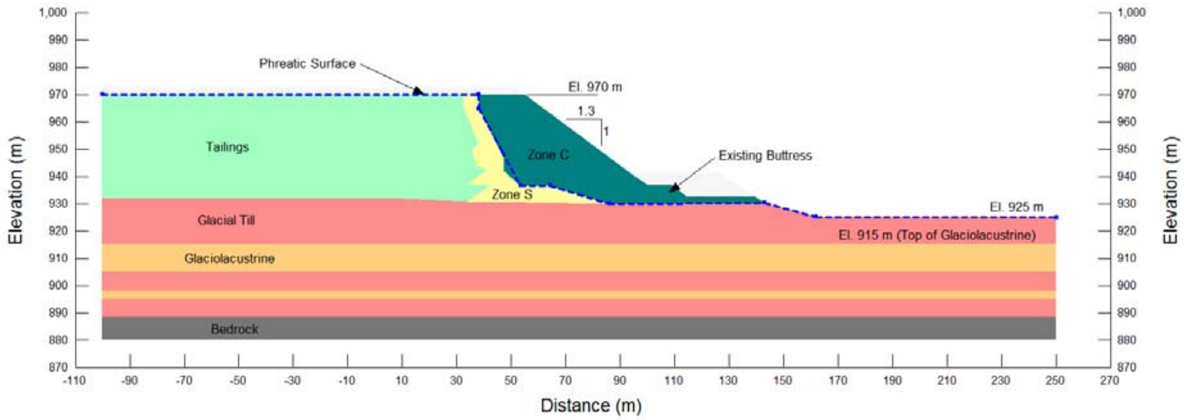


Figure 4.16 KCB Stability Model Set-Up – Section D – Crest Elevation 970 m

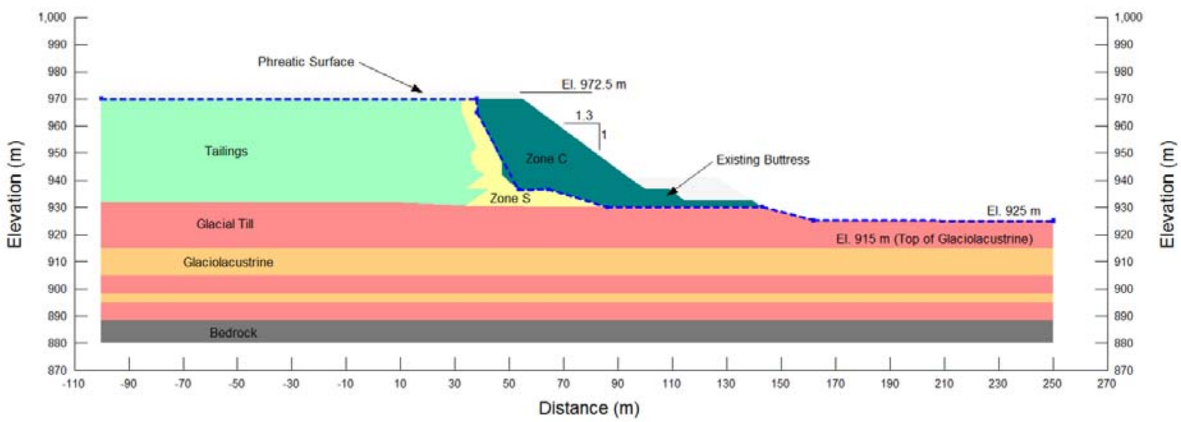


Figure 4.17 KCB Stability Model Set-Up – Section D – Crest Elevation 972.5 m no Buttress

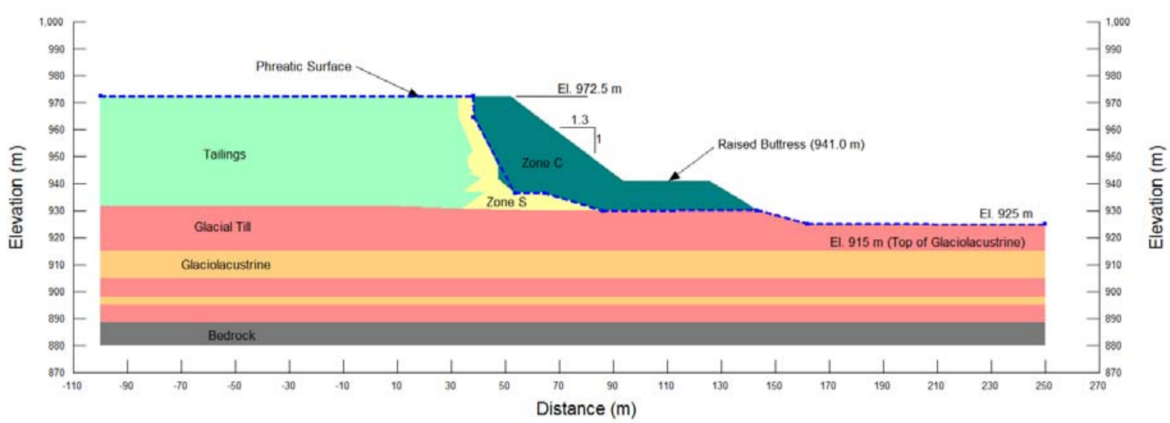


Figure 4.18 KCB Stability Model Set-Up – Section D – Crest Elevation 972.5 m w/ Buttress

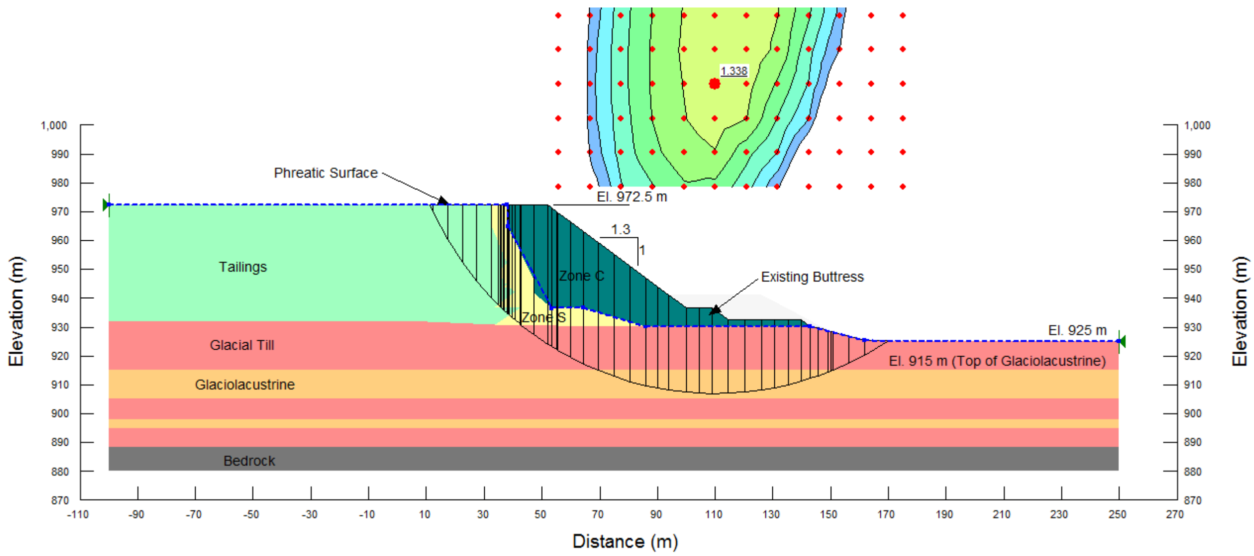


Figure 4.19 KCB Stability Model –Typical Critical Slip Surface with Peak GLU Strength

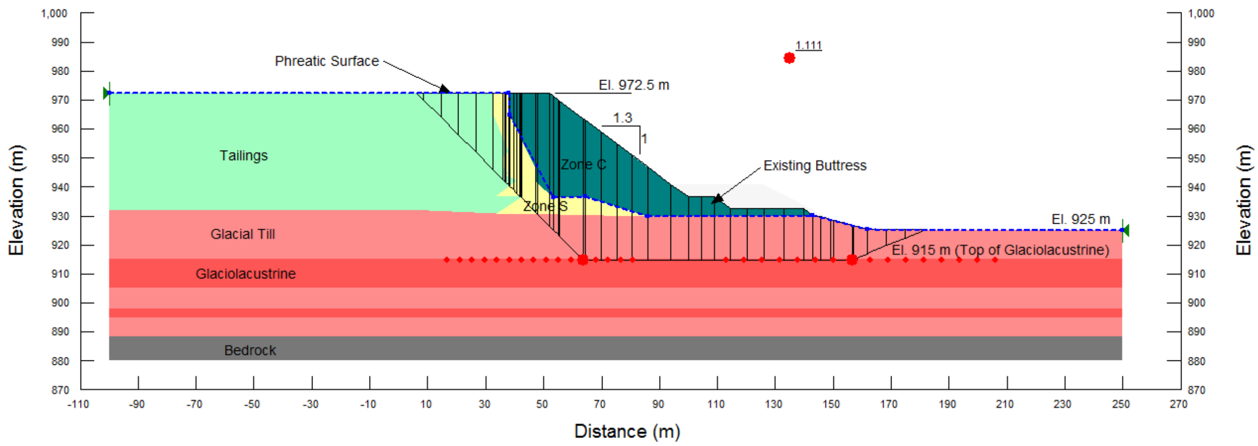


Figure 4.20 KCB Stability Model –Typical Critical Slip Surface with Residual GLU Strength

Analysis Results

FoS for each stability case is summarized in Table 4.10.

Table 4.10 KCB Stability Analysis Results – Section D

Stability Case	CDA FoS Criteria	Embankment Crest Elevation (m)	Toe Buttress? (Y/N)	Calculated FoS
Case 1 Static	1.5	970	N	1.39
		972.5	N	1.34
		972.5	Y	1.44
Case 2 – Residual Strength	Not specified, 1.1 assumed	970	N	1.16
		972.5	N	1.11
		972.5	Y	1.19
Case 3 – Pseudo-static	1.0	970	N	0.80
		972.5	N	0.77
		972.5	Y	0.82
Case 4 – Post earthquake	1.2	970	N	1.04
		972.5	N	1.00
		972.5	Y	1.07

Results show that the minimum FoS for Case 1 is not achieved when a buttress is not present for both the 970 m and 972.5 m crest elevations. With a buttress in place, the FoS is slightly below design criteria. A typical critical slip surface is shown on Figure 4.19 and Figure 4.20 for peak and residual GLU shear strength, respectively.

A FoS close to unity for Case 4 may indicate that seismic deformations could be unacceptably high in a seismic deformation analysis. Had the pseudo-static case (Case 3) been used for design, it is likely that either it or Case 4 would have been the controlling case which would have dictated the maximum allowable downstream slope and/or buttressing requirements.

4.5.4 Summary of Pre-Breach Stability Analyses

KCB carried out design checks of the AMEC and BGC stability analyses and the results, assuming the same input parameters, are similar (± 5%), as would be expected (Table 4.5Table 4.6).

A comparison of the AMEC stability analysis for Section D gives a higher factor of safety than BGC (Table 4.5Table 4.6), by approximately 10%. This is due to the BGC assumptions on geometry and groundwater levels.

If the AMEC analysis is carried over to replicate the geometry in the breach section, the FoS would be approximately 14% lower than their analysis at Section D (Table 4.7). If the BGC analysis is carried over to the breach section, the FoS is similar to their analysis at Section D (Table 4.8). The main conclusion, therefore, is that the AMEC analysis for Section D was not representative of the breach section; the BGC analysis was representative of the breach section.

KCB carried out an analysis based on KCB interpretation of pre-breach geotechnical data and the geometry at Section D. The factors of safety from these analyses (Table 4.10) are lower than AMEC (approximately 13%) and similar to BGC. The KCB analysis indicates that the FoS for seismic loading conditions does not meet CDA criteria.

5 KCB BREACH ASSESSMENT

5.1 Summary of Failure and Geotechnical Investigation

The dam failed between Stations 4+110 and 4+350 at the highest section of the PE shown on Figure 1.4. The breach released 21 million m^3 of water and tailings solids (estimated 50% tailings) which flowed into Hazeltine Creek and thence into Quesnel Lake.

The Perimeter Embankment comprises a rockfill embankment raised in stages by the centreline and modified upstream construction methods, with an upstream “core” of compacted glacial till and filter zones to restrict seepage through the dam. The dam failed during construction of the Stage 9 raise to Elevation 970 m when the dam was 40 m high. At the time of failure, the exterior dam slope was nominally 1.3H:1V, which is the steepest slope that dumped rockfill can typically be placed.

An IRP was formed by the Government of British Columbia shortly after the failure to determine the mechanism of failure of the tailings dam. The IRP’s mandate is given in their Terms of Reference dated October 6, 2014 and their report was published on January 31, 2015 (IRP 2015). KCB has carried out an assessment of the Mechanism of Failure (KCB 2015). Overall, the KCB report agrees with the Panel’s opinion on the basic mechanism of failure of the tailings dam. That basic mechanism was a sliding failure through a lightly over-consolidated glaciolacustrine clay unit (UGLU) in the foundation which dropped the embankment crest enough to allow the pond to overtop and, within a few hours, to completely breach a portion of the Perimeter Embankment.

KCB conducted the majority of the post-failure field investigations and supporting laboratory testing in the immediate breach area. That factual work was published in Progress Report Nos. 1 through 4 which were distributed to MEM and the Mount Polley Mine as they were completed. Data was also made available to the IRP, by the Mount Polley Mine. A report on the Assessment of Failure Mechanism (KCB 2015) was prepared, which included the following additional work:

- review of dam construction history and records;
- interpretation of the morphology of the failed dam and breach;
- review of instrumentation data prior to failure in conjunction with that of new instrumentation installed post-failure; and
- seepage, stability and numerical stress analyses of the dam.

5.2 Mechanism of Failure

It is our opinion that the basic mechanism of failure at the Mount Polley tailings dam was a sliding failure through the lightly overconsolidated glaciolacustrine clay unit (UGLU) in the foundation which dropped the crest enough to allow the pond to overtop and, within a few hours, to completely breach a portion of the PE. This mechanism is manifested by physical evidence of dam displacements and shear movements in the dam foundation, and is supported by back-analyses using the engineering properties of the dam and foundation soils. From all available evidence, the failure was initiated by a combination of the recent excavation at the toe in 2013, and raising of the embankment just days prior to the failure and the steep outer slope of 1.3H:1V.

At the time of failure, the FoS of the dam was calculated using limit equilibrium methods to be 1.27 using the peak drained strength of the UGLU, the pre-failure pore pressures estimated by seepage analyses and the measured pore pressures upstream of the core of the dam. The FoS reduced further to 1.19 with an allowance for construction induced pore pressures.

Numerical stress analyses of the dam show that, at these low FoS, the shear stresses induced in the UGLU below the steep outer dam slope would have exceeded the available peak drained strength, thereby initiating a progressive undrained failure mechanism in the UGLU. Using the peak undrained strength of the UGLU, the calculated FoS of the tailings dam reduces to unity.

Because of the strain-weakening behavior of the UGLU, the displacement of the dam probably accelerated once failure was initiated (FoS less than 1) as described above. This acceleration of movement subjected the UGLU to progressively larger strains and greater strength loss, with calculated FoS ultimately reducing to as low as 0.80 at the fully remolded strength of the UGLU. At this stage, rapid movement of the dam continued until the geometry of the failed mass re-stabilized at a FoS of unity.

The forensic drilling and excavations in the failed dam and breach area identified a distinctive shear plane and down-drop in the upstream till core and up-thrust of the foundation soils at the dam toe. Movements interpreted from these and other features indicate net dam displacements in the order of 5 m to 10 m along the sliding plane in the UGLU. Numerical deformation analysis of the dam by KCB shows that the down-drop of the dam crest during the failure would have been sufficient for the tailings pond water to overtop the crest of the till core and trigger the subsequent dam breach.

Large movements in the UGLU are also consistent with the small movements in the UGLU recorded by inclinometers installed post-failure, the heavily de-structured and folded varves of the UGLU in the failure zone below the dam, and the weakened state of the UGLU in the failure zone consistent with the remolding of the clay during the dam displacements.

A pre-existing shear plane in the UGLU was considered as a possible factor in the failure. The presence of shear planes or other distortions of the varved clay structure was looked for in the samples retrieved from outside the failed dam but none was found. The near-horizontal inclination of the varve bedding in free field samples of the UGLU also tends to rule out an old landslide or glacial shearing as a contributory factor in the failure. The absence of a pre-existing shear plane is corroborated by the fact that the dam probably would have failed earlier if a shear plane at lower shear strength had been present.

5.3 Quaternary Geology Update Based Upon Breach Investigations

The soil stratigraphy was studied by KCB as part of the failure investigation and this information was used to update and confirm the Quaternary geologic model discussed in Section 4.3 of this report. Some key observations from the failure investigation are summarized below:

- The preservation of laminations and layering within the lightly overconsolidated UGLU indicates that the overlying Upper Glacial Till (UGT) was deposited under relatively thin ice, likely less than 50 m in thickness based on the level of overconsolidation measured in free-field (outside of breach area) samples. Conversely, the LGLU appears to have undergone much greater glacial loading as evidenced by its overconsolidation ratio and wavy and distorted laminations that may be indicators of glacial drag.
- The gradation and texture of the UGT are typical of basal till (Levson and Giles 1997). The lower relative strength and higher moisture content of the UGT compared with the underlying MGT, however, indicates that the UGT was deposited under much thinner ice than the MGT. The preservation of structure and light overconsolidation of the underlying UGLU also indicate that the ice that deposited the UGT was relatively thin.
- When the UGLU was not present, the UGT and MGT were often differentiated in the field based on colour. The UGT was generally brown/grey, whereas the MGT was green/grey. This, and the differences in the properties described above, indicates the two units are of different age and origin.
- Organic samples collected from the LGF indicated an age of approximately 34,000 bp.

KCB's interpretation of the history of glaciation within the breach area is summarized below:

4. The LGT was deposited during the penultimate glaciation. Degree of consolidation and high strength of this deposit suggests that it was deposited subglacially under heavy glacial loading.
5. The LGF and UGF sediments were deposited as glacial outwash following the penultimate glaciation and perhaps during the advance of the second glaciation, 34,000 bp.
6. A second glacial advance deposited the basal MGT, and loaded the UGF, LGF and LGLU units.
7. A period of glacial advance and retreat during the second glaciation deposited ice-proximal glaciofluvial and glaciolacustrine sediments, including the UGLU found within the breach area.
8. A final glacial advance overrode the UGLU and deposited the UGT subglacially. The "bowl shaped" bedrock topography in the breach area may have created a stress shadow during the final glacial advance that limited glacial loading of the UGT and UGLU.

