BC Ministry of Energy and Mines
Mount Polley Tailings Dam Failure
Assessment of Failure Mechanism
May 1, 2015

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

Mr. George Warnock, P.Eng.
Manager, Geotechnical Engineering

Dear Mr. Warnock:

Mount Polley Tailings Dam Failure
Assessment of Failure Mechanism

This report presents our technical assessment and opinion on the mechanism of failure of the Mount Polley Tailings Dam.

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

Yours truly,

KLOHN CRIPPEN BERGER LTD.

[Signature]

Howard Plewes, M.Sc., P.Eng.
Project Manager

HP/HB:dl
BC Ministry of Energy and Mines

Mount Polley Tailings Dam Failure

Assessment of Failure Mechanism
EXECUTIVE SUMMARY

This report presents Klohn Crippen Berger’s (KCB) opinion on the mechanism of failure of the Mount Polley tailings dam on August 4, 2014. Our work was commissioned by the British Columbia Ministry of Energy and Mines (MEM) to assist in their investigations into the cause of the failure. This report documents our work, the progress of which was presented to the Ministry in meetings in October 2014 and January 2015.

The Mount Polley mine site is located 11 km from the town of Likely in the interior of British Columbia at N5819160 m and E595110 m. Tailings are retained by a U-shaped dam abutting a natural slope on the northwest side. The tailings dam comprises three embankments: the Main Embankment on the southeast side, South Embankment bounding the southwest side and Perimeter Embankment bounding the northeast side. The dam failed between Stations 4+110 and 4+350 at the highest section of the Perimeter Embankment. The breach released an estimated 1 17 million m³ of water (including interstitial water) and 8 million m³ of tailings solids which flowed into Polley Lake, Hazeltine Creek and 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 approximately 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 Independent Review Panel (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 30, 2015 (IRP 2015). Overall, this 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. 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 final trigger for the failure was the recent excavation at the toe in 2013 and raising of the embankment with the steep outer slope of 1.3H:1V.

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, directly from the Mount Polley Mine. This report relies on the factual data presented in those Progress Reports and the following additional work:

- review of dam construction history and records;

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1 Estimates taken from BCMEMPR website.
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.

The relevant findings from our failure assessment are described below.

**Properties of UGLU**

The dam failed by sliding on a foundation clay layer of glaciolacustrine origin, which lies approximately 10 m below the base of the embankment. This clay layer, termed the UGLU, is a high plastic, clay-rich varved lacustrine deposit ranging up to 2 m thick. The areal extent of the UGLU is largely confined to the immediate area of the failed dam.

The native UGLU is a “lightly over-consolidated” clay with a pre-consolidation pressure between 380 kPa and 420 kPa. In its native state prior to dam construction, the UGLU would have exhibited dilative response to shearing and its ultimate strength would be governed by the drained frictional strength. The weight of the 40 m high tailings dam subjected the UGLU to vertical stresses up to 800 kPa and substantial portions of the UGLU beneath the dam were loaded to stresses well above the pre-consolidation pressure. These loaded portions of the UGLU became “normally consolidated” and would have displayed a contractive response to shearing. The ultimate strength of normally consolidated clay is its undrained strength, which accounts for pore pressures developed during shearing. This change from lightly over-consolidated behavior to normally consolidated behavior occurred incrementally over time as the dam was raised. At the time of failure, the demarcation point between “lightly over-consolidated” and “normally consolidated” behavior occurred below the lower third of the dam slope.

The shear strength of the UGLU is controlled by the higher plastic zones within the clay layer. Accordingly, we estimate the peak drained strength of the UGLU as c’ = 0 kPa and φ’ = 22°, and the residual drained strength as φ’ = 12° - 14°. The similarity of these parameters to the shear strength of other clay soils reported by Stark et al. (2013) indicates that the UGLU is not a unique or special soil.

Under rapid loading and straining, the undrained strength of the UGLU is represented by Su = 0.22 (OCR)^0.8 σo’ where σo’ is the effective vertical confining stress and OCR is the overconsolidation ratio. This relationship is identical to the average relationship for homogeneous sedimentary clays recommended by Ladd (1991) and, again, indicates the UGLU is not unique.

The UGLU is also a strain-weakening material which loses appreciable strength when deformed past its peak strength, in both drained and undrained loading conditions. The strain-weakening nature of the UGLU was observed in direct shear tests, direct simple shear tests and undrained triaxial compression tests.

The piezometers installed in the UGLU after the failure found no evidence of high excess pore pressures related to the loading of the dam during construction. However, unloading and deformations in the UGLU during the failure and breach would have substantially changed the pore
pressure regime in the clay from that of the pre-failure state. Pore pressure analyses using the consolidation properties of the clay and rate of dam construction predicts that excess pore pressures up to 158 kPa may have existed at the time of dam failure.

On the other hand, evidence of “artesian” water pressures were encountered during post-failure site investigations in the permeable glaciofluvial deposits which are present about 5 m below the UGLU. These artesian water pressures were also predicted by pre-failure seepage analyses of the dam by KCB. These pressures reduce the consolidation stress and strength of the UGLU by about 5% to 15%.

**Analysis of Failure**

At the time of failure, the Factor of Safety (FoS) of the dam was calculated using limit equilibrium methods to be 1.27 using the peak drained strength of the UGLU and the pre-failure pore pressures estimated by seepage analyses. The FoS reduces 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 upthrust 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 initiate the subsequent dam breach.

A sliding plane in the UGLU is 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.

The existence of a pre-existing shear plane in the UGLU was considered as a possible factor in the failure. Samples retrieved from outside the failed dam were examined for the presence of shear planes or other distortions of the varved clay structure but none were 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.
TABLE OF CONTENTS

EXECUTIVE SUMMARY ........................................................................................................... i
1 INTRODUCTION .................................................................................................................. 1
  1.1 General ............................................................................................................................ 1
  1.2 Report Organization .......................................................................................................... 2
2 REVIEW OF HISTORICAL INFORMATION .................................................................... 3
  2.1 General ............................................................................................................................ 3
  2.2 Embankment Section ......................................................................................................... 3
  2.3 Construction History ......................................................................................................... 4
    2.3.1 Embankment Staging, Pond and Tailings Elevations ............................................... 4
    2.3.2 Foundation Preparation .............................................................................................. 4
    2.3.3 Embankment Zonation ............................................................................................... 6
  2.4 Regional Surficial Geology ............................................................................................... 8
  2.5 Regional Bedrock Geology ............................................................................................ 9
  2.6 Previous Site Investigations ........................................................................................... 9
  2.7 Instrumentation Records ................................................................................................ 10
    2.7.1 Piezometers ................................................................................................................ 11
    2.7.2 Inclinometers .............................................................................................................. 12
    2.7.3 Seepage Flows ............................................................................................................ 12
3 DESCRIPTION OF THE FAILURE ............................................................................... 13
  3.1 Timeline of Failure Event .............................................................................................. 13
  3.2 Pre-Failure Conditions .................................................................................................... 14
    3.2.1 Stage 9 Dam Raise Construction ................................................................................. 14
    3.2.2 Pond and Tailings Beach Levels ............................................................................... 14
  3.3 Post-Failure Conditions .................................................................................................. 15
  3.4 Interpretation of Field Observations ............................................................................. 16
4 POST-Failure SITE INVESTIGATIONS .................................................................... 17
  4.1 Site Investigations .......................................................................................................... 17
  4.2 Laboratory Testing ......................................................................................................... 18
  4.3 Observations of Failure Plane ....................................................................................... 19
5 SITE CONDITIONS .......................................................................................................... 21
  5.1 General .......................................................................................................................... 21
  5.2 Pre-Development Topography ..................................................................................... 21
  5.3 Soil Profile ...................................................................................................................... 22
  5.4 Glacial Tills .................................................................................................................... 22
    5.4.1 Upper Glacial Till (UGT) .......................................................................................... 23
### TABLE OF CONTENTS

*(continued)*

<table>
<thead>
<tr>
<th>Section</th>
<th>Title</th>
<th>Pages</th>
</tr>
</thead>
<tbody>
<tr>
<td>5.4.2</td>
<td>Middle Glacial Till (MGT)</td>
<td>24</td>
</tr>
<tr>
<td>5.4.3</td>
<td>Lower Glacial Till (LGT)</td>
<td>24</td>
</tr>
<tr>
<td>5.4.4</td>
<td>Material Properties for Failure Analysis</td>
<td>24</td>
</tr>
<tr>
<td>5.5</td>
<td>Glaciolacustrine Deposits</td>
<td>25</td>
</tr>
<tr>
<td>5.5.1</td>
<td>Upper Glaciolacustrine (UGLU)</td>
<td>26</td>
</tr>
<tr>
<td>5.5.2</td>
<td>Lower Glaciolacustrine (LGLU)</td>
<td>28</td>
</tr>
<tr>
<td>5.5.3</td>
<td>Material Properties for Failure Analysis</td>
<td>29</td>
</tr>
<tr>
<td>5.6</td>
<td>Glaciofluvial Deposits</td>
<td>29</td>
</tr>
<tr>
<td>5.7</td>
<td>Bedrock</td>
<td>30</td>
</tr>
<tr>
<td>5.7.1</td>
<td>Sedimentary Bedrock</td>
<td>30</td>
</tr>
<tr>
<td>5.7.2</td>
<td>Mafic-Igneous and Volcanic Bedrock</td>
<td>31</td>
</tr>
<tr>
<td>5.7.3</td>
<td>Material Properties for Failure Analysis</td>
<td>31</td>
</tr>
<tr>
<td>5.8</td>
<td>Glacial Till Core (Zone S)</td>
<td>32</td>
</tr>
<tr>
<td>5.9</td>
<td>Post-Failure Piezometer Readings</td>
<td>32</td>
</tr>
<tr>
<td>5.10</td>
<td>Post-Failure Inclinometer Readings</td>
<td>33</td>
</tr>
<tr>
<td>5.11</td>
<td>Evidence of Large Embankment Displacements</td>
<td>34</td>
</tr>
<tr>
<td>6</td>
<td>BACK-ANALYSIS OF EMBANKMENT FAILURE</td>
<td>35</td>
</tr>
<tr>
<td>6.1</td>
<td>General</td>
<td>35</td>
</tr>
<tr>
<td>6.2</td>
<td>Conceptual Sequence of Dam Failure</td>
<td>35</td>
</tr>
<tr>
<td>6.3</td>
<td>Analysis Sections</td>
<td>37</td>
</tr>
<tr>
<td>6.4</td>
<td>Material Properties for Analyses</td>
<td>37</td>
</tr>
<tr>
<td>6.5</td>
<td>Pore Pressure Conditions Prior to Failure</td>
<td>39</td>
</tr>
<tr>
<td>6.5.1</td>
<td>Steady-State Seepage Pore Pressures</td>
<td>39</td>
</tr>
<tr>
<td>6.5.2</td>
<td>Transient Pore Pressures in the UGLU</td>
<td>40</td>
</tr>
<tr>
<td>6.6</td>
<td>Limit Equilibrium Stability Analysis</td>
<td>40</td>
</tr>
<tr>
<td>6.6.1</td>
<td>General</td>
<td>40</td>
</tr>
<tr>
<td>6.6.2</td>
<td>Stability Results for Section C</td>
<td>40</td>
</tr>
<tr>
<td>6.6.3</td>
<td>Stability Results for Section D</td>
<td>41</td>
</tr>
<tr>
<td>6.7</td>
<td>Numerical Stress Analysis</td>
<td>41</td>
</tr>
<tr>
<td>6.7.1</td>
<td>General</td>
<td>41</td>
</tr>
<tr>
<td>6.7.2</td>
<td>Modelling of the UGLU</td>
<td>42</td>
</tr>
<tr>
<td>6.7.3</td>
<td>Global Factor of Safety</td>
<td>42</td>
</tr>
<tr>
<td>6.7.4</td>
<td>Local Factors of Safety and Yielding</td>
<td>43</td>
</tr>
<tr>
<td>6.7.5</td>
<td>Embankment Deformation Analysis</td>
<td>43</td>
</tr>
</tbody>
</table>
TABLE OF CONTENTS
(continued)

7 CONCLUSIONS.............................................................................................................................. 45
8 COMPARISON TO AZNALCOLLAR TAILINGS DAM FAILURE .................................................... 46
9 CLOSING......................................................................................................................................... 47