5.4 Geotechnical Parameters

Disturbed and undisturbed soil samples were collected during the site investigation to characterize the foundation soil units and dam core. Drill holes were strategically located so that observations of soil structure and differences in soil strength could be assessed within the breach and in the "free field". This section summarizes the strength properties of the soils, as determined by KCB.

5.4.1 Till Units

KCB performed consolidated-undrained triaxial tests on five samples of UGT, in both compression and extension. At representative stresses at the toe of the PE, the UGT showed a dilative response to shearing during the triaxial tests. At higher stresses below the embankment, the UGT was assumed to show a net contractive response and the undrained strength was used at later stages in the conceptual failure model.

Consolidation testing performed by the IRP (2015) indicates that the preconsolidation stress of the UGT is approximately 200 kPa, meaning that the soil is moderately overconsolidated. Liquidity indices measured by KCB, and the soil's dilatant response to shearing in the triaxial tests, are other indicators of the UGT's overconsolidation.

Laboratory strength testing was not performed on the MGT or LGT. Liquidity indices, and one consolidation test on the MGT, suggest that both soil units are heavily overconsolidated and shear wave velocity measurements indicate the soils are very stiff to hard.

Till strength parameters used for the KCB failure assessment are summarized in Table 5.1.

Table 5.1 Summary of Strength Parameters for Failure Analysis – Glacial Till Units (KCB 2015)

Parameter	Upper Glacial Till (UGT)	Middle Glacial Till (MGT)	Lower Glacial Till (LGT)
Peak effective friction angle	35° (free field) 33° (below embankment)	32°	35°
Undrained shear strength (below the embankment)	Su = $0.38\sigma_{vo}'$	-	-

5.4.2 Glaciolacustrine Units

KCB compared the index properties of the two glaciolacustrine units found within the breach area: the ULGU and the LGLU. A summary of the index properties is shown in Table 5.2.

Table 5.2 Summary of Index Properties for UGLU and LGLU (KCB 2015)

Parameter	Upper Glaciolacustrine (UGLU)	Lower Glaciolacustrine (LGLU)
Soil classification	CI-CH	CI
Specific gravity	2.77	-
Gravel content (%)	0 to 4 (2)	0
Sand content (%)	0 to 15 (8)	1 to 4 (3)
Fines content (%)	81 to 100 (90)	96 to 99 (97)
Clay content (%)	39 to 67 (50) [59]	23 to 32 (26) [31]
In situ water content (%)	13 to 54 (36)	15 to 29 (23)
Liquid limit (%)	33 to 69 (50) [61]	31 to 42 (35) [41]
Plastic limit (%)	15 to 26 (20)	11 to 23 (18)
Plasticity index (%)	18 to 49 (30) [39]	11 to 27 (17) [24]
Liquidity index	(0.5)	(0.2)
Activity	(0.6)	(0.6)
XRD clay speciation (Note 3)	47% illite, 29% chlorite, 22% smectite, and 11% kaolinite	43% illite, 27% chlorite, 24% smectite, and 10% kaolinite

Notes:
1. Values presented are minimum and maximum range of tested data. The median of this range is included in brackets.
2. Square brackets shows mean + standard deviation value representing the upper 2/3 bounds of data.
3. Median values for semi-quantitative amount of clay minerals reported for the < 2 microns fraction.

Generally, the UGLU was shown to have a higher moisture content, higher plasticity, higher clay content and higher liquidity index (indicating less overconsolidation) than the LGLU.

Drained peak and residual strength parameters for the UGLU were measured by KCB through direct shear and triaxial compression testing. Consolidation tests indicate a mean preconsolidation pressure of approximately 400 kPa, meaning that under significant portions of the dam the UGLU would have been loaded beyond its preconsolidation pressure and into a normally consolidated state. Accordingly, the undrained strength of the UGLU was assessed under direct simple shear.

Compared to the UGLU, the LGLU is very stiff to hard and overconsolidated. Oedometer testing on one sample of LGLU indicates a preconsolidation pressure in excess of 750 kPa. This means that under the dam the LGLU would have remained in an overconsolidated state and ultimate strength of the soil would be governed by the drained frictional strength. One direct shear test was performed on the LGLU which was understood to represent the "average" drained strength. To approximate the strength of high plasticity, clay rich horizons in the unit, the fully softened and residual drained friction angles were estimated using liquid limit and clay content.

Glaciolacustrine units' strength parameters used for the KCB failure assessment are summarized in Table 5.3.

Table 5.3 Summary of Strength Parameters for Failure Analysis – Glaciolacustrine Units (KCB 2015)

Parameter	Upper Glaciolacustrine (UGLU)	Lower Glaciolacustrine (LGLU)
Peak effective friction angle	22°	28°
Residual effective friction angle	14°	23° (free field) 18° (below embankment)
Undrained shear strength	Su = 50 + 0.13 σ_{vo}' (peak) Su = 36 +0.11 σ_{vo}' (20% strain) Su = 22 +0.03 σ_{vo}' (remolded)	-

1. Unit weight calculated based on median water content, specific gravity, and 100% saturation.

5.4.3 Glaciofluvial Units

The UGF and LGF are mainly differentiated by the organic content, plasticity and fines content. The UGF is dark grey, none to low plasticity, and has a strong organic odour. It also varies from a laminated silt and fine grained sand to well graded sand with some gravel. The LGF is brown, non-plastic and primarily fine grained sand, but can also be present as sand and gravel with no silt to coarse gravel in a silt matrix. Both are dense to very dense.

No advanced laboratory testing was carried out on either unit, however peak friction angles of 30° and 33° were assigned to the UGF and LGF, respectively, based on soil classification and index properties.

5.4.4 Bedrock Units

KCB identified three different bedrock units within the breach area termed sedimentary mudstone, mafic-igneous and volcanic bedrock. The mudstone has very high moisture content and very high plasticity and appears to be localized directly below the breach. Some slickensides and shears were noted in the mudstone unit.

The mafic igneous and volcanic bedrock were primarily encountered as sandy, gravelly silt and silty sand, respectively, and have much lower moisture content and plasticity than the mudstone. CPT tip resistance and shear wave velocities indicate that both of these units are very dense.

The relatively low strength units above the bedrock control embankment stability. Therefore, the bedrock units were not assigned shear strength parameters in KCB stability analysis and were assumed impenetrable.

5.4.5 Till Core

KCB collected two bulk samples of till core material, one at the PE crest and other within the breach area in the remnant core. Samples were compacted to 95% and 92% of the Standard Proctor maximum dry density and tested under triaxial compression. All samples showed net contractive response during shearing and a drained friction angle of 33° was measured. For stability analysis, the drained and undrained strengths were used for the initial and later stages of the conceptual failure model, respectively. For the undrained strength, an S_u/σ_{vo}' ratio of 0.38 was adopted.

5.5 Potential Contributing Factors – Geotechnical Basis

5.5.1 General

An assessment of potential contributing factors was carried out to determine what, if any, role they may have had in reducing the FoS of the dam and potentially contributing to the failure. Wherever possible, the potential influence on the FoS has been assessed with a stability analysis. The following sections provide a discussion on the potential influence that various parameters could have had on stability, recognizing that the mechanism of failure still clearly identifies sliding on the weak UGLU as the failure mechanism. Stability analyses for various conditions are included in the Assessment of Failure Mechanism report (KCB 2015).

5.5.2 Steepness of Downstream Slope

The steepness of the downstream slope played a major role in the initiation of the failure as described in detail in KCB (2015). The over-steepened slope and increased height of the dam fill led to stress concentration below the toe of the dam within the UGLU, which ultimately triggered undrained strength behavior and progressive failure of the dam.

The change in the FoS of the dam with a 2H:1V downstream slope (versus the 1.3H:1V slope at the time of failure) would have increased the FoS by approximately 25% and the stress concentrations would also have been diminished.

5.5.3 Buttress Excavation

Foundation preparation for the PE in 2013 included excavation of soft soils in the portion of the dam foundation in which the buttress fill was planned to be placed in the last half of 2014. Excavation included removal of up to approximately 2 m of soft materials in November, 2014. Excavation was directed by MPMC without AMEC present and when the AMEC Support Engineer arrived a snowfall prevented inspection and approval. A ground foundation survey after excavation, if available, was not provided to MEM. In 2014, the excavation filled with seepage water and was, therefore, not inspected or backfilled. The 2014 construction plan was to dewater the area just prior to placement of the buttress fill. At that time the foundation would have been approved by the design consultant and surveyed. Estimates of the potential depth of excavation considered the interviews summarized in Table 2.1. In addition, shallow drilling carried out in the area in 1999 indicated an average depth of 1.5 m of soft soils (DH-99- 115, 116, 118, 119). The stability of the breach section was analyzed (KCB 2015) to assess the change in FoS that a 2 m excavation would have, which indicated a reduction of FoS of approximately 5% in the peak undrained shear strength case. Given the uncertainty in the actual depth of excavation and the lateral width of the excavation, the probable range of influence could vary.

5.5.4 Pond Water Level and Phreatic Surface Upstream of the Core

The original water balance for the TSF (KP 1995c) developed for monthly average conditions showed in year 14 of mine operations there would be a predicted (95 percentile upper estimate) surplus of water of 2.9 Mm^3. In May 2014, nearly 20 years after the mine started, the pond volume was estimated to be between 8 Mm^3 and 9 Mm^3, with the site needing to discharge approximately 3 Mm^3 per year, year round (BGC 2014, pg. 22). At the time of the failure, the pond level was approximately 3.2 m below the till core of the dam in the area of the breach and the pond surface extended to approximately 50 m upstream of the core zone, with the pond depth increasing away from the dam towards the reclaim water pump barge.

The steady state pore water pressures acting on the dam are influenced by the pond level and the effectiveness of the upstream toe drain. The upstream toe drain (perforated plastic pipe encapsulated in sand and gravel) was placed at approximately El. 946.3 m and extended near horizontally northward to Station 4+575 m, where it exited in a solid pipe under the dam and discharged into the downstream rockfill. The drain water then flowed by gravity to the perimeter sump pond. Piezometers located upstream of the core, in the area of the breach, indicated that the drain was effective and lowered the piezometric pressures upstream and acting on the core of the dam.

Stability analyses carried out (KCB 2015) indicate that the upstream toe drain reduces the piezometric pressures in the tailings upstream of the core which reduces seepage gradients in the foundation below the core. Analysis showed that if the upstream drain was not functional, piezometric levels upstream of the core would increase to near-hydrostatic and the FoS would have been reduced by approximately 12%.

5.5.5 Artesian Pressures

Steady state seepage analyses (KCB 2015) indicate that artesian pressures in the confined glaciofluvial layer located below the UGLU were calculated to be on the order of 6 m to 8 m above original ground surface. The effect of not including the artesian pressures would increase the FoS by approximately 8% as shown in Table 5.4.

Table 5.4 Stability Analysis Difference without Artesian Pressure

UGLU Strength Model	Factor of Safety		Difference in FoS
	Using Seep/W model input	Without Artesian Pressure	
Stage 1 – Peak Drained	1.27	1.37	+8%
Stage 2 – Peak Undrained	1.02	1.09	+7%

5.5.6 Rate of Material Placement – Excess Pore Pressure

Approximately 3.0 m of construction fill was placed on the top of the dam in the area of the dam breach during Stage 9A construction (May, 2014 to August 3, 2014). The last 1 m thick lift of Zone U, mine rockfill was placed July 29 to July 31, 2014.

Piezometers were not located in the foundations soils in the breach area to measure the pore pressure response due to loading. Consequently, an estimate of the potential pore pressure response was made to determine the expected pore pressure increase due to loading. The stability analysis indicated that incorporating the increase in pore pressure would reduce the factor of safety by approximately 6% (KCB 2015).

5.5.7 Piping and Cracking

Timing and Water Flows

MEM interviewed the MPMC Electrician, [redacted], on September 12, 2014 and questioned him on the timing of the events. [redacted] also provided a print out of the pump instrumentation record that documents the times. Figure 5.1 is a printout of the instrumented pump level record, along with observations provided from the MEM interview. KCB have estimated the flows based on the approximate pump capacity and storage capacity of the PE SCP.

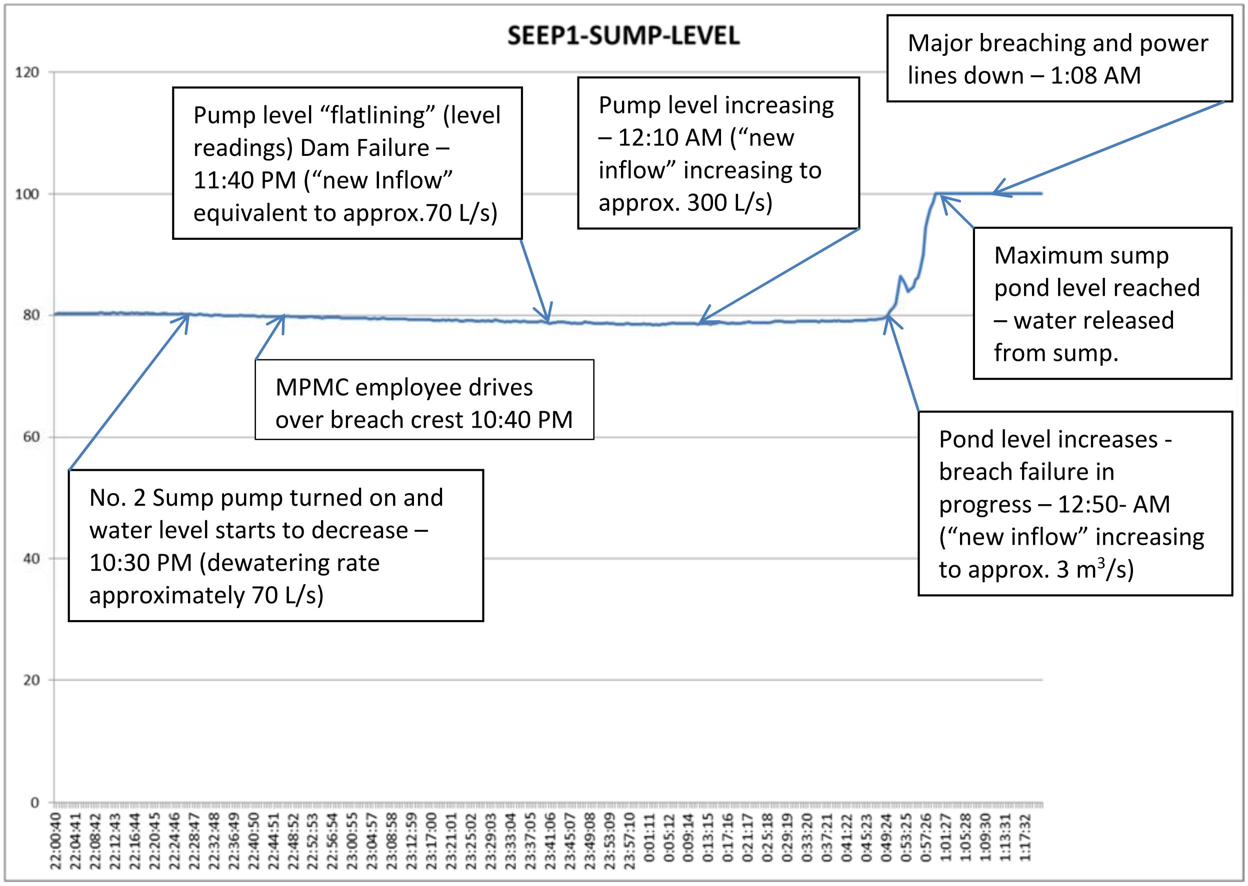


Figure 5.1 Perimeter Sump Water Level Record and Breach Observations

IRP (2015) had a different time sequence than presented in Figure 5.1 and the main discrepancies in the Panel version and the MEM version is that the Panel's time for the pump being turned on and the employee driving past the dam is approximately 1 hour later; i.e. the MPMC electrician turned on the pump at 10:30 PM and drove across the dam at approximately 10:40 PM.

The key point in the time comparison is that the MPMC employee drove across the dam prior to a levelling off and increase in the sump pump level, hence prior to the dam failure that triggered the increase in flow.

Review of Piping

Evidence of potential piping was observed in the left bank of the breach failure and this has been discussed by the IRP (2015), which also identified observations of washing of some fines in the filter. In addition, comparison of QC gradation data against the technical specifications indicates that the filter material was frequently coarser than specified.

To assess the potential effect of piping and internal erosion on dam stability, KCB assessed the PE stability with the following assumptions:

- The "base case" seepage model for Section C was adopted from the failure mechanism report (KCB, 2015) which assumes that the upstream toe drain is functioning.
- A hypothetical 0.5 m thick "zone of piping" was added to the model at El. 941 m that penetrates Zone S, Zone F and Zone T, connecting the upstream cycloned sand zone with the Zone C rockfill shell. The piping zone is located below the upstream drain where seepage gradients across the core are high.
- The same saturated hydraulic conductivity (5×10^{-2} m/s) and unsaturated conductivity functions used for Zone C were used for the piping zone.

Results show that the phreatic surface in Zone C does not become elevated due to a hypothetical piping and stays within the Zone T drainage blanket. However, the flux at the toe of the dam increases by over 1 order of magnitude, from 3×10^{-7} $m^3/s/m$ to 7×10^{-6} $m^3/s/m$, which equates to a seepage rate of <1 L/s over the 200 m nominal width of the dam failure.

The analysis also shows that the drainage capacity of the rockfill can easily accommodate an increase in flow from an assumed defect in the core, because the flow into the defect is "throttled" by the upstream tailings. Therefore, the stability of the embankment with respect to foundation failure would not have been affected by piping caused by internal erosion of the core.

If a major pipe had developed immediately preceding failure that connected the downstream shell of the dam to the pond, large scale erosion of the embankment face leading to undermining and regression of the exterior embankment slope would have been expected. This type of failure mechanism is at odds with the subsurface and morphological evidence of the deep failure through the foundation. Also, large flows would have been easily observed emanating from the toe and recorded in the Sump Pump levels.

Observations of the remaining core zones indicate that the core material is very dense and competent. One sub-horizontal, thin tailings filled crack was observed within the core but the crack did not penetrate the full width of the core. Further discussion is provided in KCB (2015).