List of Tables

Table 2.1 Embankment Staging, Pond Elevation and Estimated Tailings Elevations at Failed Embankment Section ........................................................................................................................................ 5
Table 2.2 General Description of Fill Materials .................................................................................. 6
Table 3.1 Timeline of Failure .............................................................................................................. 13
Table 5.1 Description of Main Soils and Bedrock Units ...................................................................... 22
Table 5.2 Summary of Index Properties for UGT, MGT, and LGT.................................................... 23
Table 5.3 Summary of Material Properties for Failure Analysis – Glacial Till Units ....................... 25
Table 5.4 Summary of Index Properties for UGLU and LGLU ........................................................ 25
Table 5.5 Summary of Material Properties for Failure Analysis – Glaciolacustrine Units ............... 29
Table 5.6 Summary of Index Properties for Failure Analysis – Glaciofluvial Units ......................... 29
Table 5.7 Summary of Material Properties for Failure Analysis – Glaciofluvial Units .................... 30
Table 5.8 Summary of Material Properties for Failure Analysis – Bedrock Units ............................ 31
Table 5.9 Post-Failure Observations of Artesian Pressures in Glaciofluvial Foundation Deposits ........................................................................................................................................... 33
Table 6.1 Conceptual Sequence of Dam Failure .................................................................................. 36
Table 6.2 Summary of Material Properties for Failure Analysis – Fill Materials and Impounded Tailings ........................................................................................................................................... 38
Table 6.3 Factors of Safety from Limit Equilibrium Stability Analyses .......................................... 40
Table 8.1 Comparison of Mount Polley and Aznalcollar Tailings Dam Failures ......................... 46

List of Figures

Figure 1.1 Site Location
Figure 2.1 Tailings Storage Facility Pre-Failure Topography August 2013
Figure 2.2 Historical Aerial Photographs
Figure 2.3 Embankment Layout Aerial View from July 2014 with Contours from August 2013
Figure 2.4 Embankment Layout August 2013
Figure 2.5 Foundation Stripping and Preparation at Failure Area
Figure 2.6 As-Constructed Section of Perimeter Embankment Showing Staging and Fill Zonation
Figure 2.7 Regional Surficial Geology
Figure 2.8 Regional Bedrock Geology
Figure 2.9 Pre-Failure Site Investigations
TABLE OF CONTENTS
(continued)

Figure 2.10 Pre-Failure Instrumentation
Figure 2.11 Section G (AMEC) with Piezometer Data
Figure 2.12 Section G’ (KP) with Piezometer Data
Figure 2.13 Section D (AMEC) with Piezometer Data
Figure 2.14 Flow Records from Upstream Toe Drain
Figure 3.1 Post-Failure Topography on August 5, 2014
Figure 3.2 Aerial View of Dam Failure and Breach Zone
Figure 3.3 Zonation of Dam Failure and Breach Zone
Figure 3.4 Plan of Dam Failure and Breach Zone
Figure 3.5 Select Key Features from Post-Failure Mapping
Figure 3.6 Aerial View of Select Key Features from Post-Failure Mapping
Figure 3.7 View of Right Abutment of Breached Embankment
Figure 3.8 View of Left Abutment of Breached Embankment
Figure 3.9 Reference Photographs (1 of 3)
Figure 3.10 Reference Photographs (2 of 3)
Figure 3.11 Reference Photographs (3 of 3)
Figure 3.12 Hypothesized Sequence of Embankment Failure
Figure 4.1 Geophysical Survey Line Locations
Figure 4.2 Sonic Core Hole Locations
Figure 4.3 Test Pit, CPT, Vane Shear and Mud Rotary Sampling Hole Locations
Figure 4.4 Post-Failure Site Investigations by Others
Figure 4.5 Observations of Shear Plane in Zone S Till Core
Figure 4.6 Photographs of Upthrusted Glaciolacustrine Clay in TP14-01
Figure 5.1 Original Topography
Figure 5.2 Original Topography and Aerial Photograph from 1985
Figure 5.3 Perimeter Embankment Profile at Breach Area
Figure 5.4 Original Ground Elevation Encountered in Sonic Core Holes
Figure 5.5 Representative Photographs of UGT
Figure 5.6 Standard Penetration Test Blowcounts in UGT
Figure 5.7 CPT Shear Wave Velocity Measurements in Glacial Tills
Figure 5.8 Top Elevation of UGLU Encountered in Sonic Core Holes
Figure 5.9 Thickness of UGLU Encountered in Sonic Core Holes
Figure 5.10 Representative Photographs of UGLU
Figure 5.11 Maximum Liquid Limit of UGLU in Sonic Core Holes
Figure 5.12 Maximum Water Content of UGLU in Sonic Core Holes
Figure 5.13 Geologic Section C Water Content Data
Figure 5.14 Geologic Section D Water Content Data
Figure 5.15 Summary of Consolidation Test Results on UGLU and LGLU
Figure 5.16 Undrained Shear Strength (Su) N<sub>k</sub> from CPT Soundings in UGLU and LGLU
TABLE OF CONTENTS  
(continued)

Figure 5.17  Undrained Shear Strength (Peak and Remolded) from Vane Shear Tests in UGLU
Figure 5.18  Geologic Section C Undrained Shear Strength Data
Figure 5.19  Geologic Section D Undrained Shear Strength Data
Figure 5.20  Peak and Residual Shear Strengths of Thin-Walled Piston, Sonic Core, and Block Specimens from Direct Shear Tests on UGLU Unit
Figure 5.21  Stress Paths from Triaxial Compression Tests on UGLU Unit
Figure 5.22  Summary of Monotonic Direct Simple Shear Tests on UGLU Unit and Proposed Undrained Shear Strength Functions
Figure 5.23  Normalized Stress-Strain from Monotonic Direct Simple Shear Tests on UGLU
Figure 5.24  Representative Photographs of LGLU
Figure 5.25  Top Elevation of LGLU Encountered in Sonic Core Holes
Figure 5.26  Empirical Strength Relationships for Clays
Figure 5.27  Top Elevation of Bedrock Encountered in Sonic Core Holes
Figure 5.28  Geological Section F with Sonic Core Holes and Geophysical Survey Data
Figure 5.29  Post-Failure Piezometer Location Plan
Figure 5.30  Section G (AMEC) with Piezometer Data
Figure 5.31  Section C (KCB) with Piezometer Data from Section G’ (Knight Piesold)
Figure 5.32  Section D (AMEC) with Piezometer Data
Figure 5.33  Seepage Gradients in Foundation Soil and Bedrock
Figure 5.34  2014 Inclinometers Showing Movements Measured in UGLU
Figure 5.35  Inclinometer Reading SH14-03
Figure 5.36  Inclinometer Reading SH14-04
Figure 5.37  Inclinometer Reading SH14-06
Figure 5.38  Inclinometer Reading SH14-09
Figure 5.39  Inclinometer Reading SH14-16
Figure 5.40  Evidence of Large Embankment Displacements
Figure 6.1  Location of Sections C and D Use in Analyses
Figure 6.2  Section C Pre-Failure Geometry and Zonation
Figure 6.3  Section D Post-Failure Geometry and Zonation
Figure 6.4  Steady-State Seepage Analyses Results for Pre-Failure Conditions
Figure 6.5  Predicted Excess Pore Pressure at the Middle of UGLU
Figure 6.6  Slope Stability for Stages 1 to 5
Figure 6.7  Shear Stress-Strain Behaviour of UGLU Under Drained and Undrained Conditions
Figure 6.8  Global Factors of Safety for Peak Drained and Undrained Shear Strengths in UGLU
Figure 6.9  Mobilized Stresses and FoS Against Yielding at the End of Stage 6B and Stage 9
Figure 6.10  Deformed Shape of the Embankment and Shear Strain in UGLU at Maximum Horizontal Displacements of 0.3 m and 6 m
TABLE OF CONTENTS
(continued)

List of Appendices

Appendix I  As-Built Records and Reference Information
Appendix II  Site Investigation References
Appendix III  Steady State Seepage Analyses
Appendix IV  Limit Equilibrium Stability Analyses
Appendix V  Numerical Stress Analyses
Appendix VI  Pore Pressure Analysis of UGLU
1 INTRODUCTION

1.1 General

This report presents Klohn Crippen Berger’s (KCB) opinion on the mechanism of failure of the Mount Polley tailings dam on August 4, 2014. This work was commissioned by the British Columbia Ministry of Energy and Mines (MEM) to assist in their investigations into the cause of the failure. This report documents our work, the progress of which was presented to the Ministry in meetings in October 2014 and January 2015.

The Mount Polley mine site is 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 comprises three embankments: the Main Embankment on the southeast side, South Embankment bounding the southwest side and Perimeter Embankment bounding the northeast side. The dam failed between Stations 4+110 and 4+350 at the highest section of the Perimeter Embankment. The breach released an estimated 17 million m³ of water (including interstitial water) and 8 million m³ of tailings solids which flowed into Polley Lake, Hazeltine Creek and Quesnel Lake.

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2 Estimates taken from BCMEMPR website.
• 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.

1.2 Report Organization

Section 2 provides a review of historical information on the site conditions and construction of the Perimeter Embankment in the vicinity of the breach. A selection of supporting factual data in the form of reports and drawings is contained in Appendix I.

Section 3 describes the failure including a chronology of the event and pertinent post-failure morphological features.

Section 4 describes the site investigations and laboratory testing conducted by KCB, which was included in Progress Report Nos. 1 through 4. Selected information from this work is contained in Appendix II.

Section 5 gives our interpretation of the soil profile and geotechnical properties at the breach area.

Section 6 presents our analysis of the probable seepage and pore pressure conditions in the dam prior to failure. The failure mechanism of the dam is then assessed using limit equilibrium and numerical modelling methods.

Conclusions on the failure mechanism are discussed in Section 7.

Section 8 compares the features of the Mount Polley dam failure with the Aznalcollar tailings dam failure that occurred in Spain in 1998.
2 REVIEW OF HISTORICAL INFORMATION

2.1 General

The Tailings Storage Facility (TSF) is located 3 km southeast of the mill site. The TSF is approximately 1.8 km long by 1.6 km wide, and covers an area of about 304 ha. The pre-failure configuration of the TSF from the most recent Lidar survey on August 21, 2013 is shown in Figure 2.1. A more recent aerial view in July 2014 is shown in Figure 2.3, overlain by topographic contours from August 2013.

TSF construction started in May 1996 at the Main Embankment. Construction of the Perimeter Embankment started in December 1996. Excluding a mill shutdown from mid-October 2001 to March 2005, it was raised in nine main stages ending in August 2014. Figure 2.2 presents historic aerial photographs of the TSF for 1985, 1998, 2005, and 2010.

The following sections describe the Perimeter Embankment section, construction history including material zonation, regional geology, and instrumentation records prior to failure. Appendix I provides select supporting information that was used in preparation of this section.

2.2 Embankment Section

The plan layout of the Perimeter Embankment in the failure area is shown in Figure 2.4, including fill zonation and location of drainage systems. Figure 2.6 presents the as-built section of the failed embankment based on our compilation of design and as-built records.

The zonation of the Perimeter Embankment includes a compacted till core (Zone S) supported on the downstream side by a rockfill shell (Zone C), and upstream by a random fill zone with varying material types throughout operation (Zone U). Fine and coarse filter materials separate the core and the downstream shell (Zone F and Zone T).

An Upstream Toe Drain, installed within the upstream fill (Zone U) during Stage 5, comprises a 250 mm diameter perforated pipe embedded in a 1 m x 1 m trench backfilled with granular drain material. The drain extends between Stations 3+000 to 4+575 with the pipe invert at El. 946.3 m. The drain passes underneath the embankment at Station 4+575 (within a concrete encased steel pipe) and discharges into a ditch that conveys flows to the Perimeter Embankment Seepage Collection Pond (see Figure 2.4).

The most recent Stage 8 and 9 raises were constructed following the centerline method. Prior raises in Stages 2 to 7 were conducted following a modified upstream construction method.

Construction of the final Stage 9 raise to a crest elevation of 970 m commenced by end of April 2013 and was progressing under MEM Permit No M-200 in accordance with the design completed by AMEC in April 11, 2013 (MP00045). The raise started at El. 963.5 m which represents the end of Stages 8/8A. At the time of the failure, Zones U, S and C had been raised to nominal elevations of 969 m, 970 m and 969 m, respectively.
2.3 Construction History

2.3.1 Embankment Staging, Pond and Tailings Elevations

Table 2.1 summarizes the timeline of the embankment raises and the pond and tailings elevations for the Perimeter Embankment. Reference information for the timeline is included in Appendix I-B.

2.3.2 Foundation Preparation

Figure 2.5 summarizes the available foundation preparation records in the vicinity of the failure breach. Foundation preparation comprised an initial clearing and stripping of vegetation followed by specific preparation works in eight individual foundation areas. Reference information is included in Appendix I-C. Main observations are:

- The ground was generally stripped to native glacial till.
- Due to wet and soft ground conditions, filter fabric was laid over the till in a portion of Area 2. This explains fragments of non-woven geotextile that were encountered during the post-failure site investigation in drill holes SH14-03 and SH14-08 at the contact between the embankment fill and the native ground.
- A near surface clay deposit was encountered along the outer quarter of the Stage 2B haul road between Stations 4+275 and 4+200 (shown as Area 3 in Figure 2.5). The clay was at least 0.8 m thick and deemed a local deposit. At the same period, “pumping” was observed during construction in a “low swampy area” between Stations 4+195 and 4+250. Portions of the local clay deposit were left in place. Accordingly, during the post-failure site investigation, a 0.5 m nominal thickness of sandy clay was encountered in drill holes SH14-08 and SH14-09 immediately below the base of the embankment. The non-woven geotextile found in SH14-08 was at the contact between the overlying embankment fill and the clay.
- Stage 9 foundation preparation in Area 6 was approved for fill placement except for a section between Stations 4+100 and 4+300. Interviews with mine personnel indicated that a 2 m deep excavation at the embankment toe was conducted in this area in November 2013 in preparation for future construction of a toe buttress. This excavation was reported to be left open at the time of embankment failure. The impact of this open excavation is considered in our failure analyses.
Table 2.1  Embankment Staging, Pond Elevation and Estimated Tailings Elevations at Failed Embankment Section

<table>
<thead>
<tr>
<th>Stage</th>
<th>Raise Methodology</th>
<th>As-Constructed Crest Elevation (m)</th>
<th>Construction Period</th>
<th>Pond and Tailings Elevation</th>
<th>Reference Document</th>
<th>Design Engineer</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Upstream Zone</td>
<td>Core</td>
<td>Downstream Shell</td>
<td>From</td>
<td>To</td>
</tr>
<tr>
<td>1A</td>
<td></td>
<td>-</td>
<td>927.0</td>
<td>-</td>
<td>Dec 12, 1996</td>
<td>End of 1996</td>
</tr>
<tr>
<td>1B</td>
<td></td>
<td>934.0</td>
<td>934.0</td>
<td>934.0</td>
<td>Dec 12, 1996</td>
<td>March 17, 1997</td>
</tr>
<tr>
<td>2A</td>
<td></td>
<td>936.0</td>
<td>936.0</td>
<td>934.0</td>
<td>Feb 6, 1998</td>
<td>May 15, 1998</td>
</tr>
<tr>
<td>2B</td>
<td></td>
<td>936.0</td>
<td>936.7</td>
<td>934.0</td>
<td>Sep 23, 1998</td>
<td>Dec 22, 1998</td>
</tr>
<tr>
<td>2C</td>
<td></td>
<td>941.0</td>
<td>941.0</td>
<td>941.0</td>
<td>Apr 1999</td>
<td>Feb 12, 2000</td>
</tr>
<tr>
<td>3A</td>
<td>Modified Upstream Method</td>
<td>941.3</td>
<td>941.3</td>
<td>941.0</td>
<td>Nov 2000</td>
<td>Mar 2001</td>
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<tr>
<td>3B</td>
<td></td>
<td>942.5</td>
<td>942.5</td>
<td>941.0</td>
<td>May 2001</td>
<td>End Aug 2001</td>
</tr>
<tr>
<td>3C</td>
<td></td>
<td>944.0</td>
<td>944.0</td>
<td>944.0</td>
<td>Aug 2004</td>
<td>Mar 2005</td>
</tr>
<tr>
<td>4</td>
<td></td>
<td>948.0</td>
<td>948.0</td>
<td>944.0</td>
<td>May 2005</td>
<td>Oct 2006</td>
</tr>
<tr>
<td>5</td>
<td></td>
<td>951.0</td>
<td>951.0</td>
<td>950.5</td>
<td>Nov 2006</td>
<td>Nov 2007</td>
</tr>
<tr>
<td>6A</td>
<td></td>
<td>952.3</td>
<td>954.0</td>
<td>952.3</td>
<td>May 2008</td>
<td>Oct 2008</td>
</tr>
<tr>
<td>6B</td>
<td></td>
<td>958.5</td>
<td>958.0</td>
<td>957.3</td>
<td>Oct 2009</td>
<td>Aug 2010</td>
</tr>
<tr>
<td>7</td>
<td></td>
<td>960.5</td>
<td>960.1</td>
<td>959.9</td>
<td>Jun 13, 2011</td>
<td>Sep 21, 2011</td>
</tr>
<tr>
<td>8/8A</td>
<td></td>
<td>963.5</td>
<td>963.5</td>
<td>962.7</td>
<td>May 30, 2012</td>
<td>Oct 26, 2012</td>
</tr>
<tr>
<td>9</td>
<td>Centerline</td>
<td>964.5</td>
<td>967.0</td>
<td>966.1</td>
<td>End of April 2013</td>
<td>End of April 2013</td>
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<tr>
<td></td>
<td></td>
<td>969.0</td>
<td>970.02</td>
<td>969.0</td>
<td>Spring 2014</td>
<td>August 2014</td>
</tr>
</tbody>
</table>

Notes:
1. This construction period is for the Perimeter Embankment as indicated in references. Other construction periods are for the entire TSF.
2. Crest elevation based on cross-section from the August 5, 2014 LiDAR and from surveyed site investigation drill holes and CPT sounding at the embankment crest (i.e., El. 969.9 m at SH14-20, El. 968.9 m at SH14-21, and El. 971 m at CPT14-21).
3. Some differences were found when comparing pond levels in as-built reports and pond levels presented in historical piezometer records from AMEC 2013 (MP00044). Differences are for Stages 1 and 2 (1 m to 5 m higher in AMEC’s plot) and for Stages 3C, 4, and 6B (1 m to 2 m higher in plot).
4. Tailings elevation was not available, thus it was estimated based on tailings beach width from as-built records and a beach slope of 0.5%.
5. Refer to Stage 3 construction report. Tailings built higher than crest, although tailings assumed at crest elevation.
7. Tailings elevation inferred from 2014 LiDAR for areas east and west of the breach area.
2.3.3 Embankment Zonation

The embankment was constructed using the nine main fill zones listed in Table 2.2. Figure 2.6 shows the re-constructed pre-failure section of the embankment (Section C) and the material zones. The re-constructed section is based on reference information included in Appendix I-D and reference photographs from construction records included in Appendix I-E.

Table 2.2 General Description of Fill Materials

<table>
<thead>
<tr>
<th>Zone</th>
<th>Material Type</th>
<th>Function</th>
</tr>
</thead>
<tbody>
<tr>
<td>S</td>
<td>Glacial till</td>
<td>Till Core</td>
</tr>
<tr>
<td>B</td>
<td>Glacial till, glaciolacustrine or granular material</td>
<td>Fill Zone</td>
</tr>
<tr>
<td>C</td>
<td>Rockfill</td>
<td>Downstream Shell Zone</td>
</tr>
<tr>
<td>T</td>
<td>Fine Rockfill</td>
<td>Transition Zone/ Confining berm/Haul Road</td>
</tr>
<tr>
<td>F</td>
<td>Sand and Gravel</td>
<td>Chimney Drain, Longitudinal/Outlet Drain</td>
</tr>
<tr>
<td>G</td>
<td>Drain Gravel</td>
<td>Foundation/Longitudinal/Outlet drain</td>
</tr>
<tr>
<td>CBL</td>
<td>Random/Select Rockfill</td>
<td>Base Layer for Upstream Fill</td>
</tr>
<tr>
<td>CS</td>
<td>Cycloned Sand</td>
<td>Hydraulically or Mechanically Placed as Upstream Fill</td>
</tr>
<tr>
<td>U</td>
<td>Random fill and Tailings Sand</td>
<td>Upstream Fill</td>
</tr>
</tbody>
</table>

Fine-grained glacial till for Zones B and S were sourced from borrow areas around the perimeter of the TSF or within the TSF impoundment (during early years of construction). Zone C rockfill materials were quarried from rock exposures or mine pits. Filter materials for Zones F and T were processed from the rockfill by crushing and/or screening.

Technical specifications for the fill materials are given in the design reports and are summarized in Appendix I. The following summarizes the description of the fill materials as-placed in the embankment as described in the as-built records and reports.

**Glacial Till Core (Zone S and Zone B)**

- Zone S was reported to be glacial till comprising a medium to low plastic silty sand to sandy silt material, with some gravel, and trace to some clay. During Stage 8 only, Zone S was reported to be a mixture of glacial till and glaciolacustrine clay from the Perimeter Embankment borrow. Zone S was generally placed at or above the optimum water content\(^3\). The median compaction was generally greater than 97% of the maximum Standard Proctor dry density (see Figure I-D-2 in Appendix I-D).

- Zone B was only placed in Stages 1 and 2. It allows for a coarser gradation than Zone S, but gradation records from placed materials showed similar gradation ranges. Zone B had lower compaction requirements and testing frequencies than Zone S. Compaction was greater than 95% of the maximum Standard Proctor dry density. A coarse bearing layer (CBL) was placed below Zone B in Stages 1 and 2 to improve local trafficability prior to the fill placement.

\(^3\) Optimum water content and maximum dry density obtained from Standard Proctor Tests.
- Zone S till core width was reported to be out of compliance during Stage 8 construction, when the core width was less than the specified 5 m. During Stage 9, a minor repair to the core was needed due to a minor upstream slump (0.5 m width and 0.3 m deep repair).

Upstream Support for the Till Core (Zone U, Zone CS, and Zone CBL)

- The material used in the support zone upstream of the till core varied throughout operation and has a high variability in gradation and material characteristics. Zone U generally comprised gravelly sand fills with variable fines content (3% to 62%) interlayered with tailings placed hydraulically in cells (silty sand with fines content of 10% to 45%). Mine waste rock was used for part of Stage 6B as shown in Figure 2.6. It was used in other Stages as well.
- Rockfill layers (Zone CBL) were used to gain support over the tailings beach for fill placement during Stages 2C and 4.
- Cycloned sand (Zone CS) comprising uniform silty sand with fines content of 21% to 34% was used as upstream support during Stage 3A/3B.
- As-built construction records indicated that till core material that failed compaction requirements was also dumped towards the impoundment since Stage 4. Photographs of the contact between Zone S and Zone U showed uncompacted till/random fill materials4 (see Appendix I-D).

Downstream Support for the Till Core (Zone C)

- Zone C is rockfill comprising fine to coarse gravel with trace to some sand, some cobbles and trace of boulders. As-built records report up to 10% fines. Compaction was applied by a vibratory smooth drum roller and by the passage of trucks and other construction equipment. The lift thickness was specified to be 1 m, but construction records indicated it was placed up to 2 m thick.
- The downstream slope was steepened above El. 944 m from 2H:1V to 1.4H:1V during Stage 5 construction. Also, starting from Stage 6, the dam slope was steepened to maintain a wider embankment crest to allow for passage of haul trucks. Dumping of rockfill onto the dam slope resulted in an uncompacted outer slope at the angle of repose (1.3H:1V) for Stages 7 to 9.

Filters (Zone T and Zone F)

- Zone F (fine filter) comprises sand and gravel and Zone T (coarse filter) comprises gravel with some sand and trace cobbles. Fines content typically ranged from 0% to 15% for both materials. As-built records for Zone F show that the fine end of the particle size distributions were frequently coarser than the specified D15 gradation limit. Potential segregation and filter compatibility was assessed visually during construction.

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4 Lower tip resistance and undrained shear strength was subsequently observed by KCB in CPT14-21 results when passing through the inferred contact between Zone S and Zone U.
Impounded Tailings

- Properties for the impoundment tailings were inferred from Cone Penetration Testing (CPT) soundings conducted in 1999 during Stage 2C construction (included in Appendix I-F). A total of six CPT holes were completed between Stations 4+000 and 4+600. Up to 5 m of tailings was encountered. Sounding locations are shown on Figure 2.9.

- Tailings consisted of loose to compact (equivalent SPT (N₁)₆₀ of 10) sandy silt, interlayered with loose silt lenses ((N₁)₆₀ of 5) every 0.5 m to 1.0 m. Shear wave velocities were low, ranging from 100 m/s to 125 m/s.

- Soft “sensitive fines” were observed at 1.5 m to 2.0 m intervals, which may correspond to slimes deposited between discharge locations. Slimes showed longer dissipation times and lower coefficients of horizontal consolidation than the coarser tailings layers.

- Measured pore water pressures showed hydrostatic conditions for the tested depths. Material properties for our analyses in Section 6 were inferred from dissipation testing, shear wave velocities, and interpreted data at CPT99-13 and CPT99-14 which are located immediately upstream of the filled dam.

2.4 Regional Surficial Geology

The Mount Polley site is located in the Interior Plateau of British Columbia, an area of relatively subdued topographic relief and lower elevation between the Coastal and Cariboo Mountains. During glaciation, glaciers flowed from the mountains to merge over the Interior Plateau (Clague 1988, Clague 1991). There is evidence to suggest that at least two glaciations occurred within these valleys: the penultimate glaciation (>35,000 BP⁵), and the Fraser Glaciation (10,000 BP to 25,000 BP).