5.5.8 Foundation Soils

Natural ground conditions in the area of the breach included the presence of low strength GLU soils near surface (Figure 4.3). The near-surface GLU was reported in the site construction records as having been removed from the footprint of the dam. However, construction records also indicate that the dam fills that were being placed over the prepared foundation surface were "pumping", e.g. wavy response to construction traffic due to lower strength foundation soils and buildup of pore pressures due to the construction equipment. This suggests that portions of the foundation in the area of the breach may have some occurrences of lower strength and/or overwet soil or that the near-surface GLU was not completely excavated. The site investigation for the breach encountered unexcavated silty clay at the embankment foundation contact in only 2 of 9 drill sites within the footprint of the failed dam. This is consistent with the clay being excavated in most areas under the embankment. Furthermore, the geomorphology of the failed dam does not support a shallow sliding failure mode on near-surface GLU.

The upper glacial tills present in the dam foundation include: upper till, middle till and lower till. The failure surface crossed through the upper till and within the UGLU. As such the strength of the upper till would influence the factor of safety. The upper till is dense, however it is not as consolidated as the middle and lower tills.

5.5.9 Other Potential Factors

Convex corner at breach

The breach area is adjacent to a 25° bend in the dam alignment, with the center of the bend being at Station 4+330 m. The left abutment of the breach is at Station 4+350. It is a common misconception that the FoS is lower within a bend in the dam, versus a straight section. In fact the bend in the dam actually results in reduced loading forces and increased resisting forces, resulting in slightly higher FoS.

In the event that a failure is initiated adjacent to the corner, the resisting side forces, in a 3-D analysis, would tend to reduce. A convex corner can also introduce tensile stresses in the dam, which could potentially lead to cracking of the core. There is no evidence to suggest that these two effects occurred or influenced the failure.

Seismic or rock blast vibration

There were no reported incidents of seismic activity leading up to or during the breach event. Blasting of mine rock would have been routinely carried out in the open pits and underground workings. However, the closest distance of any potential blast is 5 km, which is more than enough distance to attenuate the blast vibrations, such that they would not be measureable at the dam.

Drainage pipes through dam foundation

Drainage pipes do not pass through the foundation in the area of the breach and the nearest foundation drainage pipe is located several hundred meters west of the breach.

6 MPMC TAILINGS MANAGEMENT PRACTICES

6.1 Organizational Structure

MPMC management personnel for the TSF have evolved and changed over the life of the project. Typically, there has been a manager assigned for the TSF each year and that person ultimately reports to the General Manager. A summary of key personnel over the life of the mine is provided in Table 6.1.

Table 6.1 Summary of MPMC Key Personnel for the TSF

Timing	General Manager	Tailings Manager	Comments
1996 - 2000	Bryan Kynoch	Don Parsons	
2001-2005	Howard Bradley (mine manager)	Howard Bradley	Care and maintenance period. 2004 OMS manual shows Howard Bradley maintaining overall responsibility
2006-2012	Tim Fisch	Ron Martel (environmental technician)	
2012-2013	Tim Fisch	Luke Moger (eligible for P.Eng.)	
2014	Dale Reimer	Luke Moger (eligible for P.Eng.)	Nicholas Bergeron (EIT) was starting to take on some of the Tailings Manager's responsibilities

The management structure consists of both internal (MPMC) and external consultants. The primary reporting structure for MPMC at the time of the failure is illustrated in Figure 6.1. The management structure for operations, which includes both external consultants and other mine departments are presented on Figure 6.1 (extracted from the OMS manual, Figure 2-1, and extended to include Corporate interview observations).

The key roles are summarized as follows:

- Imperial Metals, President, Bryan Kynoch– Ultimate responsibility for all activities carried out by MPMC.
- Imperial Metals, Chief Operating Officer, Don Parsons – Overall corporate responsibility for tailings management (MPMC 2013d).
- MPMC, General Manager, Dale Reimer – Responsible for the overall activities of the mine, including the TSF. Approves the annual OMS manual (MPMC 2013d).
- MPMC, Tailings Project Manager, Luke Moger – Responsible for planning, co-ordination and daily management of all construction activities. Responsible for permitting and the annual reports. Water Management Group reports through Luke Moger for the OMS manual.
- MPMC, Water Management Leadership Group – Senior personnel including: Mine Operations Manager (Art Frye), Mill Operations Manager (Doug Ablett), Environmental Coordinator (Colleen Hughes) and the Tailings Manager.

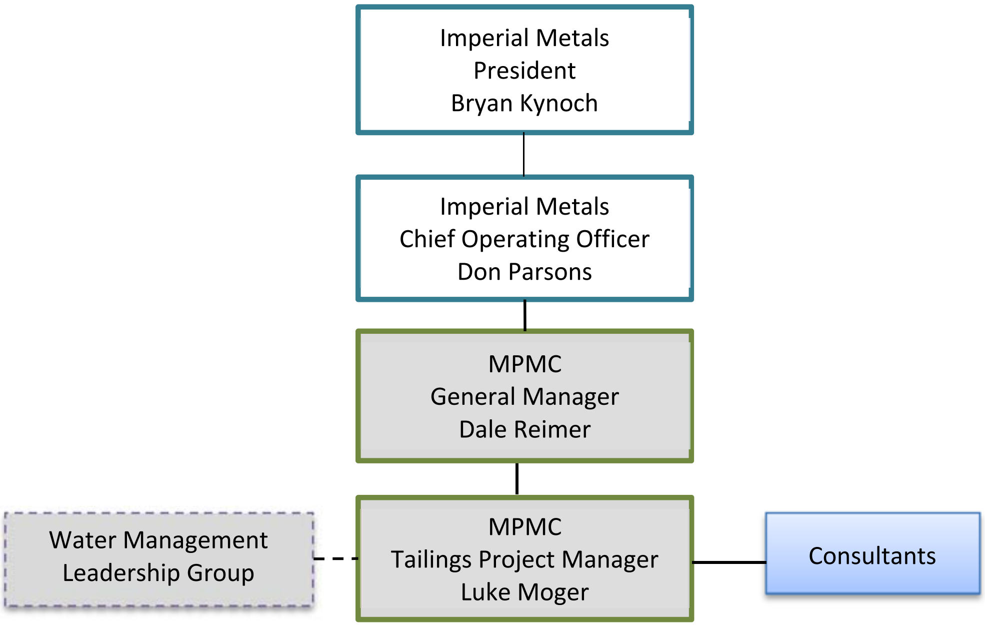


Figure 6.1 Corporate and MPMC Tailings Management – Organization Chart at Time of Failure

The organization chart shown on Figure 6.2 presents the reporting structure related specifically to the TSF. The chart is based on the 2013 OMS manual, which has not been updated and, consequently, does not include staff changes such as the new mine manager.

Overlying this reporting structure is the MPMC operating reporting structure, which is presented on Figure 6.3. The chart was provided to CIM [redacted] and includes the new mine manager, however it has not been updated with other changes in personnel.

The two organization structures did appear to create some confusion with respect to who was managing the TSF. For example, Art Frye is not included in the TSF Management Chart, although because Luke Moger reports to Art Frye it was sometimes assumed that Art Frye was responsible for tailings management [redacted].

The Tailings Manager, Luke Moger, also used the title "Project Engineer" in both the 2013 OMS Manual and the 2013 Tailings Management Framework (Under TSM Protocols), although he is not a Professional Engineer or an Engineer in Training.

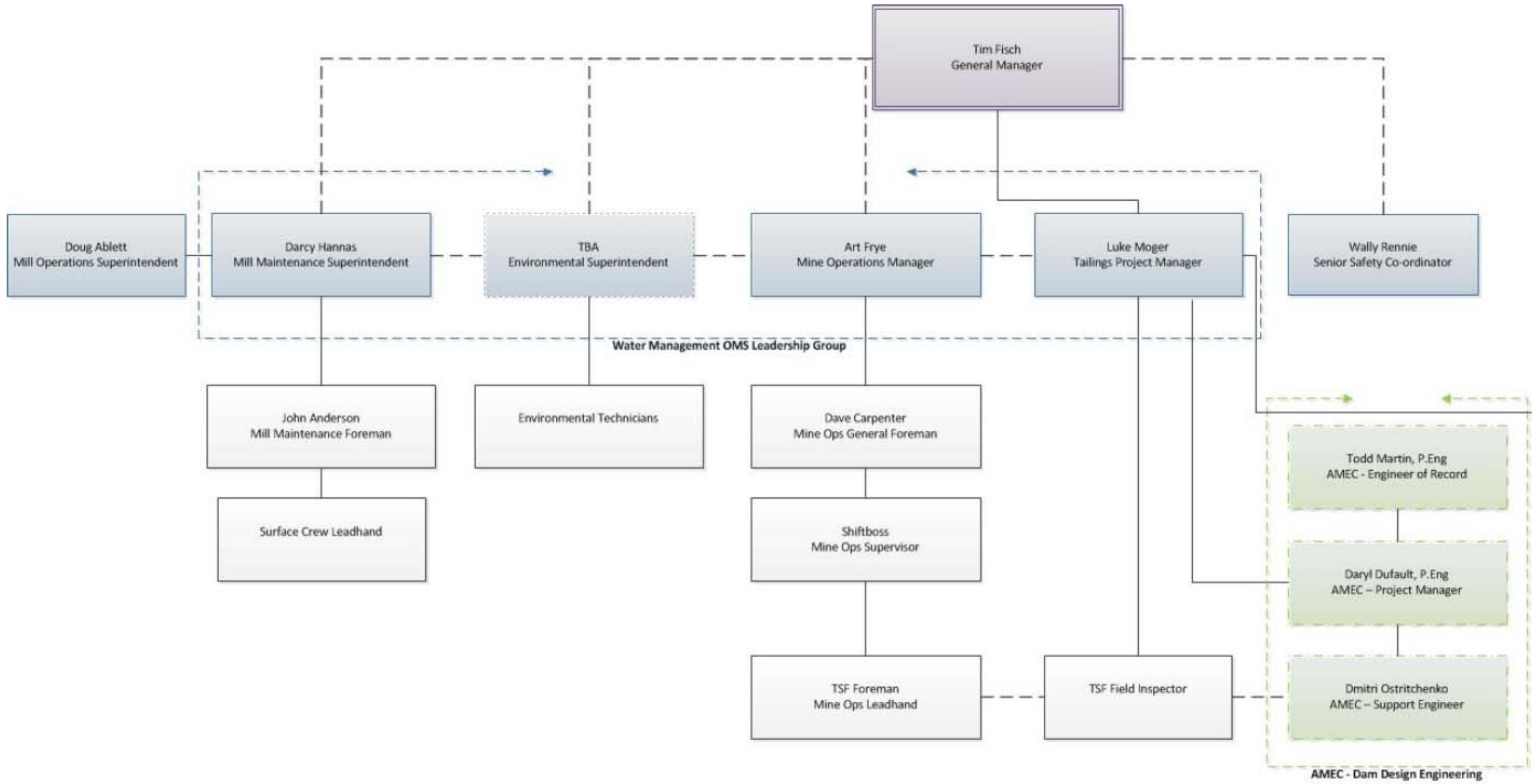


Figure 2.1 – Personnel Organizational Chart

Figure 6.2 Organization Chart – MPMC Responsibility Positions for the TSF According to 2013 OMS Manual (MPMC 2013b)

[redacted]

Figure 6.3 [redacted]

6.2 Compliance with MAC Guidelines

6.2.1 General

MPMC is member of the Mining Association of Canada (MAC), which has developed the following series of guidelines for tailings management stewardship:

- A Guide to the Management of Tailings Facilities (MAC 2011a).
- Developing an Operation, Maintenance and Surveillance Manual (MAC 2011b).
- A Guide to Audit and assessment of Tailings Facility Management (MAC 2011c).
- Towards Sustainable Mining (TSM) Assessment Protocol (MAC 2011d).

MPMC started in 2012 to incorporate the MAC Guidelines into their tailings management practices. An OMS Manual was initially developed in 2004 and generally updated annually, with the last version dated July 01, 2013 (unsigned). The OMS Manual is one component of a Tailings Management System.

MAC has developed a guideline for audit and assessment of the Tailings Management System. There has not been a formal audit or assessment of MPMC's tailings facility. However, as part of MPMC's requirement to meet the TSM protocols, MPMC initiated the TSM process in 2012 (D. Parsons- interview) and carried out an internal "self-assessment" at the end of 2012 (T. Fisch - interview) and were planning to carry out another one in 2014. A copy of the "Tailings Management Framework (Under TSF Protocols) (MPMC 2013d) which was prepared and signed by Don Parsons (COO, Imperial Metals) and Tim Fisch (General Manager) on February 5, 2013, was available for this review. CIM assumes that this document was sent to MAC as part of the internal audit procedure.

6.2.2 MPMC Tailings Management Framework

The MAC Guide to the Management of Tailings Facilities includes the main components that are shown schematically on Figure 6.4. The framework follows the life cycle of the mine and starts with policy and commitment, leading into planning and implementation, followed by checking and corrective actions, leading to continued improvement and confirmation of meeting the policy and commitment.

*A **tailings facility** includes the collective structures, components and equipment pertaining to tailings impoundment and management, including dams and reservoirs, other related facilities and appurtenances. (MAC 2011a).*

***Tailings Management System:** a documented set of processes and practices that enable an organization to manage its tailings safely and environmentally responsibly while increasing its efficiency. (MAC 2011a)*

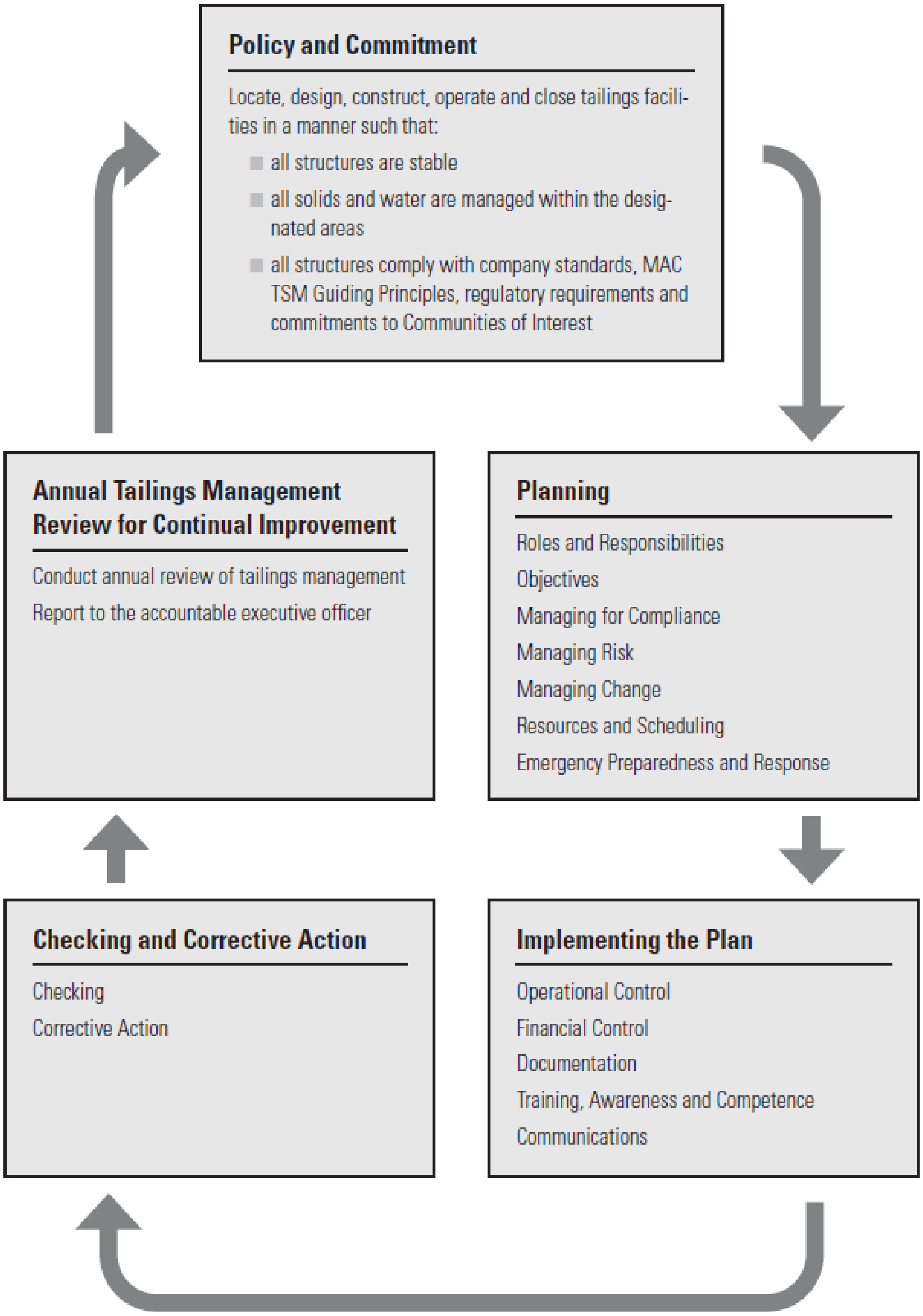


Figure 6.4 MAC Guideline – Tailings Management Framework (MAC 2011a)

MPMC was not able to provide to MEM a copy of a documented Tailings Management System and, consequently, MPMC documentation of the tailings management practices appeared to rely on the following:

- Operation Maintenance and Surveillance (OMS) Manual;
- Construction Manual (Consultant);
- As-constructed record (Consultant); and
- Annual Dam Safety Inspection Report (Consultant).
- Partial completion of the "Tailings Management Framework (Under TSM Protocols) (MPMC 2013d) – Refer to Section 6.2.5 of this report.

6.2.3 Operations, Maintenance and Surveillance Manual

MPMC has an Operations, Maintenance and Surveillance (OMS) manual for the TSF, the most recent of which was issued in 2013 (MPMC 2013b). The MPMC OMS follows the general guidance of the Mining Association of Canada (MAC 2011b). The main components of the OMS and relevant observations are summarized in Table 6.2.

Table 6.2 Review of 2013 MPMC Operation, Maintenance and Surveillance Manual

Chapter	Components	Addressed in OMS?	Observations
Roles and Responsibilities	Organization, Structure, Individual Responsibilities	Y	MPMC responsibilities for Tailings Project Manager relate only to construction, although a planning and coordinating role is articulated for the OMS manual. The organization chart does not align with the table of key personnel. Todd Martin and Daryl Dufault are listed in the organization chart as the Engineer of Record and Project Manager, respectively. However the key personnel table shows Laura Wiebe as the Engineer of Record and Project Manager. [redacted] Engineer of Record responsibilities are not defined.
	Competency and Training	Y	Short training period for MPMC Inspectors. Luke Moger's signature title is "Project Engineer"; [redacted]
	Managing Change	Y	Responsibility of the Tailings Project Manager to ensure OMS is updated to reflect changes. OMS manual to be updated at least annually (last annual revision was July 01, 2013), and as required with updates to process or regulatory changes.
Facility Description	Facility Overview	Y	Facility overview does not include any context for the TSF and the dams (e.g., area, length, height, materials, raising, etc.).
	Site Conditions	Y	Site conditions section does not include any discussion about foundation soils and the presence of weaker glaciolacustrine layers.

Chapter	Components	Addressed in OMS?	Observations
	Facility Components	Y	All components are identified.
	Regulatory Requirements	Y	Requirements are not documented in one section.
	Basis of Design and Design Criteria	N	Not clearly documented in one section. Design reports referenced. Design criterion for freeboard is incorrectly referenced.
	Construction History	Y	Documentation of construction history reliance on reference to past construction reports.
	Document Control	N	Documentation incomplete and relies on reference to a few construction reports. There is no consolidated geotechnical data report or water balance basis report. The OMS manual was not kept updated. Communication and meeting records were incomplete.
Operations	Objective	N	Missing the Operations Flowchart (MAC).
	Tailings Transport and Deposition	Y	Correctly articulates "fundamental objective to establish beaches…."
	Dam and Basin Raising	Y	General discussion, however there is no longer term (e.g. >1 year) plan for tailings and water management.
	Water Management	Y	Freeboard management plan is not complete and was not followed during the May 2014 overtopping event.
	Environmental Protection	Y	Permit requirements for water quality and discharges included.
	Safety and Security	N	Health and safety plans are not included.
	Documentation	N	Relies on consultant Annual Reports (Dam Safety Inspection and As-Constructed Reports).
Maintenance	Objective	Y	General summary.
	Maintenance Parameters	Y	General, water management, tailings pond and embankment, discharge and reclaim pipelines.
	Routing and Predictive Maintenance	Y	Mill operations have a daily checklist.
	Event Driven Maintenance	Y	Described in the "Unusual Events Action Log."
	Documentation	Y	Inspection logs and daily checklists.
	Reporting	Y	
Surveillance	Objective	Y	General summary – missing Surveillance Flowchart (MAC).
	Surveillance Parameters	N	Inspection and surveillance log used.
	Surveillance Procedures	Y	Dam surveillance minimum quarterly. Loss of freeboard in 2014 illustrates lack of surveillance.
	Visual Monitoring	Y	Dam surveillance minimum quarterly.
	Instrument Measurement	Y	Instrumentation trigger levels provided.
	Collation and Analysis of Data	Y	Instrumentation data sent to consultant.
	Periodic Inspection and Review	Y	Annual report by consultant; "Unusual incident log"; Dam Safety Reviews.
	Documentation	Y	Annual reports by consultant.
Emergency Planning and Response		Y	Appendix E not available for review. Emergency procedures are to be implemented if TSF freeboard reaches 1.3 m (noted in text but not described in this section). Not carried out in 2014 when freeboard was reduced to zero.

The main observations with respect to the OMS manual are:

- The document follows the general MAC Guidelines, however, it does not address all of the subsections and components of the OMS, as noted in Table 6.2.
- The roles and responsibilities do not name a person responsible for all aspects of the Tailings Management System. The designated Tailings Project Manager is only responsible for construction management and consultant coordination. The knowledge, skills and accountability of the Tailings Manager are not clearly defined.
- There is no overall description of the dam foundation soils and the reasons for monitoring of the dam.
- While there is a section describing the change management process, the OMS manual had not been updated with onsite and regulatory changes (e.g., M-200Amd Permit Aug 15 2011 listed as a reference). The 2013 OMS document had not been "signed off" by the Tailings Manager and the Mines Manager.
- There is an overreliance on the consultants Annual Reports to address documentation requirements.

6.2.4 Audit and Assessment of Tailings Facility Management

The MAC guidelines provide a framework for audit and assessment of the tailing management system (MAC 2011c). Table 6.3 provides a summary of the MAC guideline components and observations made on the various components in relation to MPMC's TSF.

Table 6.3 Review of MPMC Tailings Management

MAC Guideline Component		MPMC Management Action	Observations
Policy and Commitment		Elements of a Tailings Management Framework were developed.	Checklists were used to identify the elements A documented "Policy and Commitment" statement was not available.
Planning	Roles and Responsibilities	Roles and responsibilities are defined in the OMS and in the Annual Construction Manual.	Roles and responsibilities of MPMC staff are not clearly defined. "Tailings Project Manager" position responsibilities were not clearly understood by MPMC staff and Corporate Officers and the designation of Tailings Engineer was inappropriately used on communications and documents.
	Objective	Comprehensive documentation of tailings facility plans not available.	Documentation limited to consultant Reports and OMS.
	Managing for Compliance	Reporting to MEM and MOE on water quality, as-constructed documents, annual dam safety inspections.	Out of compliance with the loss of freeboard and water overtopping in May 2014. Complies with submission of environmental reports, annual dam safety reports and as-constructed reports.
	Managing Risk	Formal risk assessment has not been carried out.	Risk assessment was not carried out and a risk management plan is not in place.
	Managing Change	Brief change management section outlined in OMS manual.	There were five different EORs in five years. The EOR's responsibilities were not clearly defined, particularly for EORs with limited participation. The OMS was not updated with the most current information and current personnel.

MAC Guideline Component		MPMC Management Action	Observations
	Resources and Scheduling	Resources in place for oversight, construction and consultant design / construction inclusion.	Concern with rock availability for construction – this was exacerbated with the lack of short term water balance planning to align dam fill placement with the rate of rise of the dam. Specialized skill development not present in all areas. Lack of integration with other departments (i.e., environmental team managing the water balance).
Implementing the Plan	Emergency Preparedness and Response	Risk assessment not carried out. Emergency Procedures are documented in the OMS.	Procedures are not well defined and do not include stakeholders. Notification procedures are not adequate for a dam breach.
	Operational Control	Controls in place to inspect, monitor, and instrument geotechnical parameters.	No controls in place for monitoring of water levels as demonstrated in the 2014 overtopping event.
	Financial Control	Financial tracking and budget control of TSF appears to be in place.	Not reviewed – supported by interview comments that annual budgets for the TSF were submitted as part of the annual budget approval.
	Documentation	Documentation control appears to be limited to consultant Reports, OMS and TSF self-assessment.	There is no documentation of the Tailings Management System, which comprises all the elements outlined in the Guide to Management of Tailings Facilities.
	Training, Awareness and Competency	No formal training plan in place. Training appears to be limited to Health and Safety and short technical training sessions with the consultant.	Concern with training and competency level of MPMC and consultant staffing.
	Communication	Communications via weekly operation meetings, short and long term planning meetings in place.	Documentation of meetings was incomplete. Communication of roles and responsibilities was incomplete.
Checking and Corrective Action		No formal plan in place.	
Annual Management Review for Continual Improvement	Annual Review	Includes the Annual Dam Safety Inspection Report.	Self-assessment for TSM was being carried out.
	Report to Executive Officer	Not aware of any formal reporting.	

The key observations with respect to the overall tailings management system are:

- There is no documented Tailings Management System that could be audited or assessed. The Tailings Management System relies on a number of documents, such as the OMS and various consultant reports, which together are assumed to make-up a tailings management framework.
- Corporate officers and the Mine Manager had limited knowledge of the Tailings Management System and were unclear on the responsibilities of the assigned personnel (See Interview

Section 2.4). The roles and responsibilities of staff were not defined to cover all aspects of tailings dam safety.

- The MPMC Tailings Project Manager's stated responsibilities only cover construction management of the TSF and coordination of the Water Management Group. There is no person mandated with responsibility for all aspects of the Tailings Management System. There is no definition of the responsibilities of the EOR.
- A formal risk assessment has not been carried out and there is no risk management plan for the TSF. The main risk management activity is limited to instrumentation trigger levels outlined in the OMS. Risk assessment outcomes should be fully integrated with change management processes.
- The MAC checklist observations include limited quantification of responsibilities, performance measures and schedules for implementation of the tailings management system components.

6.2.5 Towards Sustainable Mining (TSM) Assessment Protocol

"Launched in 2004, Towards Sustainable Mining (TSM) is an initiative of MAC designed to enhance the industry's reputation by improving its performance. MAC members subscribe to TSM guiding principles, a set of commitments that addresses all areas of our industry's performance." (MAC 2011d).

MAC has developed a guideline "A tool for Assessing Tailings Management Performance" that provides a framework for companies to evaluate their tailings management against TSM indicators. These indicators include:

- tailings management policy and commitment;
- tailings management system;
- assigned accountability and responsibility for tailings management;
- annual tailings management review; and
- operation, maintenance and surveillance (OMS) manual

The performance indicators are used, along with a prescribed performance indicators to assess the performance on a five level scale grading (C, B, A, AA, AAA), with the goal of MAC to have all facilities at a minimum A level. MPMC were in the third year of their initiation of the TSM initiative and had carried out one "self-assessment", in which they rated themselves a level A. The "self-assessment" was submitted to MAC, but was not provided to MEM. However a copy of the "Tailings Management Framework (Under TSF Protocols) (MPMC 2013d) which was prepared and signed by Don Parsons (COO, Imperial Metals) and Tim Fisch (General Manager) on February 5, 2013, was available for this review.

To receive level A certification for tailings management a company must have a formal tailings management policy, have a developed and implemented tailings management system, assign accountability for tailings management to the CEO or COO of the company, and have operations, maintenance and surveillance manuals developed for each tailings impoundment. In addition, facilities must conduct annual reviews of all their tailings management systems and report their findings to the accountable executive. While the TSM framework provides a process to check that a management system is in place, it does not necessarily indicate the level of implementation and/or effectiveness of that system onsite.

MPMC's "self-assessment" of the system is understood to consist of filling out the checklists provided in the MAC Tailings Management Guideline (MAC 2011a). While MPMC had implemented many of the items identified, the checklists were incomplete and did not contain quantification of performance measures and almost all Responsibility assignments were to the General Manager. An important omission in carrying out the TSM assessment (MPMC 2013d) was that the section on "Site Selection and Design" was deemed to be not applicable as the site had already been selected. However "Design" was continually being carried out and this section played an important role in the management of the facility.

There has been no external audit or assessment of the tailings management system and development of the system, and internal assessment is still in progress as part of the TSM initiative.

Based on our review of MPMC operations described in the previous sections, MPMC is likely at the C for most of the indicators and, accordingly, appeared to be "working towards improving their tailings management system".

6.3 MPMC Practices Relevant to the Failure

6.3.1 Tailings Facility Management

Management of the TSF evolved over the life of the mine and was influenced by a number of factors relating to the overall Tailings Management System.

For the period of 2006 to January 2012, the TSF was managed by Mr. Ron Martel, Environmental Technician. Mr. Martel was actively involved in all aspects of the TSF, including construction, consultant overview, water balance, environmental reporting and regulatory reporting. Mr. Martel [redacted] and TSF responsibilities were transferred to Luke Moger and the Tailings Water Management Group. [redacted] resulted in a loss of MPMC corporate knowledge with respect to understanding the TSF management and associated water management.

Luke Moger was appointed in 2012 as the Tailings Project Manager responsible for all aspects of construction of the dam. The Water Management Group was responsible for site wide water management and integration of that into the TSF management. However, the "definition" of responsibilities were not clearly documented in the OMS manual and resulted in there not being "one person" responsible for the TSF who also had the training and expertise to fulfill the requirement of that position. Indicators of this include:

- The Environment Group was responsible for water balance, however this was not adequately integrated into TSF planning. Consequently, the Water Management Group did not fulfill its assumed mandate of management of water, as demonstrated by the dam overtopping in May 2014 and the delays in planning the necessary dam raises.
- The Mine Manager was not fully aware of all of the activities of the TSF or its management.
- The Tailings Managers responsibilities were defined as being for the construction of the dams.

Imperial Metals became a member of MAC in 2012 was in the early stages of implementing the TSM initiative, a major component of which includes commitments to ensure the safe operation and management of tailings. As discussed in Section 6.2.5, the TSM initiative was not complete.

6.3.2 Water Balance and Water Management

MPMC recognized in 2004 that a discharge permit would be required to manage surplus water. Interim measures to reduce surplus water requirements were initiated with the installation of evaporators. A Bio-treatment facility was also installed to provide treatment of seepage water.

MPMC initiated a study of site specific water quality objectives to allow discharge of water into Polley Lake. However, MPMC were not permitted for site specific water quality objectives. MPMC were permitted to discharge up to 1.3 Mm^3/year, depending on the water quality, however, only the seepage water met this criteria resulting in approximately 100,000 m^3/yr being released. Consequently, surplus water from the TSF needed to be stored until another discharge plan was developed

MPMC indicated that they had “ordered” a Reverse Osmosis water treatment system with a treatment capacity of 3 Mm^3/year that was scheduled to be implemented in late 2014 or early 2015.

The expanding mine footprint, with the development of Wight Pit and expansion of other pits, increased the volumes of mine contact water with time resulting in typical annual surplus volumes on the order of 2 Mm^3 (2013).

MPMC did not carry out adequate long term planning of the water balance to predict annual volumes of surplus water. MPMC were focused on meeting minimum annual requirement, although even this was exceeded with the dam overtopping in May 2014. The methodology that MPMC adopted for determining annual water storage requirements assumed the ability to predict water inflows in freshet (based on snowpack) and timely dam construction/raises, neither of which turned out to be reliable parameters.

MPMC management of the consultant and the MPMC Water Management Group were not adequate to define and address the prediction of predicted water volumes and to develop timely environmental management plans for the surplus water.

6.3.3 Beach Construction

The requirement for beach (Zone U) was defined by the consultants as being necessary to provide adequate support for the core of the dam. The beach width was not necessarily predicated on the requirement to maintain the free water pond away from the dam crest, but all consultants agree that this is a desirable outcome. The lack of adequate tailings beach for construction of Zone U was recognized as early as 1997 and borrow material was used to construct this zone for the ME. As the impoundment grew over time, MPMC was able to deliver tailings to develop a tailings beach, however this was limited by:

- As the dam elevation became higher, MPMC did not have adequate tailings pumping capacity to deliver tailings to the ME.
- MPMC did not have a tailings deposition plan that was integrated with the water balance, which would likely have demonstrated that construction of a beach was not practical as the increasing water inflow volumes would "over-ride" the beach.

MPMC initiated studies to assess how a beach could be established and/or how material could be used to construct Zone U, these studies and practices included:

- The use of cyclone sand was assessed in 2007, which demonstrated that cyclone sand could be produced for Zone U and possibly for the downstream shell zone of the dam. It appears that this option was not pursued due to the limitation of the pumping capacity to provide adequate hydraulic head to produce cyclone sand, and cost.
- Construction of sand cells was trialled in 2009 and implemented in 2011. This practice was reasonably productive and implemented. However, the construction of sand cells was constrained by:
    - Variability in the ore being milled, which resulted in "system upsets" due to high fines (silts and clays) in the tailings, which reduced productivity.
    - Delivery of tailings to the ME was still not possible due to pumping limitations.
    - The increasing pond water level, due to an increasing water balance surplus resulted in intermittent flooding of the sand cells and increased the requirements on materials required for the sand cell.

Up to the time of failure, rockfill and sand cell construction continued to be used to augment the Zone U requirements.

6.3.4 Mine Planning and Rockfill Supply for the TSF

Long term planning for supply of rockfill to the TSF was not integrated into the long term plan for dam raising (determined by the water balance). The delivery of mine rockfill required the construction of haul roads to the TSF and planning of pit development to meet the dam fill requirements. The upsets in the water balance, as shown with the dam overtopping in May 2014, illustrate that the planning of dam raises was not adequate. The main mine haul road to the toe of the dam was under construction at the time of the breach. The haul road was to be used to provide rock for placement of the buttress fills around the toe of the PE and ME.

7 CONSULTANTS PRACTICES

7.1 Roles and Responsibilities

Consultant Summary

The consultants responsible for the dam design and construction have changed over time, and accordingly, the persons responsible for various stages of the dam have also varied with time. In addition, different consultants may have a different reporting structure or use different terms to describe key roles.

Table 7.1 provides a summary of the key personnel that were involved with the TSF over the period of 1996 to the time of the breach.

Table 7.1 Summary of Consultant Key Personnel over Life of Mine

Position	KP 1996-2010		AMEC Environment and Infrastructure				BGC	
		2006 (DSR)	2011	2012	2013	2014	2013 (2012 - Annual Inspection Report)	2014 (Dam Raise Report)
Senior Geotechnical Engineer	Ken Brouwer		Todd Martin	Daryl Dufault		Andrew Witte	Todd Martin	Todd Martin
Review Engineer	varies	Todd Martin		Steve Rice	Steve Rice	Steve Rice	Tom Harper	Tom Harper
Project Manager	varies	Michael Davies	Daryl Dufault	Daryl Dufault	Laura Fidel (nee Wiebe)	Dmitri Ostritchenko	Daryl Dufault	Daryl Dufault
Engineer of Record	Ken Brouwer		Todd Martin	Daryl Dufault	Laura Fidel (nee Wiebe)	Steve Rice		
Support Engineers – Periodic Site Visits			Dmitri Ostritchenko	Dmitri Ostritchenko	Dmitri Ostritchenko	Luke Marquis		
Resident (Site) Engineer	varies							

EOR Consultant

At the time of the breach, the EOR was working at AMEC who were responsible for QA/QC for construction of the TSF up to Elevation 970.0 m, which was scheduled for completion in August 2014 and, at which time, the consultant BGC was scheduled to take over the project and assume QA/QC responsibilities for continuing the dam raise to 972.5 m.

The responsibilities of the EOR were not properly documented by MPMC, although it could be implied that the EOR is responsible for the design of the dam, the dam safety inspections and Quality Assurance. However, it should be noted that the EOR for Mt. Polley had limited or no control over the dam raising schedule, equipment availability, budgets, water management, hiring of MPMC inspectors and other operational aspects of the TSF that can directly impact dam safety.

EOR Construction Management Team

The AMEC management team, as outlined in the Annual Construction Reports, is shown graphically in Figure 7.1 and responsibilities are summarized in Table 7.2.