Figure 2.7 shows the regional surficial geology surrounding the TSF. The TSF sits on morainal blankets of variable thickness interpreted as lodgement till. Surficial soils are attributed to the later Fraser Glaciation. Mapping of streamlined subglacial bedforms show that the direction of glacial movement was in the northwest direction.

The till blanket comprises poorly sorted, moderate to well compacted, clayey to silty diamicton. Less compact, boulder rich tills with a sandy matrix are found at higher elevations and classified as ablation till.

Glaciolacustrine deposits (interbedded sand, silt, and clay) are reported to underlie terraces within major river valleys in the vicinity of Mount Polley. Glaciofluvial sediments associated with melt-water channels or drainage courses (typically moderately sorted, weak to moderately compacted, cobble or boulder cobble gravel with a sand matrix) are also present in major valleys as terraces (up to 70 m above river levels) or as irregular deposits at higher elevations on valley slopes or plateaus.

Glaciolacustrine and glaciofluvial sediments were deposited in the valleys during glacial advance and retreat. According to Clague (1991), the glaciolacustrine deposits associated with the penultimate

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⁵ Conventional Radiocarbon Age (BP)
glacial retreat are described as mainly “massive and stratified silt, sand, gravel, and minor diamicton”. Structures within these sediments include folds, faults and sedimentary dykes that are both syndepositional (occurred during deposition) and post-depositional. A 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”.

The glacial till and glaciolacustrine deposits encountered in the post-failure site investigations, as discussed in Section 5, are consistent with the published regional surficial geology.

### 2.5 Regional Bedrock Geology

Figure 2.8 shows a regional bedrock geology map for the TSF area. The Mount Polley mine is located in an alkali intrusive complex in the Quesnel Trough, a 35 km wide northwest trending volcanic sedimentary belt of regional extent. The Quesnel Trough constitutes a regional synclinal structure formed within a Triassic continent-margin basin, infilled with Triassic sediments and then Triassic to Jurassic volcanic rocks.

A major portion of the TSF sits on what is described as quaternary thick alluvium (Qal), with no description on the underlying bedrock type. The bedrock outcrops in the hill slope confining the west side of the TSF present jointing, shearing, and vertical dikes mainly formed of intrusions of melanocratic syenite rock from the late Triassic.

A 1.5 km long fault is inferred to cross the right abutment of the South Embankment and extend along the western edge of the impoundment crossing the left abutment of the Perimeter Embankment at approximately Station 4+500 (see Detail 1 in Figure 2.8). The rock in this fault zone comprises mainly pseudoleucite syenite in the form of vertical dikes and pyroxene/hornblende-biotite monzonite at the Perimeter Embankment.

### 2.6 Previous Site Investigations

Campaigns of site investigations were conducted from 1989 to 2012 at the TSF. Figure 2.9 presents the plan of pre-failure site investigations conducted at or near the failed embankment. The investigations comprised the following:

- In 1989, MP89-231 was drilled below the failed embankment to 122 m depth by tricone rotary drilling. The overburden was characterized from drill cuttings, which is usually imprecise in quantifying soil properties or identifying thin layers. Hence, no glaciolacustrine deposits were identified in MP89-231. The water table was found at surface.

- From 1995 and 1999, a total of 4 test pits and 17 drill holes were completed in the vicinity, with 2 test pits and 11 drill holes below the failed embankment. These investigations were restricted to 5 m depth, except for DH99-132 which was 7.5 m deep. A clay layer, up to 2 m thickness, was encountered near surface overlying stiff to very stiff glacial till. The areal extents of this layer was confined to the center of the failed embankment area, at DH99-114
to DH99-119 as shown in Figure 2.9. This correlates with the shallow clay deposit encountered during the foundation preparation as discussed in Section 2.3.2.

- In 1996, a 61 m deep groundwater monitoring well (GW96-1A/1B) was installed downstream of the Perimeter Embankment Seepage Collection Pond, approximately 100 m downstream and east of the failed embankment. A soft glaciolacustrine deposit was encountered from El. 919 m to El. 923 m, with an SPT blow count of 6. Artesian conditions were also encountered in a permeable glaciofluvial unit from El. 896 m to El. 885 m.

- As discussed earlier, five CPT’s were completed in 1999 through the upstream tailings beach to 8 m below tailings level. Three CPT’s are upstream of the failure area. CPT99-14 reached 7.9 m below the tailings surface and encountered soils classified as silty clay. Tailings thickness in the area was about 5 m, indicating this CPT sounding reached about 3 m into the underlying foundation.

- In 2011, two sonic core holes (VW11-10 and VW11-11) were drilled at the embankment toe. VW11-11, located west of the failed dam, encountered glaciolacustrine deposits at El. 933 m to El. 929 m and VW11-10, located east, encountered a thin layer of glaciolacustrine at El. 917 m. Vibrating wire piezometers were installed in foundation soils in both holes.

- Also in 2011, one inclinometer (SI11-04) was installed downstream of the embankment toe, approximately 150 m east of the failed embankment. A stiff to hard glaciolacustrine layer was encountered at El. 914.9 m. In 2012, a replacement inclinometer (SI12-01) was installed adjacent to SI11-04 due to a suspected malfunction in SI11-04. This is discussed in Section 2.7.2.

In summary, detailed information on the soil profile below the failed embankment was restricted to a 7.5 m depth below the ground surface. The investigations encountered glacial till overlain in local areas by a deposit of glaciolacustrine clay. Deeper investigations outside the failure area revealed the presence of three other glaciolacustrine deposits at different elevations and depths, and the occurrence of artesian pressures within glaciofluvial deposits at depth.

### 2.7 Instrumentation Records

Figure 2.10 presents the pre-failure instrumentation plan at Corner 1 of the Perimeter Embankment. Instrumentation includes: 17 vibrating wire piezometers, 2 slope inclinometers and flow measurements from the Upstream Toe Drain. Reference information for the instrumentation is included in Appendix I-G.

Monitoring frequency varied throughout the operation of the TSF. Frequency was initially every two weeks for inclinometers and piezometers, and was reduced to once a month after 2012 (MP00217). Seepage measurements from the Upstream Toe Drain were typically recorded 3 to 8 times per year starting in 2007.
2.7.1 Piezometers

The piezometers were either installed during construction in the tailings impoundment and fill materials, or installed in foundation soils during site investigations. Twelve of the 17 vibrating wire piezometers were functioning in 2014. The functioning piezometers are located along three sections as shown from Figure 2.10 to Figure 2.13:

- Section G (AMEC): Two in foundation soils (glacial till and glaciolacustrine);
- Sections G' (KP): One installed in the upstream tailings, one in the upstream fill (Zone U), and one in the till core (Zone S); and
- Section D (KP and AMEC): Three in foundation soils (glacial till, glaciolacustrine, and glaciofluvial), two in the filter materials, one in the till core, and one in the upstream tailings.

Salient observations from the piezometers are as follows:

- Piezometers G2 and D04 were installed in the till core (Zone S) at a nominal elevation of 948 m. Pore pressures in G2 began to increase as the pond elevation rose above about El. 960 m, indicating saturation of the core. No pore pressure response was observed in D04 in response to the pond rise. However, the pore pressure in piezometer D03 located downstream in Filter Zone F increased as the pond level rose above 955 m, which is an indication of high seepage through the core. Other explanations are that the readings for D04 and D03 are interchanged and misreported or D03 is actually in the core.

- Piezometers installed in the upstream tailings (G3 and D05) and in Zone U (G1) all responded as the upstream pond level rose above the elevation of the piezometer tips. However, the piezometric levels were typically 5 m to 10 m below the adjacent pond level, reflecting the influence of the Upstream Toe Drain.

- Piezometers installed in drain and fill materials downstream of the core showed no response to embankment raising or pond level rise. The exception is the anomalous response in D03 which is noted above.

- Only two piezometers at Section D outside the failure area, D01 and D02, are installed below the embankment. Both were installed in the upper glacial till deposits. Neither piezometer showed any transient increases in pore pressures caused by loading during periods of embankment raising. D01 gradually increased by about 5 m over time, likely in response to seepage pressures increasing with the rise in the upstream tailings pond.

- Piezometers D6, D7, G4 and G5 are located outside the failure zone and beyond the downstream toe of the embankment. G4 and G5 indicate a water table near the ground surface and a downward gradient. Piezometric elevations at D6 and D7 are about 12 m below the ground surface and also indicate a downward gradient.
2.7.2 **Inclinometers**

Only one inclinometer (SI11-04) was installed in 2011 at the Perimeter Embankment. It is located approximately 150 m southeast of the dam failure and 20 m downstream of the embankment toe. As such, this inclinometer would not have given prior warning of the dam failure.

The 2012 as-built report (MP00217) indicated the following: “**In late 2012, readings from an inclinometer located downstream of the Perimeter embankment (SI11-04) showed compression failure deformation consistent with settlement at depths from ground surface to 15 m below ground surface. AMEC recommended that additional instrumentation be installed, as the SI11-04 would likely cease functioning due to the deformation.**” Based on the soil profile at SI11-04, this deformation occurred in the upper glacial till unit. No significant displacements were observed in glaciolacustrine layer encountered at El. 914.9 m, which is described as very stiff to hard. The elevation and characteristics of this unit are similar to the Lower Glaciolacustrine Layer (LGLU) discussed later in this report.

An inclinometer casing with compression fittings (SI12-01) was subsequently installed to replace SI11-04 and set 42 m below ground surface. Readings for SI12-01 began on March 12, 2013 and continued until August 13, 2014. No preferential displacement trends or shear planes were observed in SI12-01. Cumulative displacement was in the order of 5 mm for a period of 10 months.

Installation details and readings from SI11-04 and SI12-01 are included in Appendix I-G. Only a handwritten field log and simplified log included in the inclinometer reading plots were available at the time this report was completed.

2.7.3 **Seepage Flows**

The flow rates from the Upstream Toe Drain were measured in the ditch located downstream of the pipe outlet. Flow records extracted from as-built reports are included in Appendix I-G and shown in Figure 2.13. Relevant observations include:

- Seepage rates from the Upstream Toe Drain increased with time as the tailings pond rose. The seepage rate on July 2014 just prior to the dam failure was 23.4 L/s at a tailings pond elevation of 966.3 m.

- A temporary “spike” in seepage rate to 91 L/s was reported for April 2013. The tailings pond during the spike was rising to El. 962 m, when the embankment crest was at El. 965 m. The seepage rate diminishes after the pond surpasses El. 962 m. The increased seepage may be related to the higher permeability of the Zone U fills placed in Stages 6B and 7, which comprised mainly rockfill. The seepage dropped as this rockfill was covered by deposited tailings.

Flow measurements from the Outlet Drains (see Figure 2.4) were reported from July 2000 to November 2006 and yielded flow rate of less than 1 L/s.
3 DESCRIPTION OF THE FAILURE

3.1 Timeline of Failure Event

The Perimeter Embankment failed on August 4, 2014 east of Corner 1 between Stations 4+110 and 4+350. The timeline of the failure event is summarized in Table 3.1 based on mine staff interviews conducted post-failure. These are reported in detail in the Inquiry Report (KCB 2015c).

Table 3.1 indicates the dam failed within a period of 2 hours to 3 hours between 10:35 pm on August 3, 2014 and 12:10 am August 4, 2014. The rapid rise in the perimeter sump and the loss of power at the mill indicates that the subsequent breach of the dam initiated shortly after and was fully advanced by 1:08 am on August 4, 2014. Subsequent release of water and tailings slurry continued at full force until 4:00 pm on August 4, 2014.

The rapid timeline of the failure events prevented the possibility of any remedial actions to repair the dam or breach. It leads to the conclusion that a “brittle” failure mechanism was activated during the failure, as no prior evidence of distress in the dam was visually evident to mine staff.