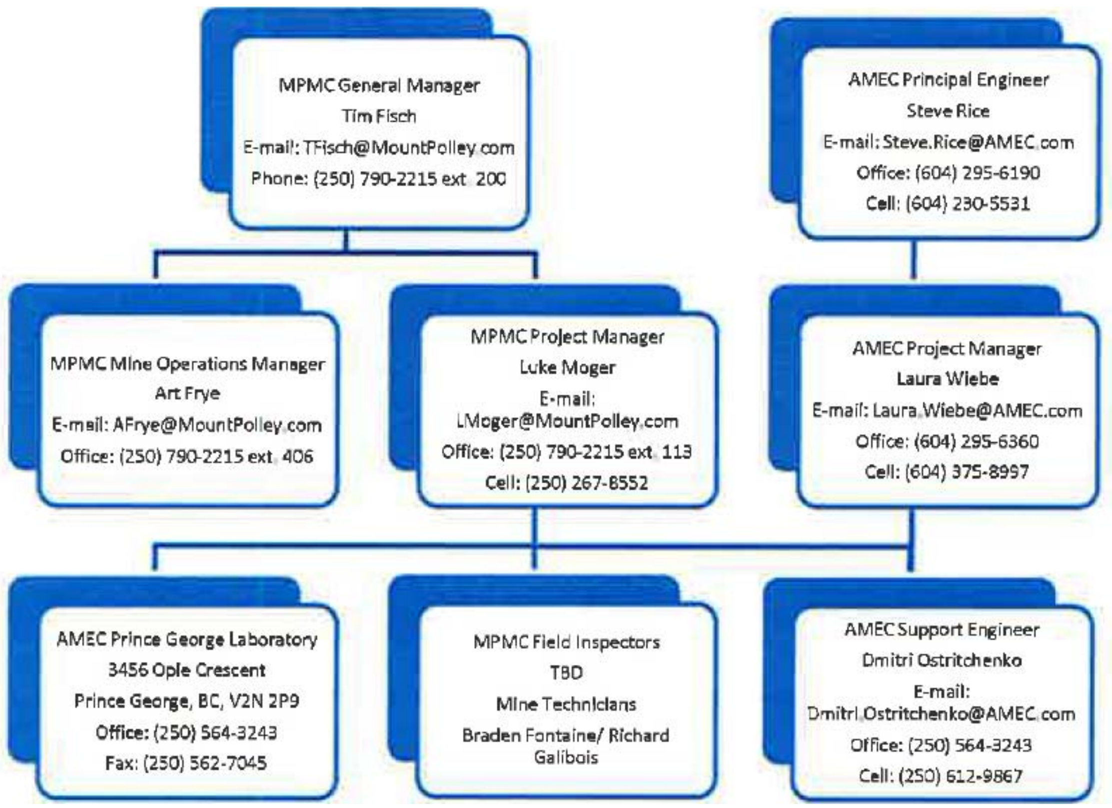


Figure 7.1 Organization Chart for AMEC – MPMC (AMEC 2013b)

Table 7.2 AMEC Construction and Design Team Responsibilities – 2011 to 2014

Position	Location	Responsibility
Senior Geotechnical Engineer	Vancouver	Responsible for design of the facility.
Principle / Review Engineer	Vancouver	Provides senior review of design and construction reports.
Project Manager	Vancouver	"Overall responsibility for upcoming and future dam raising projects. Act as the EOR for the TSF. Review of all monthly construction progress reports and liaison with the Senior Geotechnical Engineer and MPMC's Tailings Project Manager" (MPMC 2013b).
Engineer of Record	Vancouver	No documented position description was found
Support Engineers	Prince George	"Provides full time construction monitoring at the commencement of construction. After all MPMC Field Inspectors have achieved sufficient confidence and commensurate approval, the Support Engineer will provide primarily remote assistance by reviewing daily reports and instrumentation

Position	Location	Responsibility
		data as required. Conduct monthly site visits (actual frequency to be determined by site performance) to verify construction methods and specifications are being followed." (MPMC 2013b)

The assigned positions and the organization chart were, at the time of the breach, in a state of flux as the core design team had transferred from AMEC to BGC in 2012 and 2013, and the 2013 AMEC Project Manager (EOR) went on [redacted] leave in 2014. The AMEC support engineers were not registered professional engineers (one obtained his P.Eng. status in July 2014).

7.2 Relevant Consultant Practices

7.2.1 Overview

Three main consultants, KP, AMEC and BGC, have been engaged by MPMC for design and QA/QC of construction for the TSF. A summary of some of the key elements of the designs and operations is summarized in Table 7.3 and further discussion of these aspects is included in the remaining subsections.

Table 7.3 Summary of Consultant Practices

Date	Consultant	Activity
1996	KP	Feasibility Design Report does not include site investigation of the PE. Site investigations appeared to only have been carried out for the ME.
1997-2006	KP	Subsequent site investigations for construction and raising of the PE focus on borrow materials, foundation stripping requirements and impoundment soil liner conditions. "Deep" site investigation drill holes in the PE footprint were not carried out.
2005	KP	KP response to MEM's query with respect to a weaker GLU layer in the groundwater observation well (GW96-1A). KP indicated that such layers were discontinuous and were not a stability concern for the PE.
2006	AMEC	DSR assessment did not identify that the site characterization of the foundations for the PE was not adequate.
2007	AMEC	Dam optimization report indicated area of potential optimization: 1) reduce buttress, 2) reduce till core width; 3) reduce freeboard; 4) reduce consultant QC staff.
Jan. 2011	AMEC	AMEC become the EOR. The transition occurred over a period of time and there does not appear to have been a consolidated review of all site investigation data. Reliance for foundation conditions appears to have been placed with the DSR and KP assessment.
2011	AMEC	Instrumentation was installed in the PE. However, drilling planned for the breach area (Section G) was moved 150 m northwest because "access to Section G was difficult". Instrumentation not available to measure foundation pore pressures. PE slope steepened to 1.3H:1V with supporting stability analyses. QC of filter data accepted.
2012	BGC	[redacted] AMEC assigns EOR designation to the Project Manager (Daryl Dufault).
2013	AMEC	[redacted] AMEC assign EOR designation to their current Project Manager (Laura Fidel).
2013	BGC	BGC carry out the Annual Dam Safety Inspection for 2012. It appears that the EOR designation will eventually be transferred to BGC
2014	AMEC	AMEC asked by MPMC to complete QA/QC up to El. 970 m and AMEC assign the EOR designation to the senior reviewer (Steve Rice).

Date	Consultant	Activity
2014	BGC	After reaching El. 970 m, BGC were to take over EOR and QA/QC responsibilities. Design report for raise to 972.5 m indicates that the dam can safely be raised to the full height with the buttress being placed after the raise.

7.2.2 Knight Piesold

Overview

KP was retained prior to 1996 and was responsible for design stages from Stage 1a to Stage 6b (El. 958.5), up until January 2011. Site investigations and design work were carried out to support a Feasibility Study for the TSF (1996) and to obtain environmental approvals. Permit approvals, however, were carried out for each incremental raise of the dam. The applications for each of the staged raises of the dam included detail design of each stage and construction drawings for the raise.

At the end of KPs role as EOR, the PE dam slopes were nominally 1.8H:1V and the dam was stable.

Site Investigations

The practice of applying for incremental raises of the dam, however, may have led to the inherent assumption that the Feasibility Design was suitable for the detail design of each raise, with the exception of working out some of the construction aspects that were deemed important in the various areas of the facility. As described in Section 3.3.2 of this report, site investigations mainly focussed on the ME foundations, borrow materials, TSF liner soils, and groundwater monitoring wells.

Foundation investigations for the PE (Station 2+600 to 4+650), approximately 2 km in length included one deep drill hole (MP89-231) near Station 4+200. The hole was drilled for ore condemnation purposes using a rotary drill, with observations of drill cuttings. This type of drilling is not suitable for determining geotechnical properties of various materials in the foundation (USBR, 1987 pg.70). Common practice in site investigations (USBR 1974, pg. 108) is to drill holes, depending on the complexity of the site conditions, at a minimum spacing of approximately 150 m and to have the holes go at least as deep as the height of the dam. In 2008, 11 deep holes were drilled in the proposed borrow pit area downstream of the PE which identified the LGLU and glacial till soils.

As summarized in Table 8.1 and Table 8.2, in 2006 MEM requested KP to assess the characteristics of glaciolacustrine soils that were identified in a groundwater monitoring well (MW96-1a) located 150 m downstream of the breach area. KP responded that "site investigations confirm that the glaciolacustrine deposit encountered in MW96-1A is a discontinuous unit and will not adversely affect dam stability". However, there were no site investigations within the dam footprint that went as deep as the elevation of the glaciolacustine soils within the foundation of the dam, with the exception of MP89-231, discussed above.

Geology Model and Soil Properties

The KP understanding of the geology model for the site included dense glacial till overlying dense glaciolacustrine and glaciofluvial soils. As discussed in the previous section, KP did not believe that there were any less consolidated glaciolacustrine units in the PE foundations and that the GLU was at depth. These opinions were also expressed in the interviews summarized in Table 2.6.

KPs understanding of the strength of the GLU evolved over time with observations of movement in the inclinometers and direct shear tests carried out in 2006. For the embankment loads that were being applied, KP were correct in using the effective stress approach for the LGLU for the ME and were also using lower strengths than originally assumed ($\phi' = 24°$, (KP 2009b)). The assumed model for the PE was that the GLU was at depth and had a similar strength as the glacial till ($\phi' = 33°$), although no test data was available to confirm the parameters.

Water Balance, Beaches and Upstream Drains

In approximately 2006, responsibility for management of the water balance was transferred from KP to MPMC. The MPMC responsibility also included the determination of the dam raises required to maintain freeboards. KP requested that MPMC maintain a minimum tailings beach (10 m to 20 m) around the facility. However, in response to MEM request (Table 8.2) for specification of the minimum design beach width, KP responded "*the tailings impoundment have been designed to remain stable for any condition, therefore there is not a requirement for a minimum beach width in terms of embankment performance*". The KP guidance was to only permit temporary flooding (<2 months) and maximum depth of water of 0.5 m against the dam core.

The KP design included an "upstream toe drain" in the PE at approximately El. 946.3 m. Piezometer measurements taken in July 2014, prior to the breach, indicate that the drain was performing and fulfilling its function of keeping a lower hydraulic head against the core of the dam.

7.2.3 AMEC

Overview

AMECs first involvement with the project came with the DSR carried out in 2006, which was followed up with an "Optimization Review" in 2007. In consideration of the DSR, the Optimization Review, and the desire of MPMC to use the design engineers from the Huckleberry Mine, AMEC were selected to become the EOR for the TSF in January 2011, which they maintained up until the breach. During AMECs tenure, several key engineers moved to BGC and, as a result, the EOR designation was scheduled to move to BGC when El. 970 m was reached. At the time of the breach the dam elevation at the breach was El. 969.5 m.

The scope of the DSR did not include documentation of a review of the foundation geotechnical database with respect to soil conditions, both spatially and with respect to glacial history and, consequently, the lack of site investigation data for the PE was not raised as a concern.

An Optimization Report (AMEC 2007) was prepared in 2007, which considered the potential for reducing the size of the buttress, reducing the width of the core zone, reducing the freeboard, and reducing the role of the EOR in the day to day QA/QC of construction.

When AMEC assumed the role of EOR in January 2011, it does not appear that they carried out a comprehensive review of the available geotechnical data, particularly with respect to the PE.

Site Investigations, Geology Model and Soil Properties

In 2011, AMEC carried out a site investigation program that was focussed on the installation of additional instrumentation (piezometers and inclinometers) and an assessment of the GLU to determine if the clay had been pre- sheared. Three holes were drilled near the downstream toe of the PE and the spacing between the holes was on the order of 400 m to 600 m apart. The holes were located on the previous instrumentation lines (Sections J, D and G). Drill hole VW11-11 was planned to be drilled in the area of the breach (Section G), however it was moved approximately 200 m west due to access problems.

The site investigation program concluded that: 1) there was no evidence of pre-shearing in the GLU; 2) the GLU was overconsolidated; and, 3) effective stress parameters would apply. AMEC correctly assessed that undrained strength parameters would not apply in the GLU soils that they encountered in the site investigation program. AMEC did not have the KP direct shear test data and developed an empirical based effective peak shear strength of $\phi' = 28°$, which was appropriate.

AMEC addressed the previously raised MEM (2005b) concern with respect to less consolidated glaciolacustrine soils noted in MW96-1A. AMEC concluded: “The upper GLU in GW96-1 is described as firm, but is not of significant concern in this instance as the drill hole location is approximately 140 m further downstream from the current toe of the dam… In general clay layers within the GLU appear to be discontinuous and less than 5 mm in thickness which suggest a distinct facies change relative to the GLU along the ME alignment… Based on available information, foundation conditions along the PE appear more favourable than those along the ME, in terms of the presence and extent of clay-rich zones.” (AMEC 2012a)

The dismissal of the potential for UGLU for the PE was not supported with site investigation data. Only three deep drill holes were drilled within the dam footprint along the toe of the PE between Station 2+200 to 4+850. Two of the holes were drilled between Station 3+700 and Station 4+850, with a spacing on the order of 500 m. The historic mineral condemnation drill hole (MP89-231) near Station 4+200 was drilled with a rotary tricone bit drill with sampling of drill cuttings – such a sampling procedure would not have been adequate to characterize the potential for weak clay layers. The extensive number of shallow drill holes (<8 m depth) would not adequately characterize soils below a dam up to 40 m high.

Average deep drill hole spacing on the ME is on the order of 150 m and on the SE approximately 330 m spacing.

The recommended number of drill holes and spacing for site investigations for a dam foundation is not fixed and the requirements should be considered with respect to the complexity of the geologic environment and the dam design assumptions. Holes should be at least to the height of the dam and some reference documents suggest a spacing of 100 m to 150 m, or less for dams. Given the occurrence of UGLU 140 m downstream of the dam and the complexity of the geologic environment, more drilling would have been warranted and would not have been in “excess of industry norms”.

Water Balance and Beaches

During AMECs tenure, the water balance in the TSF was being overwhelmed with an increasing inflow due to expansion of the mine footprint and the inability of being able to treat and discharge water. MPMC were responsible for the water balance, and as discussed in Section 6.3.1, MPMC were not able to predict the expected water levels enough in advance to allow planning of construction and water management. Consequently, the water pond continued to grow significantly each year, and the dam had to be raised faster than previously planned.

Similar to KP, AMEC desired to have tailings beach against the dam. However, they too indicated that it was not an embankment stability concern: “In terms of beach development, as long as there is not water depth to the point where the dyke raise portion that is built out in the pond-ward direction has instability issues, there is no rush to develop a beach on the impoundment from the Main Embankment” (AMEC 2007).

QA/QC of Construction

The responsibilities of the AMEC construction team are documented in the OMS and in AMECs annual construction plan. MPMC are responsible for day to day QA/QC, with QC test data sent daily to the AMEC Support Engineer. Periodic (at least annual) site visits by the senior geotechnical engineer and/or the EOR. The procedures appeared to meet the QA/QC requirements for the dam, although the following anomalies were noted:

- The Support Engineers were Engineers in Training (EITs) with limited dam construction experience.
- The Support Engineers were to prepare monthly QA/QC reports, however these were not prepared and monthly “reports” were reported to have been transmitted with intermittent emails or verbal communication.
- The QC data for the filter zone gradation was often out of specification due to segregation. This was recognized by AMEC, but there was no formal documentation that this was acceptable.

7.2.4 BGC

BGC’s involvement with the TSF developed as the key staff from AMEC moved to join BGC. The 2012 Dam Safety Inspection was carried out by Daryl Dufault, who was also the EOR for AMEC in 2012 prior to moving to BGC. In late 2013, BGC carried out “path forward” studies that presented stability analyses for dam raises up to El. 985 m. BGC also provided input into “upgrading” the water balance basis for the TSF. In 2014, BGC were contracted to carry out the detail design to support the Permit Application for the Stage 10 dam raise rom El. 970 m to El. 972.5 m, and this report was submitted to MPMC on July 25, 2014. It does not appear that BGC carried out a comprehensive review of the available geotechnical data, particularly with respect to the PE.

The BGC 2014 report recommended the adoption of a factor of safety of 1.5 for peak strength and 1.1 for residual strength of the GLU and construction of a toe buttress for the PE. The report concluded that the dam could be safely raised to El. 972.5 m, with the toe buttress constructed after the dam raise.

BGC appropriately assessed the effective strength parameters of the LGLU, however again, their geologic model did not account for the possibility of a less consolidated glaciolacustrine layer (UGLU) at a higher elevation in the foundation.

8 REGULATORY PRACTICES

8.1 Overview

Under Section 10 of the Mines Act, "*Before starting any work in, on or about a mine, the owner, agent, manager or any other person must hold a permit issued by the chief inspector and, as part of the application for the permit, there must be filed with an inspector a plan outlining the details of the proposed work and a program for the conservation of cultural heritage resources and for the protection and reclamation of the land, watercourses and cultural heritage resources affected by the mine, including the information, particulars and maps established by the regulations or the code*".

In accordance with the above, MEM carried out reviews on Mount Polley Mines Act permit M-200 applications for each stage of the dam raises, and the key mine plan changes (MEM 2013a). There were 34 Mines Act permit applications submitted and approved up until the dam breach, with 13 of these relating to the TSF dam raises and related works. Items of note from the permit review process of the Mines Act Permit and associated amendments are summarized in Table 8.1.

MEM also has a program of periodic inspection of tailings facilities which has evolved with time in response to activities occurring at various tailing facilities in the Province and the human resources available at the time. The Mount Polley Mine tailings facility was constructed in 1996-1997 and operated between 1997 to present, with a care and maintenance shutdown from 2001 to 2005. MEM inspections over the life of the mine have included Geotechnical, Health & Safety, Electrical, Mechanical, and Reclamation. Geotechnical inspections up to the date when the dam failed are summarized in Table 8.2 along with key items of interest from the site visit and the follow up response from MPMC and/or their consultant.