Table 3.1 Timeline of Failure

<table>
<thead>
<tr>
<th>Date</th>
<th>Time</th>
<th>Observation</th>
</tr>
</thead>
<tbody>
<tr>
<td>August 3, 2014</td>
<td>10:35 pm</td>
<td>The No. 2 pump was turned on in the Perimeter Embankment Seepage Collection Pond sump (normal procedure). An employee drove along the crest of dam, near the breach.</td>
</tr>
<tr>
<td>August 3/4, 2014</td>
<td>11:40 pm to 12:10 am</td>
<td>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.</td>
</tr>
<tr>
<td>August 4, 2014</td>
<td>12:10 am</td>
<td>Water level starts to modestly increase – suggesting that failure had occurred and water was overflowing the crest of the dam. The time is also near when the sand cat operator working at the South Perimeter dam thought he heard “some water”.</td>
</tr>
<tr>
<td></td>
<td>01:00 am to 1:06 am</td>
<td>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.</td>
</tr>
<tr>
<td></td>
<td>01:08 am</td>
<td>Time reported by a number of staff as to when the power went out at the mill, 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.</td>
</tr>
<tr>
<td></td>
<td>01:40 am to 2:20 am</td>
<td>Staff went to the TSF to check the power lines and reclaim water lines and realized that failure of the dam was in progress.</td>
</tr>
<tr>
<td></td>
<td>2:20 am</td>
<td>Emergency calls started being made to all potential parties and the dam area was being cordoned off.</td>
</tr>
<tr>
<td></td>
<td>04:30 am to 4:45 am</td>
<td>Checking on Polley Lake – lake level rose by 1 m.</td>
</tr>
<tr>
<td></td>
<td>05:15 am</td>
<td>Breach flow was observed as also reporting to Hazeltine Creek. Starting to get light outside.</td>
</tr>
<tr>
<td></td>
<td>5:30 am</td>
<td>Reclaim barge on tailings – water still “roaring” out of the TSF. Muddy water flowing out of the breach.</td>
</tr>
<tr>
<td></td>
<td>06:00 am</td>
<td>Helicopters on site: breach flow was observed as “going over the displaced block of soil on the downstream side of the dam”.</td>
</tr>
<tr>
<td></td>
<td>12:00 pm</td>
<td>Flow had reduced but was still “significant”.</td>
</tr>
<tr>
<td></td>
<td>04:00 pm</td>
<td>Flow had abated.</td>
</tr>
</tbody>
</table>

Note: Failure timeline and observations as presented in KCB (2015c).
3.2 Pre-Failure Conditions

3.2.1 Stage 9 Dam Raise Construction

As described in Section 2, the Stage 9 raise of the Perimeter Embankment commenced in 2013 and continued in 2014 up to the time of failure. Stage 9 construction in 2014 included: the raise of the till core (Zone S) from El. 967 m to El. 970 m (3 m raise), the raise of the upstream and downstream support materials to El. 969 m (3 m raise), and the re-location of the seepage recycle pipe located near Station 3+950 (see Figure 2.4). Further details on the construction sequencing are as follows:

- The till core (Zone S) was raised to El. 967 m by October 2013. Till core placement resumed in June 2014 starting at Station 3+100. Daily construction reports indicated the till core was completed to El. 968.8 m on July 31, 2014, but surveyed elevations from drill holes and a CPT sounding at the embankment crest located west and east of the breach area indicate the till core was likely closer to El. 970 m (see Table 2.1).

- Raise of the downstream shell (Zone C) started in June 2014. AMEC’s daily reports indicate, by August 1, 2014, Zone C placement reached the target elevation and was being graded to El. 969 m. On August 3, 2014, Zone C placement at the re-located seepage recycle pipe was completed.

- Hydraulic placement of tailings in the upstream shell (Zone U) started November 2013 and was last reported at El. 967.6 m in June 11, 2014 between Stations 4+286 and 4+396 m (AMEC’s construction daily report TSF14-06-11). Post-failure survey contours to the east and west of the failure area showed that Zone U was likely between El. 968 m and El. 969 m.

As discussed in Section 2.3.2, a 2 m deep excavation was reported to be made in November 2013 downstream of the embankment toe for the foundation of a future toe buttress. The excavation was open between Stations 4+100 and 4+300 at the time of failure.

Figure 2.6 shows the estimated Stage 9 embankment raise configuration at the time of failure.

3.2.2 Pond and Tailings Beach Levels

Historical pond elevation and tailings beach levels are given in Table 2.1. The pond elevation immediately prior to failure was El. 966.8 m as reported in AMEC’s daily report dated August 3, 2014. The beach elevation at that time was estimated to be El. 966.3 m as inferred from the post-failure tailings contours outside the breach area, the observed submergence of the tailings beach seen in Figure 2.3 and photograph records from August 2014 in Appendix I-E.

The embankment crest was raised to meet freeboard requirements and to allow development of beach in front of the embankment. Starting in June 2011 (Stage 7), rises in the pond level hampered the ability to maintain the tailings beach in front of the embankment. Subsequently, the tailings beach was partially submerged during Stage 8 and completely submerged during the final Stage 9 construction (see Figure 2.3).
Although not in the vicinity of the failure, a “near-overtopping” incident was reported on May 25, 2014 near the corner between the Main and South Embankments (Corner 3), between Stations 1+475 and 1+515. The incident was described in AMEC’s daily report as: “water is seeping through the u zone and ending up on top of our till which then flows into the filter due to the water elevation in the sand cell being 0.2 m higher than the top of the till @ corner 3.” A temporary berm comprised of random fill (Zone U) over the sand cell and glacial till was subsequently placed upstream of the core to contain the water. The pond water elevation at the time of this incident was 966.5 m.

3.3 Post-Failure Conditions

Figure 3.1 shows the post-failure topography of the TSF as determined from a Lidar survey by MPMC on August 5, 2014. Figure 3.2 presents an aerial plan view of the immediate breach area.

Field mapping and visual reconnaissance of the failure area was conducted by KCB and the findings are documented in Progress Report No. 2. Figures 3.3 and 3.4 present a zonation of the breach area, with select key field mapping features presented in Figures 3.5 and 3.6, and in reference photographs in Figures 3.7 to 3.11. Relevant observations are discussed below:

- Zones 1 to 5: Upthrust ground and bulging at the dam toe was observed from nominally Stations 4+110 to 4+330, indicating a failure width of 220 m. This corresponds to a failure aspect ratio\(^6\) of 5.5.

- Zone 1: The failed dam was not subject to erosion during the subsequent dam breach and was left largely intact. The toe of the dam was upthrust vertically approximately 6 m. The original ditch at the dam toe was also displaced laterally approximately 10 m. The dam displacements associated with the toe bulge are visible in Figure 3.9a.

- Zone 2: Between Stations 4+180 and 4+220, the failed dam was overtopped and the downstream slope subjected to surface water flow, leaving a 0.3 m to 0.6 m layer of water washed rock on the lower half of the slope and shallow erosion channels up to 0.6 m deep (Figure 3.9b and 3.9c). The upthrusted toe bulge was also eroded by the water flow (Figure 3.9d), indicating that the toe bulge occurred prior to pond release.

- Zone 3 and 4: The main breach of the dam and release of water and tailings occurred between Stations 4+200 and 4+300. Initial downcutting of the dam in Zones 3 and 4 reached elevation 940 m, followed by downcutting to the original ground between Stations 4+250 and 4+290 in Zone 4.

- Zones 5 and 6: The left abutment of the failed dam was eroded during the breach to a steep angle of about 60°. Intact features of the rockfill placement in Zone C are clearly visible in the exposure in Figure 3.8. The construction lifts are back-tilted 10° upstream into the impoundment. Surface mapping of the abutment revealed several shallow scarps oriented parallel to the eroded slope (perpendicular to the dam crest). These scarps are seen in Figure 3.11c and are interpreted to be relaxation of the slope into the breach area as the dam

\(^6\) Failure aspect ratio is defined in the failure width to dam height, which was nominally 40 m at the time of failure.
was eroded out. The stability of the steep left abutment slope was therefore considered marginal by MPMC and site investigations within the breach area were restricted to keep a safe distance.

- **Zone 3:** Similar scarps were detected on the right abutment of the breach zone as denoted in Figure 3.7.

- **Zones 7 and 8:** As a consequence of the upstream construction method of the Perimeter Embankment, tailings lost by erosion through the breach destabilized the upstream side of the embankment. Large portions of Zone U, S and C were lost on either side of the breach, extending from Stations 4+100 to 4+450. This included the upstream toe drain and concentrated seepage observed emanating from the exposed upstream slope is considered to be from the “broken” ends of the drain. Locations of these seep points are shown on Figures 3.5 and 3.6.

- **Zones 7 and 9:** The loss of the upstream tailings exposed the construction lifts in the rock fill and till core on the upstream face of the embankment. These construction lifts are visible in Figure 3.10d, 3.11a and 3.11b. On the left abutment, horizontal lift layers were observed outside the failure zone beyond Station 4+350. Similarly, on the right abutment, lift interfaces were horizontal up to about Station 4+100 and were noticeably tilted longitudinally towards the breach from Station 4+150 to 4+250. Field measurements of the longitudinal tilting ranged from 5° to 6° towards the breach.

- **Zone 8:** A large segment of the compacted till core was left intact between Stations 4+200 and 4+250. Construction lifts are visible in Figure 3.10c and were measured to be back-tilted nominally 16° into the impoundment. A subsequent excavation into the core revealed a distinct shear plane extending downstream through the core and into the underlying native till. This important observation is discussed further in Section 4.3.

### 3.4 Interpretation of Field Observations

The field observations clearly point to a foundation failure of the tailings embankment as the cause of the ultimate dam breach. The upthrust at the dam toe and back-tilting of the body of the dam cannot be explained by alternate failure mechanisms, including dam overtopping or internal piping through the till core. The upthrust at the dam toe show that the displacements of the failed dam were 5 m to 10 m or more.

The geomorphological evidence shows that the failed dam was subject to initial overtopping of water onto the exterior over a wide spread area. This indicates that the pond water had overtopped the core of the dam, which can only be explained by a large down-drop of the dam crest. Subsequent erosion and concentration of the overtopping water flow lead to rapid downcutting of the dam, thereby allowing the stored tailings to escape the impoundment. Figure 3.12 depicts our interpretation of this failure progression.
4 POST-FAILURE SITE INVESTIGATIONS

KCB was retained by MEM to conduct a Site Investigation (SI) program to collect pertinent geotechnical information to support an evaluation of the mechanism of failure of the tailings dam. Field activities were conducted between mid-September and late-November 2014. Subsequent laboratory testing continued to the end of January 2015.

Progress Report No. 2 (KCB 2015a) sets out the results of the field investigations and laboratory index testing. Progress Report No. 4 (KCB 2015b) presents the procedures and results from more advanced laboratory testing.

The main objectives of the SI Program were to:

- collect field data to help determine the failure mechanism;
- characterize the profile and geotechnical properties of the foundation soils both inside and outside the failed area;
- investigate and characterize construction materials within the dam that may have influenced the failure mechanism; and
- locate the seat of the dam failure through the foundation materials, if applicable.

Figures 4.1 to 4.3 present the layout of the investigations conducted by KCB. Select information from the SI program is included in Appendix II for ease of reference.

Figure 4.4 shows the location of cone penetration testing conducted by the Panel under the direction of Thurber Engineering Ltd. This information has been reviewed but is not included in the summary below.

4.1 Site Investigations

The site investigations comprised:

Geotechnical Investigation

- Visual reconnaissance and surficial mapping at the dam breach area to establish the post-failure configuration and provide insights into the nature of the failure mechanism.
- Eight electric resistivity survey lines and seven seismic refraction survey lines to estimate bedrock depth and approximate material boundaries.
- Sonic coring to delineate the soil profile, obtain samples for geotechnical testing, and install instrumentation.
- Thirty-two sonic core holes were drilled at 22 primary investigation locations. Two sonic core holes (SH14-20 and SH14-21) were located on the dam crest on the left and right side (looking downstream) of the breach, respectively. The proximity of these two holes to the dam breach was restricted by MPMC due to safety concerns.
Undisturbed thin-walled tube samples collected in mud rotary drill holes. High quality samples, confirmed with non-destructive X-Ray scanning, were used for advanced laboratory testing. Additionally, preserved cores from sonic drilling and undisturbed block samples were collected targeting the main soil profile units.

- Trench and test pit excavations in the exposed Till Core and in the upthrusted toe bulge to map features and to collect representative disturbed and undisturbed samples.
- Two bulk samples from the embankment till core were collected for advanced testing.

**In Situ Testing**

- Standard Penetration Testing (SPT) was conducted on the upper glacial till soils at four locations to a maximum depth of 13.8 m below ground surface. Two SPTs were completed within the failed area and two in the free field downstream area.
- Seismic Cone Penetration Testing (CPT) was conducted at 21 locations adjacent to sonic drill holes. One CPT was completed at the embankment crest through the Till Core.
- Vane Shear Testing (VST) was conducted at four locations in the upper glaciolacustrine deposits. One test was completed within the failed area and two in the free field downstream area.

**Instrumentation**

- A total of 22 vibrating wire piezometers (VWP) were installed in the foundation soil units and bedrock at selected depths. VWP’s were preferentially placed in the glaciolacustrine and glacioluvial deposits.
- Six of the VWP’s were installed in SH14-20 and SH14-21 on the embankment crest, at the nearest locations to the embankment breach allowed by MPMC due to safety concerns. These VWP’s were intended to measure the piezometric conditions within the foundation soils which were not disturbed by the failure. However, it is noted that the measured piezometric levels reflect post-failure conditions and would be influenced by the loss of the stored water pond, removal of the tailings upstream by erosion through the breach and subsequent post-failure drainage of the tailings deposits.
- A total of five inclinometers were installed within the failed embankment. Each was installed with a minimum 3 m embedment into bedrock, to depths of 23.5 m to 61.7 m. The objective was to record potential post-failure “creep” movements in the foundation soils.