Table 8.1 MEM M-200 Permits Relating to the TSF

M-200 Permit Issue Date	Application Topic	TSF Technical Documents Referenced	MEM Reviewer	Geotechnical Items of Interest	MPMC/Consultant Response
August 3, 1995	Stage 1 Design	▪ TSF Design report Vol I/II (KP 1995b, 1995c). ▪ TSF Inspection Manual (KP 1995d).	Chuck Brawner	Only one drill hole in the ME. Comments on filter design, compaction criteria, till liner, seepage cut-off, underdrains, modified centerline, winter operation (C.O. Brawner 1995a).	KP plan to drill 2 additional holes to about 15 m depth in the first stage of construction (KP 1996a).
September 23, 1996	Approval to Construct Tailings Storage Facility to Elevation 934 metres	▪ TSF letter report (KP 1996b). ▪ Borehole logs for PRW 96-1 to 4 (KP 1996c). ▪ Geotechnical information from 1996 Borehole Investigation (KP 1996d). ▪ CPT Investigations at ME (KP 1997b).	George Headly	Dam design and site investigations, drainage pipe system (MEM 1996a).	Modification of foundation drains including vertical drain holes intersecting a glaciolacustrine/fluvial soils sequence have been installed to provide relief of possible foundation pore pressures (MEM 1996a).
April 7, 1998	Approval to Construct – Tailings Storage Facility to Elevation 940 metres	▪ On-going construction requirements (KP 1997f). ▪ OMS manual for Stage Ib Embankment (KP 1997e). ▪ Report on Stage Ia/Ib Construction (KP 1997c).	Fred Matich	Confirmed that MPMC had installed pressure relief wells. Relevant data to be assembled to supplement existing information to facilitate identification of potential contingency measures (MAJM 1997).	N/A – comments from working meetings captured in MAJM (1997).
June 13, 2000	Approving Constructing of Tailings Storage Facility to Elevation 944 metres (Stage 3, design option 2)	▪ Evaluation of cycloned tailings for embankment construction (KP 1999a). ▪ Cycloned sand construction of stage 3 and ongoing stages (KP 1999c). ▪ Addendum to report on cycloned sand construction of stage 3 (KP 2000a).	Chuck Brawner	Reviewed cyclone sand alternative prepared by KP (C.O. Brawner 2000).	MPMC did not proceed with this alternative (MPMC 2000).
August 2, 2000	Continuation of June 13, 2000 permit (approves use of sand fill for downstream shell construction)	None	N/A	N/A	N/A

M-200 Permit Issue Date	Application Topic	TSF Technical Documents Referenced	MEM Reviewer	Geotechnical Items of Interest	MPMC/Consultant Response
May 30, 2001	Construction of Tailings Storage Facility to Elevation 945 metres	▪ TSF construction to elevation 945 metres (MPMC 2001a)	Chris Carr	Construction of dam crest 1 m higher to provide design freeboard until mid-2002 (MPMC 2001a).	None
May 25, 2005	Tailings Storage Facility Stage 4 Construction	▪ TSF design to ultimate elevation - report (KP 2005a) ▪ TSF design to ultimate elevation - letter (KP 2005b)	Chris Carr	Updated OMS manual with emergency response plan required (MEM 2004).	Updated OMS manual under development and will be sent to MEM by Nov 30, 2004 (MPMC 2004).
August 2, 2006	Tailings Storage Facility Stage 5 Construction	▪ Stage 5 TSF Design - report (KP 2006)	Chris Carr	"Glaciolacustrine deposit is noted in GW96-1A on cross-sections 9 and 10. The material is described as firm. What are the characteristics and extent of this deposit and could it have an influence on dam stability locally?" (MEM 2005b).	Site investigations confirm that the glaciolacustrine deposit encountered in GW96-1A is a discontinuous unit and will not adversely affect dam stability (KP 2005b).
February 19, 2008	Tailings Storage Facility Stage 6 Construction	▪ Stage 6 TSF Design - report (KP 2007b) ▪ Stage 6 TSF Design - letter (KP 2007c)	Chris Carr	Cross sections showing stability analysis for dam raise to elevation 958m. Slope inclinometer depth vs cumulative displacement plots showing cumulative displacement from the date of installation. Results from direct shear testing on lacustrine soils, if these tests have been completed (MEM 2007b).	Two brass tube samples collected at a depth of 2.5 m to 3 m in a test pit at the ME. Incorporated stabilizing berm into ME design. Three new inclinometers installed at the toe of the ME. No measured deformations recorded at ME (KP 2007c).
August 15, 2011	Mining the C2 and Boundary zone pits (Stage 7 Construction)	▪ TSF Calculation (MPMC 2011). ▪ Mount Polley Water Balance 2010 Update (MPMC 2011) ▪ Permit Conditions Response (MPMC 2011) ▪ Stage 7 Dam raise (AMEC 2011)	No review cited	N/A	N/A
June 29, 2012	Tailings Storage Facility Stage 8 Construction	▪ Stage 8 construction monitoring manual (AMEC 2012c) ▪ Design Slope 2H:1V (overall), as constructed slope 1.3H:1V (AMEC	George Warnock	Although the downstream slopes is designed at 2H:1V overall, the updated stability analysis was completed on 1.3H:1V slopes and materials strengths	N/A

M-200 Permit Issue Date	Application Topic	TSF Technical Documents Referenced	MEM Reviewer	Geotechnical Items of Interest	MPMC/Consultant Response
		2013a) ▪ Stability analysis carried out with 1.3H:1V slope (AMEC 2012d)		appear to be reasonable and the calculated FoS acceptable (MEM 2012a).	
October 15, 2012	Tailings Storage Facility Stage 8A Construction	▪ Stage 8 construction monitoring manual (AMEC 2012c) ▪ 2012 Stage 8A construction drawings and stability analysis for embankment raise to El. 965 (AMEC 2012d)	George Warnock	Short term design factor of 1.3. Stability analysis indicates this is achieved for all embankments and soil strengths appear reasonable. FoS for the PE was 1.81. Transition from upstream to fully centreline construction in order to achieve a FoS of 1.5 at closure (MEM 2012b).	Discussed with AMEC the FoS for the ME and plans for a buttressing program in 2013 (MEM 2012b).
July 25, 2013	Northwest PAG dump Expansion and South Haul Road	N/A	N/A	M-200 condition C.3 for a comprehensive water management plan to be developed prior to March 31, 2014.	Water management plan extending to July 18, 2014 submitted by MPMC to MEM.
August 9, 2013	Tailings Storage Facility Stage 9 Construction	▪ Stage 9 construction monitoring manual (AMEC 2013b) ▪ 2013 OMS manual (MPMC 2013b) ▪ 2013 Site water Balance (MPMC 2013a)	Heather Narynski	MEM required a commitment from MPMC to moving towards increasing the FoS for the ME to achieve FoS of 1.5. The FoS for the PE was 1.63. MEM expected MPMC to continue their transition to centerline construction and provide additional buttressing. Understood that sufficient mitigation measures are in place relating stability analysis to monitored piezometer data (MEM 2013b, 2013c).	Agree with all points and FoS is something site is working into their designs prior to the next raise submission (MPMC 2013c).
July, 2014	Stage 10 Dam Raise	▪ 1.3H:1V slopes with PE toe buttress ▪ FoS = 1.5 for peak strength case ▪ Report indicated PE buttress placement would occur after the dam raise ▪ Reference (BGC 2014)	N/A	Application received by MEM but was not yet reviewed. Design Report was issued by BGC on July 25, 2014 and received by MEM on July 28, 2014	

Table 8.2 MEM Geotechnical Inspections

Date of Inspection	Inspector	Geotechnical Items of Interest	MPMC/Consultant Response
September 20, 1995	G. Headley	Identified drainage requirements for near surface sand in ME (MEM 1995a).	Loose wet sandy sediments in the proposed ME seepage pond be extensively excavated to adjust design if required (KP 1995e).
October 11 and 19, 1995	J. Stevenson G. Headley	Additional test pits to define permeable and / or soft foundation soils required (MEM 1995b).	None
October 19, 1995	C. Brawner G. Headley	Identified sands in ME that may require a cutoff or drainage. Noted medium strength GLU (C.O. Brawner 1995b).	None
July 9 and 13, 1996	G. Headley	None – foundation drainage options were discussed with decision on choice of method left with the consultant (MEM 1996b).	None
August 26, 1996	G. Headley	None - detailed TSF inspection and review meeting at the KP office as part of an independent expert review by F. Matich (MEM 1996c).	None
September 27-28, 1996	G. Headley	None – design, site investigations at this stage, construction quality, and geotechnical engineering are satisfactory (MEM 1996d).	None
May 27, 1997	G. Headley	One metre head increase in several foundation soil piezometers, and a small increase in drainage flow (MEM 1997).	None
June 4, 1998	G. Headley	Minor tension cracking near upstream crest over 100 m by 5 m wide segment (MEM 1998).	KP indicated due to fine tailings being deposited between spigot points and that there would be no effect on core stability or permeability performance (MEM 1998).
June 17, 1999	G. Headley	Focus on the cyclone sand trials (MEM 1999).	None
August 17, 2000	G. Headley	Transition Zone T material placed against Zones S and B should be carefully controlled by placement method and selection of material gradation (MEM 2000).	None
April 25, 2001	C. Carr	MEM strongly supports installation of two inclinometers (extending through the glaciolacustrine sediments) at the downstream toe of the buttress in the ME. The location of the tailings pipeline along the perimeter embankment creates a potential risk of embankment washout in the event that a pipeline was to rupture (MEM 2001).	Installation of inclinometers forwarded to KP for review. Tailings line relocated (MPMC 2001b).

Date of Inspection	Inspector	Geotechnical Items of Interest	MPMC/Consultant Response
February 3, 2005	C. Carr	The water balance indicates that there will be a surplus water balance and a discharge permit will be required. Indicates better control of general rockfill is required to limit segregation (MEM 2005a).	None
October 13, 2005	N. Rose	None (MEM 2005c).	Advised the inspector that a tailings deposition plan is being developed to manage the beach/pond according to KPs recommendations (MEM 2005c).
August 30, 2006	N. Rose	Ministry requests specification of the minimum design beach width that is required for construction and operation (MEM 2006).	KP response: "the tailings embankments have been designed to remain stable for any conditions, therefore there is not a requirement for a minimum beach width in terms of embankment performance." MPMC have a fundamental objective to establish beaches and plan to only allow temporary (<2 months) flooding of beaches by a maximum of 0.5 m (MPMC 2006).
July 31, 2007	N. Rose	None (MEM 2007a).	Monitoring to be conducted in accordance with the OMS manual (MPMC 2007).
June 07, 2008	D. Apel	New operating procedures required for managing tailings line breaks. Requests to remove rocks >4" from the core zone – noted 12" rock boulder in core zone. No beach in southeast corner of ME (MEM 2008).	Written response on monitoring program (MPMC 2008) KP indicated that 12" rock was okay. Beach zone filled in with rockfill.
April 12, 2012	G. Warnock	Familiarization visit.	None
September 24, 2012	M. Cullen	None (MEM 2012c).	None
September 13, 2013	M. Cullen	None (MEM 2013c).	None

8.2 Permitting and Inspection History

8.2.1 Geotechnical Aspects

The permitting approach for the TSF was based on a staged process with each dam raise submitted for approval. The implication with this approach was that detailed design of each dam raise would be required and approved. This contrasts with the approach taken on other projects where detailed design of the Ultimate Dam is required and a permit is approved for the Ultimate Dam. Permit amendments would then only be required if there were significant changes to the design, operation or Ultimate Dam height.

Geotechnical overview of the geotechnical design of the TSF was carried out with a combination of Permit reviews and site Inspections. The Regulatory oversight can be divided into a number of key time periods, summarized as follows:

1995 to 2008

From 1995 to 2001, the work included the Feasibility Design of the dam to El. 965 m and permit applications for construction up to El. 942 m (Stage 2C). At this time, the geotechnical review was focused on the ME and MEM used outside external reviewers (Chuck Brawner and Fred Matich) to provide independent review. The review process resulted in changes to the dam design, chimney drain and pressure relief wells, as well as additional site investigations to assess the glaciolacustrine soils. Recommendations for inclinometer installations were made and acted upon when the mine started up after the care and maintenance period.

From 2001 to 2005 the mine was shut down and placed into care and maintenance. A 1 m dam raise was required to maintain freeboard.

From 2005 to 2008 the TSF was approved up to El. 958 m. Geotechnical review included MEM questioning KP on the less consolidated glaciolacustrine soils in MW 96-1A and requesting KP to collect undisturbed of GLU for testing. KP responded to both of these requests and 3 additional inclinometers were installed in the ME.

During the period of 1995 to 2008, MEM had consistent Geotechnical Engineers (G. Headley - 1995 to 2000 and Chris Carr – 2001 to 2008) reviewing the Permits and carrying out most of the site inspections.

2008 to 2012

This was a period of transition for MEM, MPMC and the consultants working on the TSF. MEM's Manager of Geotechnical Engineering left in 2008 and MEM was not able to hire a replacement until 2011. A Senior Geotechnical Engineer position was also filled with MEM in 2012 after an extended period of this position being vacant. Ron Martel, the MPMC tailings manager passed away in January 2012 and duties were transferred to Luke Moger of MPMC. AMEC started with an optimization review of the dam design in 2008 and took over from KP as EOR on January 2011.

2012 to 2014

MEM's new Manager of Geotechnical Engineering (George Warnock) joined MEM in October 2011 and carried out a familiarization visit in April 2012. The geotechnical review (June 2012) of the Stage 8 raise was based on a design section for the PE with a slope of 1.3H:1V with a small buttress (12 m wide by 7 m high) in some areas and a crest at El. 963 m. The next design stage (Stage 9) for the PE (April 2013) increased the height to El. 970 m, with a 1.3H:1V slope, which effectively covered the buttress. The MEM Geotechnical Engineers again questioned AMEC regarding the use of a FoS of 1.3, as the CDA Guideline is not clear on this aspect and some consultants chose to interpret the "Construction" period to include the operation period for the TSF, as opposed to the Starter Dam. MEM required a commitment from MPMC towards increasing the FoS for the ME to achieve FoS of 1.5. The PE had a FoS > 1.5.

In May 2014 the southeast corner (Corner 3) of the TSF was overtopped and water flowed across the crest of the dam and into the downstream rockfill shell. The reported flow was low (small stream) and was stopped with placement of fill and raises of the dam crest. The event, however, highlighted MPMC's lack of water management planning and planning of dam raises. The event was recorded as a "geotechnical incident" and MEM (Steve Rothman) viewed the site by air. MEM geotechnical personnel followed up with MPMC mine personnel on the incident.

On July 25, 2014 the BGC report on the Stage 10 dam raise to El. 972.5 was issued to MPMC. This report was submitted to MEM in an application for Stage 10 dam raise on July 28, 2014, but had not been reviewed by MEM for permit. The breach occurred a week later on August 4, 2014.

8.2.2 Overtopping Incident May 2014

MPMC assumed responsibility for the water balance and planning of the dam raises in approximately 2010, whereas previously this was carried out by KP.

A dam overtopping incident occurred on May 24, 2014. The incident comprised a low flow of water passing over the crest of the dam near Corner "3". The water flowed over the till core of the dam and into the filter/rockfill zones. A rainfall of 24 mm had occurred within the last 24 hours and the event included associated snowmelt within the catchment area. MEM staff (Steve Rothman, Senior Health and Safety Inspector of Mines) flew over the site on May 27, 2014. MEM geotechnical staff requested MPMC follow-up with an "Advice of Geotechnical Incident" form which outlines the details of the event and MPMC's response, and in future to provide MEM with a call regarding similar incidents as this would be considered a "dangerous occurrence", as per Section 1.7.3 of the HSRC.

Based on MEM's understanding of the incident from discussions with MPMC during a May 27th teleconference call, it was determined that MPMC appeared to have the situation under control. MEM indicated that follow-up would be required to confirm whether an "overtopping" and possible unauthorized discharge occurred, as well as to discuss future dam design and operations.
The "Advice of Geotechnical Incident" form submitted indicated the incident as "loss of design operating freeboard allowance at tailings storage facility". MEM did not have record of receiving correspondence from MPMC during the incident to clarify whether a dam "overtopping" occurred or what the minimum freeboard was during the event. The first survey of freeboard was received by

MEM on June 2nd in AMEC's memo dated May 30th that indicated the pond elevation and the dam elevation at corner "3" to be recorded as the same elevation (zero freeboard) on May 26th. Based on this information, MEM considered the incident to be classified as a dam "overtopping".
MEM's follow-up on the incident included weekly updates from MPMC on the status of the site conditions (freeboard, construction activities etc.), a memo issued by AMEC outlining the timeline and incident daily status, and a water management plan endorsed by AMEC.

The overtopping incident indicates the following:

- MPMC did not adhere to the OMS manual which indicates that the mill will stop operations when the pond level reaches 0.9 m below the dam crest.
- MPMC were not adequately monitoring the water levels to be aware of the incident in a timely manner.
- MPMC water balance calculations were not appropriate to predict the anticipated water levels.

8.2.3 Water Discharge and MOE Aspects

A significant influencing factor on the water balance was the ability to allow discharge of surplus water to the environment. The TSF was permitted as a "zero discharge" facility, but as early as 2005 MPMC, MEM and MOE recognized that there was a surplus of water, which would increase as the mine footprint grew and that water discharge would be required. This section of the report is based on limited information obtained from interviews and document references and a complete review of all relevant data and reports has not been carried out.

Potential alternatives to reduce the quantity of water for discharge and allow discharge included the following:

- Spray evaporators, which were installed downstream of the ME and were modestly productive in evaporating water.
- A bio-treatment cell that could provide some low level treatment for some parameters associated with the seepage water.
- Application for Site Specific Water Quality Objectives (SSWQO's) for discharge of water into Hazeltine Creek and/or Polley Lake. We understand that the SSWQO's were not approved and ultimately approval was only given for discharge of water that met the aquatic guidelines. We understand that this only allowed MPMC to discharge approximately 100,000 m^3 /year versus the permitted 1.2 Mm^3/year from the seepage ponds.

None of the above approaches were adequate to manage the increasing volumes of water. In 2014, MPMC reported that they were commissioning an ion exchange water treatment plant that was scheduled start treating water in 2014/2015. Reports or documentation of this option were not received by MEM.

8.3 Review of Health Safety and Reclamation Code Requirements

A summary of the relevant sections of the HSRC requirements pertaining to tailings dams is presented in Table 8.3.