**4.2 Laboratory Testing**

Laboratory tests were conducted on representative disturbed soil and bedrock samples to determine soil index properties. Undisturbed thin-walled tube and block samples were tested to determine consolidation and strength properties. The laboratory testing included:

- 991 in situ water content tests at approximately 0.5 m to 1.0 m intervals on the sonic core.
17 specific gravity tests for glacial till and the upper and lower glaciolacustrine units.
3 organic content tests in the upper glaciolacustrine.
139 Atterberg Limits in fine-grained foundation soils.
12 X-ray Diffraction tests in fine-grained foundation soils to assess clay mineralogy.
127 particle size tests, with 125 hydrometer tests.
2 Standard Proctor Tests on Till Core samples.
8 Triaxial Permeability and Compression tests on the upper glacial till and the Till Core.
1 Triaxial Extension test on the upper glacial till.
4 Triaxial Compression tests on the upper glaciolacustrine.
15 Direct Shear tests on the upper and lower glaciolacustrine deposits.
15 Direct Simple Shear tests on the upper and lower glaciolacustrine deposits.
10 Oedometer Consolidation tests on the upper and lower glaciolacustrine deposits.
3 Oedometer Consolidation tests on reconstituted samples of the upper and lower glaciolacustrine deposits.

4.3 Observations of Failure Plane

In addition to the drilling investigations, trenches and test pits were undertaken to investigate the nature of the dam displacements and identify the shear plane invoked by the dam failure. These included a deep trench excavation\(^7\) into the remnant near-vertical till core in Zone 8 (Figure 3.3 and 3.10c) and a test pit (TP14-01) into the upthrust ground bulge at the embankment toe. Salient observations are:

- The dominant feature exposed in the till core excavation was a shear zone that cross cut the core dipping steeply in the downstream direction. The shear zone was typically identified in each excavation slice by the following:
  - An abrupt color change in the till core upstream and downstream of the shear. Generally the till downstream of the shear was a mixture of grey and brown till with some color “banding” which were likely construction lifts. The heterogeneity in the fill color is attributed to variability in the till fill borrow source. Upstream of the shear, the till core was consistently a more homogeneous brown color, and no grey till was observed.
  - A zone of disturbance and softening within the shear zone, often accompanied with foreign materials such as tailings sand, gravel and cobbles. The width of the disturbed zone, and the degree of disturbance, was highly variable. Shear zone width was highly

\(^7\) The excavation of the till core was planned and supervised by Thurber Engineering Ltd. KCB was present during the excavation to make independent observations.
variable and ranged from closed and only perceptible by the abrupt colour change to up to 2 m wide.

c. Zones of seepage and saturated tailings emanating from the shear zone.

Excavation of test pits at the base of some of the slices showed that the shear zone continued into the foundation, to El. 925 m, and appeared visually to extend to deeper elevations. This is over 3 m below the natural ground surface (after site preparation) which ranged in elevation from 928.0 m to 928.7 m. This also indicated this feature was not a construction interface between material zones.

Selected photographs of the shear zone are shown in Figure 4.5. The shear plane surface interpolated from survey data is shown in Figure 4.5b.

- In TP14-01, a glaciolacustrine clay layer was encountered at El. 927.5 m, which is 6.5 m above the top elevation of the glaciolacustrine layers in adjacent sonic core holes (El. 921 m). Varve laminations in the clay were inclined 40° to 45°, dipping towards the upstream direction. Figure 4.6 shows the clay exposure and inclined varve laminations.

These observations indicate that the sliding plane invoked by the dam failure passed through the lower portions of the compacted till core and exited at depth near the embankment toe. Total displacements along the shear plane were likely in excess of 5 m. These findings are consistent with the field observations of the failed dam discussed in Section 3.3 and 3.4.
5 SITE CONDITIONS

5.1 General

This section describes our characterization of the site conditions at the failed embankment and selects parameters for the failure analyses in Section 6. Information presented includes: site topography prior to TSF construction, soil profiles below the failed embankment, characterization of the foundation soils and bedrock units, a summary of observations from piezometers and inclinometers installed post-failure, and forensic evidence of shearing and displacements in the dam and foundation.

Most of this information is based on results from the post-failure site investigation and laboratory testing presented in Progress Reports Nos. 2 and 4 (KCB 2015a and 2015b), which contain additional information. Select information from these reports and supplementary data summaries are given in Appendix II.

5.2 Pre-Development Topography

Figures 5.1 and 5.2 show the original topography and an aerial view of the TSF prior to the start of construction. The TSF rests in the upper catchment of a tributary to Edney Creek draining to the southeast. The Main Embankment crosses the tributary creek valley and confines the southeast side of the TSF. The Perimeter Embankment and the South Embankment are extensions of the Main Embankment across topographic lows on the southwest and northeast sides. Together, the three embankments result in a U-shaped tailings dam abutting a natural slope on the northwest side.

Figure 5.1 shows the set-out-line (S.O.L) of the tailings embankment. The figure includes the six location points where the S.O.L bends and ends on the valley abutment. These are labelled Corners 1 to 5, and 1.5 in the design and as-built reports. The embankment failure occurred adjacent to Corner 1 in a topographic low along the Perimeter Embankment.

As described in Section 3, references to “swampy areas” were reported during the foundation preparation works near Corner 1. These observations are consistent with the relatively flat topographic low in the “saddle” to the east of Corner 1.

Figure 5.3 shows the Perimeter Embankment profile along the S.O.L based on the original ground surface digitized from Drawing 1625.101 in Knight Piesold 1995 TSF design report (MP00001, KP 1995), the pre-failure surface from August 2013, and the post-failure surface from August 5, 2014. The profile shows that the embankment failed near the highest section of the Perimeter Embankment and the subsequent breach eroded down to the original ground surface.

Figure 5.4 compares the contact elevation between embankment fill and native ground encountered in sonic core holes. Excluding the upthrusted ground in SH14-05, the contact elevations confirm the topographic low below the failed embankment.
### 5.3 Soil Profile

The soil profile encountered in the vicinity of the embankment failure consists of the ten main soil and bedrock units listed in Table 5.1. The following sections describe the units and relevant geotechnical properties.

**Table 5.1 Description of Main Soils and Bedrock Units**

<table>
<thead>
<tr>
<th>Unit Name</th>
<th>ID</th>
<th>General Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Upper Glacial Till</td>
<td>UGT</td>
<td>CLAY AND SAND (CL-SC), some silt, trace gravel, low plasticity, very stiff to hard near surface and becoming firm to stiff at contact with UGLU brown/grey.</td>
</tr>
<tr>
<td>Upper Glaciolacustrine Unit</td>
<td>UGLU</td>
<td>CLAY (CI-CH), some silt, trace sand, intermediate to high plasticity, firm to stiff, grey, typically laminated with fine grained sand or silt, high dry strength. Laminations are sub-horizontal in the free field beyond the dam toe and may be inclined or deformed beneath the failed dam.</td>
</tr>
<tr>
<td>Middle Glacial Till</td>
<td>MGT</td>
<td>SANDY CLAY (CL), some gravel, some silt, low plasticity, hard, greenish-grey.</td>
</tr>
<tr>
<td>Lower Glaciolacustrine Unit</td>
<td>LGLU</td>
<td>CLAY (CI), some silt, trace sand, intermediate plasticity, hard, greenish-grey with some dark grey layering at the base of the layer, high dry strength, laminated with wavy layers of clay/silt and trace fine grained sand.</td>
</tr>
<tr>
<td>Upper Glaciofluvial</td>
<td>UGF</td>
<td>SANDY SILT (ML), trace gravel, non to low plasticity, dark grey, strong organics odour, varies in fines content from laminated silt and fine grained sand to well graded sand with some gravel.</td>
</tr>
<tr>
<td>Lower Glaciofluvial</td>
<td>LGF</td>
<td>SANDY SILT (ML), trace gravel, non to low plasticity, brown, sand is primarily fine grained, varies in fines content from laminated silt and fine grained sand layers or sand and gravel with no silt to coarse gravel in a silt matrix.</td>
</tr>
<tr>
<td>Lower Glacial Till</td>
<td>LGT</td>
<td>SANDY SILT (ML-CI), some gravel, some clay, low to intermediate plasticity, hard, brown and at times reddish brown near the bottom of the unit.</td>
</tr>
<tr>
<td>Weathered Sedimentary Bedrock</td>
<td>WB (Sed)</td>
<td>CLAY (CH), some silt, trace sand, high plasticity, greenish-grey to at times brown, hard, varying sand content from trace to sandy, at times having discontinuous green, red/brown, and yellow/green sand pockets and seams, high dry strength. Slickensides and smooth fracture surfaces occasionally observed at various inclinations.</td>
</tr>
<tr>
<td>Weathered Mafic-Igneous Bedrock</td>
<td>WB (Mafic)</td>
<td>SANDY GRAVELLY SILT (ML), trace clay, low plasticity, hard, dark grey, massive, all gravel and sand particles are black, angular, fine grained mafic rock, fines content decreases with depth and gravel content increasing with depth, becoming coarse grained and clast supported.</td>
</tr>
<tr>
<td>Weathered Volcanics Bedrock</td>
<td>WB (Vol)</td>
<td>SILTY SAND (SM), fine to medium grained, some gravel, poorly graded, angular, reddish brown to purple, gravel and sand grains are medium to fine grained volcanoclastic rock, reddish brown in colour</td>
</tr>
</tbody>
</table>

### 5.4 Glacial Tills

Three glacial till units were identified: the Upper Glacial Till (UGT), the Middle Glacial Till (MGT), and the Lower Glacial Till (LGT). Table 5.2 compares the index properties for these tills.
Table 5.2  Summary of Index Properties for UGT, MGT, and LGT

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Upper Glacial Till (UGT)</th>
<th>Middle Glacial Till (MGT)</th>
<th>Lower Glacial Till (LGT)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Soil classification</td>
<td>CL-SC</td>
<td>CL</td>
<td>ML-CI</td>
</tr>
<tr>
<td>Specific gravity</td>
<td>2.74</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Gravel content (%)</td>
<td>2 to 21 (12)</td>
<td>0 to 13 (6)</td>
<td>9 to 22 (14)</td>
</tr>
<tr>
<td>Sand content (%)</td>
<td>15 to 42 (36)</td>
<td>3 to 47 (37)</td>
<td>37 to 52 (48)</td>
</tr>
<tr>
<td>Fines content (%)</td>
<td>37 to 83 (52)</td>
<td>43 to 97 (51)</td>
<td>29 to 54 (40)</td>
</tr>
<tr>
<td>Clay content (%)</td>
<td>13 to 26 (17)</td>
<td>12 to 30 (19)</td>
<td>9 to 17 (13)</td>
</tr>
<tr>
<td>In situ water content (%)</td>
<td>7 to 31 (12)</td>
<td>4 to 24 (13)</td>
<td>4 to 25 (9)</td>
</tr>
<tr>
<td>Liquid limit (%)</td>
<td>21 to 29 (25)</td>
<td>20 to 38 (25)</td>
<td>20 to 46 (26)</td>
</tr>
<tr>
<td>Plastic limit (%)</td>
<td>11 to 16 (13)</td>
<td>13 to 20 (15)</td>
<td>13 to 16 (14)</td>
</tr>
<tr>
<td>Plasticity index (%)</td>
<td>6 to 15 (12)</td>
<td>6 to 18 (11)</td>
<td>6 to 30 (13)</td>
</tr>
<tr>
<td>Liquidity index</td>
<td>(0.15)</td>
<td>(-0.20)</td>
<td>(-0.10)</td>
</tr>
<tr>
<td>Activity</td>
<td>(0.60)</td>
<td>(0.60)</td>
<td>(0.70)</td>
</tr>
</tbody>
</table>

Note:
1. Values presented are minimum and maximum range of tested data. The median of this range is included in brackets.

5.4.1  Upper Glacial Till (UGT)

The UGT comprises a low plastic, brown to brown-grey, silty, clayey sand matrix, with trace to some gravel. The upper till is moderately layered with thin lenses or partings of fine-grained, silty sand and/or silt. Vertical fine-grained sand streaks with rust mottling were observed in the upper one to two meters and are attributed to desiccation cracks infilled with fine sediments. Representative photos of the UGT are shown in Figure 5.5.

The UGT is continuous in extent and thickness below the failed embankment. Thickness varies from 8 m to 10 m and the bottom of the UGT is typically El. 920 m to El. 922 m.

Water content varies from 7% to 31%, with a median of 12%. The median liquidity index of 0.15 indicates the UGT is moderately overconsolidated. A preconsolidation stress of 200 kPa is inferred for the UGT based on consolidation testing conducted by the Independent Review Panel (IRP 2015).

Figure 5.6 presents SPT blow counts in the UGT. The median N_{value} of 11 in the free field indicates that the native UGT is stiff in consistency. CPT soundings in the free field gave a median tip resistance of 30 bars and undrained strengths over 100 kPa (see figures in Appendix II-D).

Figure 5.7 shear wave velocity data from the seismic CPT soundings. The minimum shear wave velocity in the free field was 260 m/s versus 160 m/s for tests conducted below the failed embankment. This is an indication of disturbance of the UGT during the failure.

Triaxial consolidated undrained tests in compression and extension were conducted in free field samples at effective confining stress of 200 kPa (Figures VI-2 and VI-4 in Appendix II-E). The UGT showed net dilatant response to undrained loading reflecting the overconsolidated in situ state. The effective friction angle at the peak strength is 34° in compression and 33° in extension.
Flexible-walled permeability tests showed vertical hydraulic conductivities ($K_v$) ranging from $5 \times 10^{-10}$ m/s to $5 \times 10^{-11}$ m/s, which is within a range of $1 \times 10^{-9}$ m/s to $1 \times 10^{-11}$ m/s for matrix rich tills reported by Eyles (1983). Estimates of hydraulic conductivity from CPT dissipation testing, shown in Table II-3 in Appendix II, reflected higher permeability in the horizontal direction with an anisotropy ratio ($K_h/K_v$) ranging from 1 to 10, with median of 2.

### 5.4.2 Middle Glacial Till (MGT)

The MGT comprises a low to medium plastic, green-grey, sandy, silty clay matrix, with trace to little gravel. The MGT is sporadically layered with thin lenses or partings of fine-grained, silty sand and/or silt. The sandy clay matrix is generally unoxidized and contains a lesser portion of gravel sizes, but greater portion of fines than the LGT.

The MGT appears to be continuous below the failed embankment with thickness varying from 2 m to 5 m, and is generally present between El. 916 m and El. 920 m.

Water content ranges from 4% to 24%, with median of 13%. The median liquidity index of -0.20 indicates the soil is heavily overconsolidated. A preconsolidation stress of 400 kPa is inferred for the MGT based on consolidation testing conducted by the IRP (2015).

CPT soundings in the free field yielded a median tip resistance over 84 bars and undrained strengths over 100 kPa. The free field shear wave velocity ranges from 230 m/s to 470 m/s. These results indicate the till is very stiff to hard. No advanced laboratory testing was conducted on samples of this unit. Thus, shear strength and hydraulic conductivity properties for analysis in Section 6 were estimated based on soil classification and index properties.

### 5.4.3 Lower Glacial Till (LGT)

The lowermost till unit is a low to medium plastic, oxidized, brown to reddish-brown, sandy silt having a massive clast-supported structure with occasional boulders and cobbles. The LGT is discontinuous and is mainly found in bedrock lows between El. 910 m to El. 919 m.

The LGT is heavily overconsolidated with a median liquidity index of -0.10. CPT soundings classify the LGT as very hard in consistency, which agrees with the single shear wave velocity measured of 420 m/s. No advanced laboratory testing was conducted on samples of this unit. Thus, shear strength and hydraulic conductivity properties for analysis in Section 6 were estimated based on soil classification and index properties.

### 5.4.4 Material Properties for Failure Analysis

The material properties for the glacial till units adopted for the failure analysis presented in Section 6 are given in Table 5.3.
Table 5.3  Summary of Material Properties for Failure Analysis – Glacial Till Units

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Upper Glacial Till (UGT)</th>
<th>Middle Glacial Till (MGT)</th>
<th>Lower Glacial Till (LGT)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unit weight (kN/m³)</td>
<td>22.7</td>
<td>22.5</td>
<td>23.1</td>
</tr>
<tr>
<td>Peak effective friction angle</td>
<td>35° (free field)</td>
<td>32°</td>
<td>35°</td>
</tr>
<tr>
<td>(below embankment)</td>
<td>33° (below embankment)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Undrained shear strength</td>
<td>Su = 0.38σvo’</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>(below the embankment)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Horizontal hydraulic conductivity</td>
<td>4x10⁻⁹ (free field)</td>
<td>7x10⁻⁹</td>
<td>1x10⁻¹⁰</td>
</tr>
<tr>
<td>kₕ (m/s)</td>
<td>2x10⁻⁹ (below embankment)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Vertical hydraulic conductivity</td>
<td>2x10⁻⁹ (free field)</td>
<td>7x10⁻⁹</td>
<td>1x10⁻¹⁰</td>
</tr>
<tr>
<td>kₙ (m/s)</td>
<td>1x10⁻⁹ (below embankment)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Anisotropic ratio (kₕ/kₙ)</td>
<td>2</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>Coefficient of consolidation</td>
<td>2x10⁻³ (Note 2)</td>
<td>4x10⁻³ (Note 2)</td>
<td>-</td>
</tr>
<tr>
<td>(cm²/s)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Compression index (Cc)</td>
<td>0.110</td>
<td>0.090</td>
<td>-</td>
</tr>
<tr>
<td>Recompression index (Cr)</td>
<td>0.009</td>
<td>0.015</td>
<td>-</td>
</tr>
<tr>
<td>Preconsolidation stress (P'c)</td>
<td>200</td>
<td>400</td>
<td>-</td>
</tr>
<tr>
<td>Initial void ratio (e₀)</td>
<td>0.41</td>
<td>0.50</td>
<td>-</td>
</tr>
</tbody>
</table>

Note:
1. Unit weight calculated based on median water content, specific gravity, and 100% saturation.
2. At stress levels up to 1000 kPa, laboratory testing shows similar coefficient of consolidation (vertical) for stresses higher and lower than the preconsolidation stress (P'c).

5.5 Glaciolacustrine Deposits

Two glaciolacustrine units were identified: the Upper Glaciolacustrine (UGLU) and the Lower Glaciolacustrine (LGLU). Table 5.4 compares the index properties for these units.

Table 5.4  Summary of Index Properties for UGLU and LGLU

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

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.
5.5.1 Upper Glaciolacustrine (UGLU)

The UGLU comprises a medium to high plastic, laminated/varved grey clay and silt, with trace of sand.

Figures 5.8 and 5.9 show the top elevation of the UGLU and thickness encountered in each sonic core hole. The UGLU deposit is encountered approximately 10 m below the embankment at El. 920 m to El. 924 m, and ranges up to 2 m thick. The areal extent is largely confined to the immediate area of the failed embankment and is thickest beneath the embankment toe in the center of the failure area.

Figure 5.10 shows representative photographs of the UGLU from sonic core holes. Laminations in the native UGLU downstream of the failed embankment are sub-horizontal, whereas heavily destructured and folded varves are observed in drill holes below the failed embankment. This is an indication that the failure surface was within this unit.

The clay fraction of the UGLU ranges from 39% to 67% with a median of 50%. XRD analyses classify the clay-sized particles to be composed principally of illite, chlorite, and smectite.

Figures 5.11 to 5.12 show the areal distribution of the maximum liquid limit and maximum water content measured in the UGLU in each sonic core hole. The maximum liquid limit ranges from 33% to 69%, with median of 54%. In the free field, the maximum water content ranges from 30% to 54%, with median of 37%. Below the embankment, the maximum water content is slightly lower ranging from 18.5% to 38%, with a median of 32%. The water content is lower than the free-field due to consolidation under the weight of the embankment.

Water content profiles in the sonic core holes are shown in Figures 5.13 and 5.14 for the geological sections C and D. The water contents in the UGLU are markedly higher than any other soil unit and they extend continuously from the free field to underneath the failed embankment.

The liquidity index in the UGLU ranges from 0.1 to 1.0, with median of 0.5 (see Figure 15 in Appendix II-B). This indicates the UGLU is lightly over-consolidated.

Figure 5.15 summarizes the consolidation test results on free field samples for the UGLU. The preconsolidation pressure ranges from 380 kPa to 420 kPa, with a mean of 400 kPa. For a depth of 10 m below ground, the Over-consolidation Ratio (OCR) of the native clay is nominally 4. The UGLU also shows higher compressibility and slower consolidation rates at loads above the preconsolidation stress.

Figure 5.16 shows the undrained shear strength from the CPT testing in the UGLU and LGLU, and Figure 5.17 shows results from the Vane Shear testing (VST) in the UGLU. The native UGLU had a median CPT tip resistance of 22 bars (see Table II-2 in Appendix II) and interpreted undrained shear strength of 140 kPa, which indicates very stiff consistency. Vane shear testing gave a median peak undrained strength of 130 kPa and remolded strength of 34 kPa. The sensitivity of the clay ranged from 1 to 7.

Undrained shear strength profiles are shown in Figures 5.18 and 5.19 for Sections C and D. The minimum undrained shear strength of the UGLU is typically greater than 100 kPa in the free field, whereas reduced strengths as low as 50 kPa were found below the failed embankment. This is
counter-intuitive as higher strength should be observed due to consolidation of the clay under the weight of the embankment. This indicates a loss of strength within the UGLU during the failure, which is consistent with the disturbed structure of the clay seen in Figure 5.10b. It is notable that the lower shear strength of 50 kPa is close to the median remolded strength of 34 kPa from the VST.

In its native state prior to dam construction, the UGLU would have exhibited dilative response to shearing and its ultimate strength would be governed by the drained frictional strength. The weight of the 40 m high tailings dam subjected the UGLU to vertical stresses up to 800 kPa and substantial portions of the UGLU beneath the dam were loaded to stresses well above the pre-consolidation pressure. These loaded portions of the UGLU became “normally consolidated” and would have displayed a contractive response to shearing. The ultimate strength of normally consolidated clay is its undrained strength, which accounts for pore pressures developed during shearing. This change from lightly over-consolidated behavior to normally consolidated behavior occurred incrementally over time as the dam was raised.

The shear strength of the UGLU is controlled by the higher plastic zones within the clay layer. Accordingly, from direct shear and triaxial compression testing in Figures 5.20 and 5.21, we estimate the peak drained strength of the UGLU as \( c' = 0 \) kPa and \( \phi_r' = 22^\circ \), and the residual drained strength as \( \phi_r' = 12^\circ - 14^\circ \) (see Figure 5.20). The similarity of these parameters to the shear strength of other clay soils reported by Stark et al. (2013) indicates that the UGLU is not a unique or special soil (see Figures IV-3 and IV-4 in Appendix II).

Given the low hydraulic conductivity and contractive behaviour of the normally consolidated UGLU below the embankment, the rapid failure of the embankment would have mobilized the undrained strength of the UGLU. Furthermore, the principal mode of deformation within the thin UGLU layer would be in the horizontal direction. Accordingly, the undrained shear strength was evaluated by Direct Simple Shear (DSS) tests that can simulate a horizontal sliding failure through the UGLU (Ladd 1991). The DSS apparatus also allows cyclic reversals of shearing of the clay, which was used to assess the strength loss of the clay with increasing strains.

The undrained strength of the UGLU obtained from DSS testing is represented by \( Su = 0.22 \ OCI^{0.8} \sigma_v' \), where \( \sigma_v' \) is the effective vertical confining stress (see Figure 5.22). This relationship is identical to the average relationship for homogeneous sedimentary clays recommended by Ladd (1991) and, again, indicates the UGLU is not unique.

The UGLU loses appreciable undrained strength when deformed past its peak strength. Strengths at 20% strain and following 4 or 5 cycles of loading are also shown in Figure 5.22. The strength loss with increasing levels of strain is also seen in Figure 5.23.

The vertical hydraulic conductivity for the UGLU in oedometer testing ranged from \( 1.2 \times 10^{-10} \) m/s to \( 5.4 \times 10^{-9} \) m/s. Estimates of hydraulic conductivity from CPT dissipation testing, shown in Table II-3 in Appendix II, show higher permeability in the horizontal direction with an anisotropy ratio up to 30.
5.5.2 Lower Glaciolacustrine (LGLU)

The second glaciolacustrine clay unit (LGLU) is 3 m to 5 m below the UGLU. The LGLU comprises medium plastic, varved lacustrine greenish-grey clay. Figure 5.24 shows representative photographs of the LGLU. Undulating, wavy, and inclined bedding is observed in both the free field and below the failed embankment, indicating the structure in the LGLU pre-dated the failure events.

Figure 5.25 shows the top elevation of LGLU encountered in sonic core holes. The LGLU is not continuous below the failed embankment and the top of the unit varies widely from El. 915 m to El. 921 m. The LGLU is typically less than 2 m thick below the failed embankment, but is up to 4.6 m thick at drill hole SH14-15.

The clay-size fraction of the LGLU is less than the UGLU, ranging from 23% to 32%, with a median of 26%, compared to a median of 50% for the UGLU. XRD analyses identify the clay-sized particles to be principally illite, chlorite, and smectite. Water content in the LGLU is also less than the UGLU, ranging from 15% to 29%, with a median of 23%, which is 12% lower than the median water content in the UGLU.