Table 8.3 Review of HSR Code Requirements

Code Reference	Code Requirement	History of Compliance
1.1.2	Notwithstanding the absence of a specific code requirement, all work shall be carried out without undue risk to the health and safety of any person.	No undue risks have been noted in the MEM inspection records.
10.7.30	The owner, agent, or manager shall undertake monitoring programs, as required by the chief inspector, to demonstrate that reclamation and environmental protection objectives including land use, productivity, water quality and stability of structures are being achieved.	MPMC carried out geotechnical monitoring of the dam. The May 2014 overtopping incident does indicate that the monitoring program for freeboard was not effective.
10.1.8	Tailings Impoundments, water management facilities, dams and waste dumps shall be designed by a professional engineer.	The TSF dam designs have been carried out by a professional engineer.
10.1.5	Major impoundments, water management facilities and dams shall be designed in accordance with the criteria provided in the Canadian Dam Association, Dam Safety Guidelines.	The dams were designed in general accordance with the criteria.
10.5.1	The manager shall ensure that operation of a tailings or water management facility does not commence until an “as-built” report prepared by a professional engineer certifying that the facility was designed and constructed according to section 10.1.5 to this code has been submitted to the chief inspector and a permit to operate the facility has been received.	Stages 1, 2, 3, 4, 5, 6, 7, 8, 9 “as-built” reports have been prepared for each stage and permits have been received for each.
10.5.2	An Operation, Maintenance and Surveillance (OMS) Manual shall be prepared and provided to an inspector and to all employees involved in the operation of a major dam or major impoundment, prior to commissioning. The manual shall be revised regularly during operations, decommissioning and closure of the structure.	An OMS manual has been prepared and has been revised a number of times. The OMS manual (MPMC, 2013b) was out of date and was not signed.
10.5.3	The manager shall submit an annual dam safety inspection report prepared by a professional engineer on the operation, maintenance and surveillance of the tailings and water management facilities and associated dams to the chief inspector.	Annually: 1997 to 2013 2014 - Exempt
10.6.7	The long term stability of exposed slopes of major impoundments shall meet the criteria provided in the Canadian Dam Association, Dam Safety Guidelines at the time of permitting or as amended by the chief inspector.	The dams were designed in accordance with the criteria.
10.6.8	A major impoundment classified as high and very high failure consequence during operation and closure shall have an Emergency Preparedness Plan.	An emergency preparedness plan was not required for Mount Polley as the TSF dams were classified as “significant”. However, a brief section on emergency preparedness is included with the OMS manual.
10.7.18	Impoundment facilities shall be inspected, monitored and maintained to ensure stability.	Inspections and instrumentation monitoring was carried out.
10.7.19	All permanent spillways shall be designed by a professional engineer in accordance with the Canadian Dam Association Dam Safety Guidelines and installed prior to final abandonment of the tailings dam.	Not applicable.

8.4 Regulatory Practices Relevant to the Failure

8.4.1 Design and Permit Review

Tailings facilities typically operate over a long period of time and, consequently, there is a challenge in maintaining continuity and consistency of review and documentation. The original permit approval for the TSF was for a relatively low height and, therefore, the scrutiny of data for the Ultimate Dam, and hence the PE, was a low priority, as this could be dealt with later with respective permit applications.

The process of approving multiple small incremental raises could lead to the assumption that because the previous raise was acceptable it is likely that a new raise will be if it is supported by a stability analysis. This can remove the more rigorous review procedure that would be applied to a new large dam. This observation applies to MEM, MPMC and the Consultants and can result in the lack of critical thinking and of looking at the "big picture" as the structure becomes larger.

The MEM Geotechnical Engineer questioned both AMEC and KP with respect to the potential presence of a less consolidated GLU, and ultimately relied on the Designers professional judgement.

The MEM Geotechnical Engineer questioned both KP and AMEC on the requirement for minimum beaches and ultimately relied on the Designers professional judgement that only limited beaches were required for embankment stability.

It is possible that MEM could have required the EOR to use a FoS of 1.5 when it was identified as an issue in 2013. However, AMEC had calculated a FoS of 1.58 (see Table 4.6) for the PE design section D. Had MEM required a FoS of 1.5, this would only have affected the ME. Further, Section 6.6 of the 2007 CDA Guidelines states that, *"Lower calculated factors of safety for static assessment may be acceptable for existing structures with demonstrated performance supported by appropriate monitoring…."* Consequently the CDA Guidelines were not explicit enough for MEM to require that a FoS of 1.5 be used, particularly as experienced and knowledgeable EORs had accepted this practice over the life of the mine. The issue was resolved by acceptance of the EORs professional judgement that FoS of 1.3 was acceptable and the commitment to move to a FoS of 1.5 for future raises (above El. 970 m). In any event this would not have had an impact on construction or design decisions surrounding the perimeter embankment. It is uncertain that if MEM had retained an independent reviewer, whether or not these design aspects would have been identified as potential data gaps or design deficiencies.

8.4.2 Site Inspections

The periodic site inspections, by their short time frame and their limitation to visual inspection and instrumentation review, could only be expected to identify significant issues of non-compliance or other visual changes of potential significance. The inspections could not determine if the site investigations, design parameters and design are adequate and this is properly left to the Permit Review stage.

Additional site inspections would not have anticipated the failure.

8.4.3 HSR Code Requirements

The HSR Code with respect to tailings dams contains relatively few conditions. The most important condition relevant to the Mount Polley failure is the requirement to use the criteria provided by the CDA Guidelines. The perceived lack of clarity in the CDA guideline for the factor of safety was important but could not be expected to be anticipated in development of the Code.

The HSR Code places limited requirements on the Mining Company for submission of documents and penalties for non-compliance. It is uncertain that if these were applied if significant data gaps could have been anticipated.

8.4.4 MEM Documentation

The TSF failure highlights the lack of a comprehensive database for each of the operating mines in British Columbia. A functional database may have allowed for a quicker understanding of the design and operating elements as inspectors and engineers change, as well as mining personnel and consultants change. However, it is uncertain that if such a database was in place that significant data gaps or design concerns could have been anticipated.

8.4.5 MOE Permit to Discharge Water

If an acceptable permit to discharge water had been developed earlier the operating pond level could have been lower and, consequently, the pond may not have overtopped the dam after the slope had slumped on the order of 3 m to 5 m. This could have potentially prevented the catastrophic release of water and tailings.

9 DISCUSSION OF IRP REPORT

The CIM investigation and the resulting conclusions and recommendations that KCB have developed for this report are materially similar to the IRP report, although there are some areas that the CIM investigation has been able to assess information in more detail and provide additional perspective. The main comparisons between the IRP report and this report include:

1. The mechanism of failure was sliding of the embankment on a weak clay layer in the foundation. The technical analysis of the mechanism of failure has been improved with the incorporation of all the site investigation data and additional detailed technical studies and, while there are some differences in strengths and conditions, the conclusion on the mechanism of failure remains unchanged.

   The principal cause of the design error was the lack of adequate site characterization and understanding of the geologic history and engineering properties of the foundation soils. The presence of the upper glaciolacustrine clay layer had been discounted and, therefore, could not be appropriately assessed as to it distribution or its strength. The IRP stressed the lack of undrained strength analysis and inappropriate use of the observational approach and, while this understanding would have improved the overall design and performance monitoring, it would not have predicted the presence of the weak clay layer and, therefore, would not have prevented the failure.

2. The timeline of the failure presented in the IRP report has been improved with the CIM investigation, which indicates that failure occurred at approximately 11:40 PM August 3, 2014; with a major erosion breach occurring at approximately 1:08 PM, August 4, 2015, and ongoing erosion until at least noon on August 4. The Panel observed that human intervention was not a factor, although KCB notes that MPMC had carried out some excavation at the toe of the dam in November 2013. The excavation was not backfilled and resulted in a small reduction in the factor of safety.

3. The CIM investigation supports the Panel's conclusion that: "The Panel firmly rejects any notion that business as usual can continue" and supports the recommendations developed by the Panel. It is worthwhile to note however, that the Panel's review of performance of BC Dams, which indicated that "on average there will be two failures every 10 years", while statistically valid it is misleading as Mt Polley is arguably the only "major –serious" tailings dam failure in the history of mining in BC.

   The application of Best Available Technologies (BATs) and implementation and enforcement of Best Available Practices (BAP) is required. The Panel's suggestion that compacted dewatered filtered tailings is <u>the</u> BAT should be critically examined as filtered tailings technologies are currently not feasible for low-grade, high-tonnage, deposits located in the mountainous wet environments of BC. Other technologies are available to reduce risk and these need to be adopted, as appropriate. BAT for closure should reduce risk by eliminating or limiting the ability to store water.

4. The IRP concluded that: “management practices had significant influence on the design, construction and operation of the TSF”, but were unable “to provide an assessment of the role of management and oversight and its contribution to the cause of the failure”. The KCB report documents the management and oversight procedures in place leading up to the failure and believe that these factors influenced the failure.
5. The Regulatory framework for tailings dams in BC needs to be upgraded to ensure protection of the public and the environment. The review needs to consider measure for enforcement.
6. The continued development and improvement of Guidelines, such as those from the Canadian Dam Association, APEGBC and the Mining Association of Canada should continue to be promoted by those organizations.

10 CONCLUSIONS AND RECOMMENDATIONS

10.1 Conclusions

Mechanism of Failure

1. The mechanism of failure was embankment sliding on a weak clay layer located approximately 10 m depth in the foundation of the dam. The failure was initiated by the steep downstream slope, excavation at the toe and raising of the crest of the dam. These conditions induced an undrained shear response in the clay in the downstream toe of the dam, leading to progressive failure. The dam slumped (dropped in elevation) approximately 5 m, which led to overtopping and erosion of the dam. (KCB 2015).

   The dam stability was positively influenced by an upstream drain, which was constructed upstream of the core zone of the dam to maintain a lower hydraulic head against the dam, in spite of a high pond water level. However, this was somewhat offset by the presence of confined glaciofluvial layers in the foundation, which confined the groundwater leading to increased pore water pressures in the foundation clay (KCB 2015).

2. The dam slump (failure) occurred at approximately 11:40 PM, August 3, 2015. Subsequently, the dam crest was eroded, with an acceleration of flow starting at approximately 12:50 AM August 4, 2015, and major flows and power loss at approximately 1:08 AM, August 4, 2014. The dam continued to erode overnight, with major erosion reducing until approximately noon on August 4, 2014.

Cause of the Failure

3. A root cause of the failure was an incomplete interpretation of the foundation geology, which was influenced by an inadequate site characterization. Until 2011, there was only one deep drill hole in the foundation of the PE, which is approximately 2 km long. Additionally, the drill hole was intended as geological condemnation holes and there was no geotechnical testing or collection of undisturbed samples. In 2011, geotechnical investigations included three drill hole locations. The drill holes were approximately 500 m apart and missed the upper clay layer (UGLU). In 2006 MEM noted the presence of the UGLU in a drill hole 140 m downstream of the breach area but the significance of this was discounted by both AMEC and KP as not being applicable to the dam footprint. The glacial history of the dam foundation was not adequately understood.

   The interpretation of the geotechnical properties of the lower clay layer (LGLU) was appropriate and the dam would not have failed if the UGLU was not present. The planned implementation of a buttress and the use of the residual shear strength (BGC 2014) would have inherently accounted for an undrained strength response in the LGLU.

   The use of the constructed 1.3H:1.0V slope was based on the inherent assumption that there was a high degree of certainty that the foundation soils were dense and strong. The site investigation data does not support this certainty and the slope was too steep for the soil foundation.

Factors Influencing the Failure

4. MPMC management of the water balance for the TSF was not adequate and was not developed by or approved by a Qualified Professional. The water balance did not adequately predict the required dam raises and dam elevations with time so the appropriate material supply and dam construction could be planned and executed. The overtopping event in May 2014 was a symptom of the lack of water balance prediction, water level monitoring and appropriate planning.

   The large volume of stored water did not directly influence the factor of safety of the dam (due to the upstream drain outlined in Conclusion 3), however the large volume of water contributed directly to the erosion of the dam and the accompanying release of tailings.

5. The constructed steep dam slope (1.3 H:1.0V) was influenced by the requirement to increase the dam height before the mine planning and delivery of rockfill from the mine could be executed. The additional mine rock was planned to be used to flatten the slopes to 2H:1V, but was not going to be available until later in 2014 or early 2015. The unplanned increase in dam height was due to the lack of a comprehensive water balance (Conclusion 4).

   The MPMC excavation at the toe of the slope was carried out without a plan and the presence of the AMEC Support Engineer and, due to weather conditions, could not be inspected and, therefore, was not backfilled and remained open until the failure. The excavation resulted in a small reduction in the factor of safety.

   The AMEC and BGCs stability analyses indicated an adequate factor of safety (FoS > 1.5) and, therefore, MPMC did not have a sense of urgency in execution of fill placement at the toe of the dam and achieving the 2H:1V slope. The consultants did not recognize that they did not have adequate site characterization and that the 1.3H:1V slope inherently assumed dense and strong foundation soils.

6. MPMC was a member of the Mining Association of Canada (MAC) and, therefore, committed to implementation of the Tailings Management Systems outlined in the MAC Guidelines. However, at the time of the failure the requirements for development, execution and documentation of the Tailings Management System had not been achieved by MPMC.

7. Project Permits applications were submitted to MEM in incremental stages (9 stages up to the time of failure). This process may have led to some complacency in the issuance of annual permits with respect to requesting a comprehensive review of the geotechnical properties of the foundations soils. The complacency may have applied to MPMC, MEM and the Consultants.

8. The TSF had five Engineers of Record in the period of 2010 to 2014 and, although it should be expected, it is unlikely that each that each EOR would have had the time and resources to become fully conversant with the dam design. The responsibilities of the EOR were not documented and could be inferred to be either the person responsible for managing

individual projects each year or the person responsible for understanding all aspects of the dam design. This lack of clarity in the responsibility of the EOR created a culture of "someone else is responsible for understanding all aspects of the dam design".

10.2 Recommendations

Corporate Governance

1. All mine operators should be required to follow the MAC Guidelines
2. The MAC Guidelines should be improved to require the following:
   - Designation of a Dam Safety Manager for each TSF.
   - Guidance on what is required to document the Tailings Management System such that it can be audited by a Third Party, such as ISO.
   - Guidance on developing a Corporate Culture that includes Tailings Dam Safety as part of the Human Health, Safety and Environment Systems currently in place at the mines.
   - Roles and responsibilities should define the responsibilities of the EOR and the Dam Safety Manager.
   - The MAC Guidelines require Mines to carry out Risk Assessments and to develop Risk Management Plans for all identified risks. This process needs to be reinforced and become part of the annual update of the Operations, Maintenance and Surveillance Manual.
   - Guidance on integrating the Emergency Preparedness and Response Plan (EPRP) for the TSF within the overall minesite EPRP.

Professional Practice

3. The Professional Practice Guidelines prepared by the Association of Professional Engineers and Geoscientists of BC should be followed. The most current relevant guideline is the Dam Safety Review Guideline (2014). APEGBC are currently preparing a guideline for Site Characterization of Dams, as recommended by the IRP.

   The EOR should carry out a comprehensive Dam Safety Review and sign the Assurance Statement. The EOR should be current on tailings dam design practice and following the guidelines prepared by CDA, APEGBC, ICOLD, and others.

   The CDA Guidelines for tailings dams should be improved to include:

   - Clarity on the use of Factors of Safety.
   - Emphasis on risk reduction at all stage of the TSF, with emphasis on landform design and reducing the risk of long term environmental legacies.
   - Review of the intervals for Dam Safety Reviews reflecting the continued staged construction of tailings dams.

4. The use of Independent Review Boards is recommended for all TSFs. The makeup of the Board, and the Terms of Reference of the Board need to be defined to appropriately consider: the complexity of the TSF.

Best Available Technologies (BAT)

5. BATs for tailings storage should be applied to reduce the risk as far as reasonably possible. The BAT of filtered tailings, as proposed by the IRP, is appropriate for some facilities but is not an appropriate technology, at this time, for most low-grade, high-tonnage operations in the climatic and topographic conditions of BC.

   BATs are available for reducing the physical and environmental risks of TSFs and these should be identified and assessed transparently and incorporated into the design, permitting, operation and closure. BATs for closure should lead to stable landforms that reduce the long term risks to the public.

Regulatory Practice

6. A Manager of Tailings Dam Safety should be appointed within MEM, whose responsibilities include oversight of all tailings dams in BC and oversight of geotechnical inspection functions of tailings dams within the Province.

   A data management and tracking system is required to manage site inspections, dam safety reviews, dam design data, etc. A register of dam safety records (e.g. tailings dam safety recommendations, inspections, etc.) should be maintained for each TSF and the register should be publically available on a website. A factual data record should be maintained for each TSF.

   The use of dam slopes steeper than 2H:1V should be prohibited unless approved by MEM.

7. The Ministry of Environment needs to coordinate with MEM to regulate TSF water release during wet years and extreme precipitation events. The accumulation of surplus water, on an annual basis, should be limited.

8. The regulatory framework for tailings dams needs to be reviewed. The MEM Code review should require Mines to follow both CDA and MAC Guidelines. The MEM Code review should incorporate appropriate enforcement regulations.

11 LIMITATIONS OF THIS REPORT

This report has been prepared based on the factual information received from MEM up to May 28, 2015 and the CIM interviews with MPMC, construction and consultant employees whom may or may not have been directly involved with the TSF. As requested by MEM, KCB has reviewed and summarized the information to provide an opinion on the tailings management, construction and regulatory practises relevant to the TSF breach. The purpose of the report was to support the CIM investigation and is not to assign responsibility or determine who was at fault or who caused the failure. The report identifies information that was collected and reviewed, recognizing that the documentation of this information was not always complete or consistent. The reader should refer to the references cited for specific details and further information.

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REFERENCES

AMEC. 2006. Dam Safety Review – Mount Polley Mine Likely, British Columbia. December. MEM Reference ID: MP00104.

AMEC. 2007. Optimization Potential – Follow Up from Dam Safety Review. February. MEM Reference ID: MP10035.

AMEC. 2011. Mount Polley Mines Tailings Storage Facility – Stage 7 2011 Construction Monitoring Manual. April. MEM Reference ID: MP00039.

AMEC. 2012a. Mount Polley Mine Project: Tailings Storage Facility 2011 Site Investigation – FINAL. March. MEM Reference ID: MP10012.

AMEC. 2012b. Mount Polley Mine Tailings Storage Facility 2011 Construction As-Built Report and Annual Review. March. MEM Reference ID: MP00041.

AMEC. 2012c. Mount Polley Mines Tailings Storage Facility – Stage 8 2012 Construction Monitoring Manual. March. MEM Reference ID: MP00040.

AMEC. 2012d. “RE: 2012 Stage 8A tailings Storage Facility Construction Drawings and Stability Analyses for Embankment Raise to 965 m.” September. MEM Reference ID: MP00042.

AMEC. 2013a. Mount Polley Mine Tailings Storage Facility Stage 8/8A 2012 As-Built Report. March. MEM Reference ID: MP00217.

AMEC. 2013b. Mount Polley Mine Tailings Storage Facility – Stage 9 2013 Construction Monitoring Manual. April. MEM Reference ID: MP00045.

AMEC. 2014a. Mount Polley Mine Tailings Storage Facility Stage 9 2013 As-Built and Annual Review Report. March. MEM Reference ID: MP00044.

AMEC. 2014b. “Mt. Polley – Tailings Dam SI Installation”. (File Submission – Reference ID: AMEC010405, received October 17, 2014)

AMEC. 2015. “Mount Polley Mine Tailings Storage Facility Sensitivity Analyses”, Submitted to MEM by McCarthy Tetrault on April, 2015.

BGC Engineering Inc. (BGC). 2013. Mount Polley Mine tailings Storage Facility 2012 Annual Review. April. MEM Reference ID: MP00043.

BGC Engineering Inc. (BGC). 2014. Mount Polley Mine – Tailings Storage Facility Sage 10 Raise Design Report. July. MEM Reference ID: MP00208.

Bichler, A.J. & Bobrowsky, P.T. 2003. “Quaternary geology of the Hydraulic map sheet (NTS 93/A12), British Columbia,” British Columbia Geosciences, Research and Development Branch, Open File 2003-7, scale 1:50 000.

Canadian Dam Association (CDA). 2007. “Dam Safety Guidelines.” Toronto.

Clague, J. J. 1988. “Quaternary Stratigraphy and History, Quesnel, British Columbia”, *Geographie physique et Quaternaire.* 42(3), pp. 279-288.

Clague, J. J. 1991. “Quaternary stratigraphy and history of Quesnel and Cariboo river valleys, British Columbia: implications for placer gold exploration.” *Current Research, Part A, Geological Survey of Canada, Paper.* 91-1A, p. 1-5.

Clague, J.J., Hebda R.J., and Mathewes, R.W. 1990. “Stratigraphy and Paleoecology of Pleistocene Interstadial Sediments, Central British Columbia.” *Quaternary Research*. Vol. 34, pp. 208-226.

C.O. Brawner Engineering Ltd. (C.O. Brawner). 1995a. “Mt. Polley Tailings Dam.” October 13. MEM Reference ID: MP00061.

C.O. Brawner Engineering Ltd. (C.O. Brawner). 1995b. “Mt. Polley Tailings Dam.” October 23. MEM Reference ID: MP00006.

C.O. Brawner Engineering Ltd. (C.O. Brawner). 2000. “Mt. Polley Mining Corp. Tailings Storage Facility Cyclone Sand Review.” February. MEM Reference ID: MP00117.

Fulton, Robert J. 1991. “A conceptual Model for Growth and Decay of the Cordilleran Ice Sheet.” *Geographie physique et Quaternaire*. 45(3), pp. 281-286.

Holtz, R.D. and W.D. Kovacs. 1981. *An Introduction to Geotechnical Engineering*. New Jersey: Prentice-Hall, Inc.

Hynes-Griffin, M.E., and A.G. Franklin. 1984. “Rationalizing the Seismic Coefficient Method.” Miscellaneous Paper GL-84-13, Department of the Army Waterways Experiment Station, Corps of Engineers, Vicksburg, Mississippi.

Independent Expert Engineering Investigation and Review Panel (IRP). 2015. Report on Mount Polley Tailings Storage Facility Breach. January. Province of British Columbia.

Klohn Crippen Berger Ltd. (KCB). 2015. Mount Polley Tailings Dam Failure, Assessment of Failure Mechanism. May.

Knight Piésold Ltd. (KP). 1990. Report on Geotechnical Investigations and Design of Open Pit, Waste Dumps and TSF. February. KP Reference ID: 11162/1-1.

Knight Piésold Ltd. (KP). 1995a. Report on 1995 Geotechnical Investigations for Mill Site and TSF. March. KP Reference ID: 1623/1.

Knight Piésold Ltd. (KP). 1995b. Tailings Storage Facility Design Report – Volume I – Main Report. May. KP Reference ID: 1625/1. MEM Reference ID: MP00001.

Knight Piésold Ltd. (KP). 1995c. Mt. Polley Project Tailings Storage Facility Design Report - Volume II - Appendices. May. KP Reference ID: 1625/1. MEM Reference ID: MP00002.

Knight Piésold Ltd. (KP). 1995d. Tailings Storage Facility Site Inspection Manual. May. KP Reference ID: 1625/2. MEM Reference ID: MP00003.

Knight Piésold Ltd. (KP). 1995e. "Mt. Polley Tailings Impoundment Site Inspection - Observations and Recommendations." September. KP Reference ID: 1625/1. MEM Reference ID: MP00058.

Knight Piésold Ltd. (KP). 1996a. "Response to Review Comments on Tailings Embankment Design." January. KP Reference ID: 1625/6. MEM Reference ID: MP00005.

Knight Piésold Ltd. (KP). 1996b. "Mt. Polley Project – Tailings Storage Facility." June. MEM Reference ID: MP00064.

Knight Piésold Ltd. (KP). 1996c. "Borehole Logs for PRW 96-1 to 4." fax to Graham Greenway, copied to George Headley. July. MEM Reference ID: MP00068.

Knight Piésold Ltd. (KP). 1996d. "Geotechnical Information Obtained from 1996 Borehole Investigation", fax to Ken Brouwer, copied to George Headley. July. MEM Reference ID: MP00067.

Knight Piésold Ltd. (KP). 1997a. 1996 Groundwater Monitoring Well Installation Program. February. KP Reference ID: 1628/4.

Knight Piésold Ltd. (KP). 1997b. Tailings Storage Facility Updated Design Report. June. KP Reference ID: 1627/2.

Knight Piésold Ltd. (KP). 1997c. Report on Stage 1a/1b Construction. August. KP Reference ID: 10162/7-5, MEM Reference ID: MP00019.

Knight Piésold Ltd. (KP). 1997d. Stage 2A tailings Facility Construction, Selected Excerpts from Reference Information. November. KP Reference ID: 10162/9-2.

Knight Piésold Ltd. (KP). 1997e. Operation, Maintenance and Surveillance Manual for Stage 1b Embankment (945 m). November. KP Reference ID: 10162/7-3, MEM Reference ID: MP00009.

Knight Piésold Ltd. (KP). 1997f. Report on On-Going Construction Requirements. December. KP Reference ID: 10162/9-3. MEM Reference ID: MP00008.

Knight Piésold Ltd. (KP). 1999a. Evaluation of Cycloned Sand Tailings for Embankment Construction. June. KP Reference ID: 10162/11-1. MEM Reference ID: MP00013.

Knight Piésold Ltd. (KP). 1999b. Report on 1998 Construction and Annual Inspection. June. KP Reference: 11162/10-1. MEM Reference ID: MP00012.

Knight Piésold Ltd. (KP). 1999c. Report on Cycloned Sand Construction of Stage 3 and On-Going Stages of the Tailings Storage Facility, Volumes I and II. December. KP Reference ID: 11162/12-2. MEM Reference ID: MP00014 and MP00015.

Knight Piésold Ltd. (KP). 2000a. Addendum to Report on Cycloned Sand Construction of Stage 3 and On-Going Stages of the Tailings Storage Facility. May. KP Reference ID: 11162/13-4. MEM Reference ID: MP00021.

Knight Piésold Ltd. (KP). 2000b. Report on 1999 Construction. August. KP Reference ID: 11162/13-5. MEM Reference ID: MP10032.

Knight Piésold Ltd. (KP). 2001. Report on Stage 3 Construction. October. KP Reference ID: 11162/14-3. MEM Reference ID: MP00072

Knight Piésold Ltd. (KP). 2005a. Design of the TSF to Ultimate Elevation. March. KP Reference ID: VA101-1/8-1. MEM Reference: MP00026.

Knight Piésold Ltd. (KP). 2005b. Design of the TSF to Ultimate Elevation. April. MEM Reference ID: MP00139.

Knight Piésold Ltd. (KP). 2005c. "Mt. Polley Mine – Review of Tailings Dam Filter Performance." November. KP Reference ID: VA101-1/10-A.01. Received March 27, 2015.

Knight Piésold Ltd. (KP). 2006. Stage 5 Design of TSF. July. KP Reference ID: VA101-1/12-1. MEM Reference ID: MP00027.

Knight Piésold Ltd. (KP). 2007a. Report on Stage 4 Construction. March. KP Reference ID: VA101-1/10-1. MEM Reference ID: MP00031.

Knight Piésold Ltd. (KP). 2007b. Stage 6 Design of TSF. June. KP Reference ID: VA101-1/18-1. MEM Reference ID: MP00032.

Knight Piésold Ltd. (KP). 2007c. "Re: Mount Polley Stage 6 TSF Design." December. MEM Reference ID: MP00225.

Knight Piésold Ltd. (KP). 2008a. Report on Stage 5 Construction. April. KP Reference ID: VA101-1/14-1. MEM Reference ID: MP00033.

Knight Piésold Ltd. (KP). 2008b. "Buttress Requirement for the Main Embankment." December. KP Reference ID: VA08-02223.

Knight Piésold Ltd. (KP). 2009a. TSF Report on 2008 Annual Inspection. February. KP Reference ID: VA101-1/24-1. MEM Reference ID: MP00035.

Knight Piésold Ltd. (KP). 2009b. "Buttress Requirement for the Main Embankment." July. KP Reference ID: VA09-00838.

Knight Piésold Ltd. (KP). 2009c. Report on Stage 6A Construction. July. KP Reference ID: VA101-1/23-1. MEM Reference ID: MP00034.

Knight Piésold Ltd. (KP). 2011a. TSF Report on Stage 6b Construction. January. KP Reference ID: VA101-1/29-1. MEM Reference ID: MP00036.

Knight Piésold Ltd. (KP). 2011b. TSF Report on 2010 Annual Inspection. January. KP Reference ID: VA101-1/29-2. MEM Reference ID: MP00037.

Knight Piésold Ltd. (KP). 2011c. "Re: Mount Polley Mine – Site Water Management", Letter from Les Galbraith to Ron Martel. February. KP Reference ID: VA11-00252.

Knight Piésold Ltd. (KP). 2014. "MPMC Information Request". December. KP Reference ID: VA14-01875.

Leps, T.M. 1970. "Review of Shearing Strength of Rockfill." *Journal of the Soil Mechanics and Foundations Division – Proceedings of the American Society of Civil Engineers*. SM 4, pp. 1159-1170.

Levson, V. M., and T.R. Giles. 1997. "Quaternary Geology and Till Geochemistry Studies in the Nechako and Fraser Plateaus, Central British Columbia (NTS 93 C/1, 8, 9; F/2, 3, 7; L/16; M-1)".

MAJM Corporation Ltd. (MAJM). 1997. Geotechnical Review, Drainage Aspects Main Embankment Dam Tailings Storage Facility Mt. Polley Project, British Columbia. March. MEM Reference ID: MP00017.

Mining Association of Canada (MAC). 2011a. "A Guide to the Management of Tailings Facilities".

Mining Association of Canada (MAC). 2011b. "Developing an Operation, Maintenance and Surveillance Manual".

Mining Association of Canada (MAC). 2011c. "A Guide to Audit and assessment of Tailings Facility Management".

Mining Association of Canada (MAC). 2011d. "Towards Sustainable Mining (TSM) Assessment Protocol".

Ministry of Energy and Mines (MEM). 1995a. "Geotechnical Inspection." September. MEM Reference ID: MP00154.

Ministry of Energy and Mines (MEM). 1995b. "Geotechnical Inspection." October. MEM Reference ID: MP00156.

Ministry of Energy and Mines (MEM). 1996a. "Re: Mount Polley Tailings Dam." July. MEM Reference ID: MP00069.

Ministry of Energy and Mines (MEM). 1996b. "Geotechnical Inspection." July. MEM Reference ID: MP00158.

Ministry of Energy and Mines (MEM). 1996c. "Geotechnical Inspection." September. MEM Reference ID: MP00159.

Ministry of Energy and Mines (MEM). 1996d. "Geotechnical Inspection." October. MEM Reference ID: MP00160.

Ministry of Energy and Mines (MEM). 1997. June. "Report of Geotechnical Inspector." MEM Reference ID: MP00161.

Ministry of Energy and Mines (MEM). 1998. "Report of Geotechnical Inspector." June. MEM Reference ID: MP00163.

Ministry of Energy and Mines (MEM). 1999. "Report of Geotechnical Inspector." July. MEM Reference ID: MP00166.

Ministry of Energy and Mines (MEM). 2000. “Report of Geotechnical Inspector.” September. MEM Reference ID: MP00167.

Ministry of Energy and Mines (MEM). 2001. “Report of Geotechnical Inspector.” May. MEM Reference ID: MP00170.

Ministry of Energy and Mines (MEM). 2004. “Mount Polley – supplication for permit amendment”. September. Email message from Chris Carr (MEM) to Ed Pittman (MEM). MEM Reference ID: MP00132.

Ministry of Energy and Mines (MEM). 2005a. “Report of Inspector of Mines Geotechnical.” February. MEM Reference ID: MP00172.

Ministry of Energy and Mines (MEM). 2005b. “Permit Review - TSF”. April. Email message from Chris Carr (MEM) to Ron Martel (MPMC). MEM Reference: MP00137.

Ministry of Energy and Mines (MEM). 2005c. “Report of Inspector of Mines Geotechnical.” October. MEM Reference ID: MP00174.

Ministry of Energy and Mines (MEM). 2006. “Report of Inspector of Mines Geotechnical.” September. MEM Reference ID: MP00175.

Ministry of Energy and Mines (MEM). 2007a. “Report of Inspector of Mines Geotechnical.” September. MEM Reference ID: MP00177.

Ministry of Energy and Mines (MEM). 2007b. “Mount Polley TSF Stage 6 Permit”. December. Email message from Chris Carr (MEM) to Ron Martel (MPMC). MEM Reference ID: MP00222.

Ministry of Energy and Mines (MEM). 2008. “Report of Inspector of Mines Geotechnical.” July. MEM Reference ID: MP00181.

Ministry of Energy and Mines (MEM). 2012a. “Mount Polley Mine – 2012 Tailings Construction.” June. Email message from George Warnock (MEM) to Diane Howe (MEM). MEM Reference ID: MP00221.

Ministry of Energy and Mines (MEM). 2012b. “Mount Polley Mine – 2012 Tailings Construction [Additional Raise].” September. Email message from George Warnock (MEM) to Diane Howe (MEM). MEM Reference ID: MP00223 and MP00224.

Ministry of Energy and Mines (MEM). 2012c. “Report of Geotechnical Inspector.” September. MEM Reference ID: MP00169.

Ministry of Energy and Mines (MEM). 2013a. “All geotechnical permit amendments for TSF.” August. MEM Reference ID: MP00215.

Ministry of Energy and Mines (MEM). 2013b. “RE: M-200 Amendment Approving Tailings Storage Facility Stage 9 Construction May 2013_draft”. September. Email message from Heather Narynski (MEM) to Diane Howe (MEM). MEM Reference ID: MP00186.

Ministry of Energy and Mines (MEM). 2013c. “Report of Geotechnical Inspector.” September. MEM Reference ID: MP00182.

Mount Polley Mining Corporation (MPMC). 2000. “Mt. Polley Tailings Storage Facility Application to Construct to Elevation 944 Metres Amendment to Permit M-200.” May. MEM Reference ID: MP00118.

Mount Polley Mining Corporation (MPMC). 2001a. “Mt. Polley Tailings Storage Facility Application to Construct to Elevation 945 Metres Amendment to Permit M-200”.” April. MEM Reference ID: MP00122.

Mount Polley Mining Corporation (MPMC). 2001b. “Geotechnical Inspection Report – Mount Polley Mine – April 25, 2001.” May. MEM Reference ID: MP00218.

Mount Polley Mining Corporation (MPMC). 2004. “Re: Referral of Mines Act Permit M-200 Amendment Application, Northeast Zone, Mount Polley Mining Corporation.” October. MEM Reference ID: MP00133.

Mount Polley Mining Corporation (MPMC). 2006. “Re: Mount Polley Geotechnical Inspection – August 30th, 2006.” October. MEM Reference ID: MP00149.

Mount Polley Mining Corporation (MPMC). 2007. “Re: Mount Polley Geotechnical Inspection – July 31st, 2006.” October. MEM Reference ID: MP00178.

Mount Polley Mining Corporation (MPMC). 2008. “Re: Your review of the 2007 Annual Inspection by KP.” June. MEM Reference ID: MP00180.

Mount Polley Mining Corporation (MPMC). 2010. Mount Polley Mine Tailings Storage Facility Operation, Maintenance and Surveillance Manual. 2010. MEM Reference ID: MP00207.

Mount Polley Mining Corporation (MPMC). 2011. “Permit Conditions Response.” January. MEM Reference ID: MP00052.

Mount Polley Mining Corporation (MPMC). 2013a. “2013 Site Wide Water Balance.” Excel workbook. May. MEM Reference ID: MP00228.

Mount Polley Mining Corporation (MPMC). 2013b. Mount Polley Mine Tailings Storage Facility Operation, Maintenance and Surveillance Manual. July, 2013. MEM Reference ID: MP00046.

Mount Polley Mining Corporation (MPMC). 2013c. “RE: Mt. Polley – Stage 9 Dam Raise Application.” July. E-mail message from Luke Moger (MPMC) to Heather Narynski (MEM). MEM Reference ID: MP00187.

Mount Polley Mining Corporation (MPMC). 2013d. “Tailings Management Framework (Under TSM Protocols)” July, 2013.

Mount Polley Mining Corporation (MPMC). 2014. “Mt. Polley Tailing Facility Field Review.” May. MEM Reference: MP00192.

Sladen, J.A., and W. Wrigley. 1983. "Chapter 8: A review of the geotechnical properties of lodgement till," in *Glacial Geology – An Introduction for Engineers and Earth Scientists*. Toronto: Pergamon. Nyles, N. Ed.

Stark, T. D., and M. Hussain. 2013. "Empirical Correlations: Drained Shear Strength for Slope Stability Analyses." *Journal of Geotechnical and Geoenvironmental Engineering.* 139, pp. 853-862. doi: 10.1061/(ASCE)GT.1943-5606.0000824

Stark, T.D., and H.T. Eid. 1994. "Drained Residual Strength of Cohesive Soils." *ASCE Journal of Geotechnical Engineering.* Vol. 120, No. 5, pp. 856-871.

Terzaghi, K., R. Peck and G. Mesri. 1996. *Soil Mechanics in Engineering Practice.* 3rd ed. New York: John Wiley & Sons, Inc.

Terzaghi, K. and R. Peck. 1948. *Soil Mechanics in Engineering Practice, 1st Edition*. New York: John Wiley.

United States Bureau of Reclamation. 1974. Section 81 of "*Design of Small Dams*", *Second Edition, Revised Reprint.* United States Government Printing Office

United States Bureau of Reclamation. 1987. "*Design of Small Dams*", *Third Edition,* United States Government Printing Office