Liquid limits for the LGLU range from 31% to 42%, with median of 35%. The liquidity index typically ranges from -0.2 to 0.5, with median of 0.2. Oedometer testing shown in Figure 5.15 show that the native LGLU is “moderately over-consolidated” with a preconsolidation stress in excess of 750 kPa. This is consistent with the liquidity index values.

The LGLU is very stiff to hard as reflected by a median CPT tip resistance of 98 bars and undrained shear strength over 500 kPa.

Under the stresses imposed by the tailings dam, the LGLU would have exhibited dilative stress-strain response and its ultimate strength governed by the drained frictional strength. One direct shear test yielded peak friction angle of 33° with a residual friction angle of 25° for a sample with a 35% liquid limit and 28% clay content. This result is considered to represent the average strength of the LGLU. However, the strength of the LGLU would be controlled by the higher plastic and more clayey horizons in the unit. Accordingly, strength parameters were estimated using friction angles values for properties representing the upper 2/3rds of the Atterberg Limits and clay-sized fraction. A fully-softened drained friction angle of 28° and residual drained friction angles of 18° (below the embankment) and 23° (free field) are indicated for the LGLU based on a liquid limit of 41% and a clay-fraction of 31% using empirical correlations from Stark et al. (2013). The empirical relationships used are shown in Figure 5.26.

The vertical hydraulic conductivity for the LGLU in oedometer testing ranged from 1.1 x 10⁻¹⁰ m/s to 2.2 x 10⁻¹⁰ m/s. Estimates of hydraulic conductivity from CPT dissipation testing (Table II-3 in Appendix II), show higher permeability in the horizontal direction with anisotropy ratios ranging from 50 to 100.
5.5.3 Material Properties for Failure Analysis

The material properties for the glaciolacustrine units adopted for the failure analysis described in Section 6 are summarized in Table 5.5.

Table 5.5 Summary of Material Properties for Failure Analysis – Glaciolacustrine Units

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Upper Glaciolacustrine (UGLU)</th>
<th>Lower Glaciolacustrine (LGLU)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unit weight (kN/m³)</td>
<td>18.6</td>
<td>20.0</td>
</tr>
<tr>
<td>Peak effective friction angle</td>
<td>22°</td>
<td>28°</td>
</tr>
<tr>
<td>Residual effective friction angle</td>
<td>14°</td>
<td>23° (free field)</td>
</tr>
<tr>
<td>Undrained shear strength</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Horizontal hydraulic conductivity (k_h) (m/s)</td>
<td>1x10⁸ (free field)</td>
<td>2x10⁸</td>
</tr>
<tr>
<td>Vertical hydraulic conductivity (k_v) (m/s)</td>
<td>1x10⁹ (free field)</td>
<td>2.5x10⁻⁹</td>
</tr>
<tr>
<td>Anisotropic ratio (k_h/k_v)</td>
<td>10</td>
<td>80</td>
</tr>
<tr>
<td>Coefficient of consolidation (cm²/s)</td>
<td>8x10⁻⁴ (stress higher than P'c)</td>
<td>4x10⁻³ (Note 2)</td>
</tr>
<tr>
<td>Compression index (C_c)</td>
<td>0.35</td>
<td>0.09</td>
</tr>
<tr>
<td>Recompression index (C_r)</td>
<td>0.096</td>
<td>0.035</td>
</tr>
<tr>
<td>Preconsolidation stress (P'_c) (kPa)</td>
<td>400</td>
<td>750</td>
</tr>
<tr>
<td>Initial void ratio (e_v)</td>
<td>1.2</td>
<td>0.75</td>
</tr>
<tr>
<td>Coefficient of consolidation (C_n) at stress &gt;P'_c (cm²/s)</td>
<td>8x10⁻⁴</td>
<td>4x10⁻³</td>
</tr>
<tr>
<td>Coefficient of consolidation (C_n) at stress &lt;P'_c (cm²/s)</td>
<td>3x10⁻³</td>
<td>4x10⁻³</td>
</tr>
</tbody>
</table>

Note:
1. Unit weight calculated based on median water content, specific gravity, and 100% saturation.
2. Same coefficient of consolidation (vertical) for stresses higher and lower than the preconsolidation stress (P'_c).

5.6 Glaciofluvial Deposits

Two glaciofluvial units were identified: the Upper Glaciofluvial (UGF) and the Lower Glaciofluvial (LGF). Table 5.6 compares the index properties for these units.

Table 5.6 Summary of Index Properties for UGF and LGF

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Upper Glaciofluvial (UGF)</th>
<th>Lower Glaciofluvial (LGF)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Soil classification</td>
<td>ML</td>
<td>GP/SM/ML</td>
</tr>
<tr>
<td>Gravel content (%)</td>
<td>0 to 3 (0)</td>
<td>0 to 47 (0)</td>
</tr>
<tr>
<td>Sand content (%)</td>
<td>5 to 33 (10)</td>
<td>12 to 53 (46)</td>
</tr>
<tr>
<td>Fines content (%)</td>
<td>67 to 95 (90)</td>
<td>10 to 86 (48)</td>
</tr>
<tr>
<td>Clay content (%)</td>
<td>6 to 17 (11)</td>
<td>0 to 5 (5)</td>
</tr>
<tr>
<td>In situ water content (%)</td>
<td>7 to 29 (21)</td>
<td>4 to 21 (15)</td>
</tr>
<tr>
<td>Liquid limit (%)</td>
<td>21 to 29 (25)</td>
<td>-</td>
</tr>
<tr>
<td>Plastic limit (%)</td>
<td>19 to 25 (21)</td>
<td>-</td>
</tr>
<tr>
<td>Plasticity index (%)</td>
<td>0 to 8 (5)</td>
<td>NP</td>
</tr>
</tbody>
</table>

Notes:
1. Values presented are minimum and maximum range of tested data. The median of this range is included in brackets.
The UGF and LGF underlie the LGLU and are underlain by LGT or bedrock. Evidence of “artesian” water pressures were encountered during the post-failure site investigations in those permeable deposits. This is discussed in Section 5.9.

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.

CPT soundings gave median tip resistances of 194 bars for the UGF and 217 bars for the LGF. Several soundings refused further penetration near the top of these units. The median calculated \((N_1)_{60}\) for the UGF is 38 and for the LGF is 45. These values reflect a dense to very dense state.

No advanced laboratory testing was conducted on samples of this unit. Hence, the material properties in Table 5.7 are estimated based on soil classification and index properties.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Upper Glaciofluvial (UGF)</th>
<th>Lower Glaciofluvial (LGF)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unit weight (kN/m³)</td>
<td>20.5</td>
<td>21.7</td>
</tr>
<tr>
<td>Peak friction angle</td>
<td>30°</td>
<td>33°</td>
</tr>
<tr>
<td>Horizontal hydraulic conductivity ((k_h)) (m/s)</td>
<td>4x10⁻⁷</td>
<td>1x10⁻⁶</td>
</tr>
<tr>
<td>Vertical hydraulic conductivity ((k_v)) (m/s)</td>
<td>4x10⁻⁸</td>
<td>1x10⁻⁷</td>
</tr>
<tr>
<td>Anisotropic ratio ((k_h/k_v))</td>
<td>10</td>
<td>10</td>
</tr>
</tbody>
</table>

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

### 5.7 Bedrock

Figure 5.27 shows the top of the bedrock encountered in sonic core holes and Figure 5.28 shows a geological section along the embankment toe. Included in Figure 28 are overlays of the geophysical surveys including seismic velocity and electrical resistivity profiles. The observed top of bedrock below the failed embankment is between El. 895 m and El. 918 m, approximately 12 m to 25 m below native ground surface (El. 930 m). Three bedrock types were encountered during the post-failure site investigation: weathered mafic-igneous bedrock, weathered volcanics, and weathered sedimentary bedrock.

#### 5.7.1 Sedimentary Bedrock

Weathered sedimentary bedrock is the predominant bedrock unit below the failed embankment area and was typically encountered between El. 905 m and El. 916 m.

This bedrock unit comprises a very high plastic, overconsolidated greenish-grey clay that is weakened by occasional slickensides and shears, which likely originate from historical tectonic stresses, glacial drag, glacial unloading and weathering. The sedimentary rock contains discontinuous green, red/brown, and yellow/green sand pockets and seams.
Figure 5.28 shows a good correlation between the presence of the sedimentary rock and the high resistivity anomaly shown in blue. Given the high plasticity of the sedimentary rock, a lower permeability would be anticipated in this unit than the harder mafic-igneous and volcanic rocks. This is discussed further in Section 5.9.

Also, the sedimentary bedrock is weaker than the other mafic-igneous and volcanic rocks. Hence, preferential erosion of sedimentary outcropping bedrock is likely to have contributed to the local topographic low (see Figures 5.1 and 5.4) at this area of the Perimeter Embankment.

5.7.2 Mafic-Igneous and Volcanic Bedrock

Weathered mafic and volcanic bedrock predominate to the east and west of the failed embankment area. The main characteristics for these units are:

- Weathered mafic-igneous bedrock comprised low plastic sandy gravelly silt with trace clay. The unit is hard, dark grey and massive. Gravel and sand particles are black, angular, fine grained mafic rock. It is observed that fines content decreases with depth and gravel content increasing with depth, becoming coarse grained and clast supported.
- Weathered volcanic bedrock comprised reddish brown to purple fine to medium grained silty sand with some gravel. Gravel and sand grains are angular, medium to fine grained volcanoclastic rock, reddish brown in colour.

Index properties for the weathered mafic-igneous and volcanic bedrock are presented in Appendix II-B. In situ water content ranges from 2.3% to 31.9% with median of 9.8%. CPT tip resistance when available across this unit ranged from 174 to 372 bars, with median of 194 bars. Shear wave velocity from CPT testing is around 360 m/s. These results indicate the weathered bedrock is very dense.

5.7.3 Material Properties for Failure Analysis

The material properties for the bedrock units adopted for the failure analysis presented in Section 6 are summarized in Table 5.8.

Table 5.8 Summary of Material Properties for Failure Analysis – Bedrock Units

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Weathered Mafic/ Volcanics (WB(Mafic/Vol))</th>
<th>Weathered Sedimentary (WB(Sed))</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unit weight (kN/m³)</td>
<td>23.0</td>
<td>17.9</td>
</tr>
<tr>
<td>Peak effective friction angle</td>
<td>Impenetrable</td>
<td>Impenetrable</td>
</tr>
<tr>
<td>Horizontal hydraulic conductivity (kₕ)</td>
<td>1x10⁻¹₀</td>
<td>&lt; 1x10⁻¹₀</td>
</tr>
<tr>
<td>Anisotropic ratio (kₓ/kₕ)</td>
<td>1</td>
<td>1</td>
</tr>
</tbody>
</table>

Note:
1. Unit weight calculated based on median water content, specific gravity, and 100% saturation.
5.8  Glacial Till Core (Zone S)

The CPT sounding (CPT14-21) within the Zone S Till Core gave a mean tip resistance of 94 bars with an undrained shear strength of 320 kPa in the upper 14 m of the core. These results are consistent with the expected hard consistency of compacted till.

Two bulk samples of the till core were collected during the 2014 Site Investigation program. One sample from the compacted till core was taken at the top of the embankment (southeast of drill hole SH14-21) and another sample was taken within the breach area during excavation of the till core. These bulk samples were re-compact ed to prepare specimens for testing.

Test results on samples compacted to 92% and 95% maximum Standard Proctor density and sheared at 500 kPa and 800 kPa confining stresses showed a net contractive response. The effective friction angle of the compacted till was 33° and the average undrained strength ratio was Su/p’c = 0.38 where p’c is the initial isotropic confining stress.

Flexible-walled hydraulic conductivity testing on samples at 500 kPa and 800 kPa ranged from 7 x 10⁻⁹ m/s to 4 x 10⁻¹⁰ m/s. For the seepage analysis in Section 6, the till core was divided in two zones, Zones S-1 and S-2, representing the upper and lower portions of the core, which were subjected to different consolidation stress levels and, hence, assigned different hydraulic conductivities.

5.9  Post-Failure Piezometer Readings

Figure 5.29 shows the location of piezometers installed during the 2014 post-failure site investigation together with pre-failure piezometers. Figures 5.30 to 5.32 present the instrumentation Sections D, C and G with the latest available piezometric readings. Within the failed embankment, Section C was used to portray the data instead of Section G shown previously in Figure 2.10.

The piezometers installed post-failure in the UGLU and other units found no evidence of high excess pore pressures related to the loading of the dam during construction. However, unloading and deformations in the UGLU during the failure and breach would have substantially changed the pore pressure regime in the clay from that of the pre-failure state.

Figure 5.33 shows the vertical seepage gradients in foundation soils and bedrock estimated from the latest set of piezometric readings. Two conditions are noted:

- Pore pressures below the failed embankment are largely hydrostatic, whereas outside the failed embankment, strong downward seepage gradients (0.1 to 0.7) are observed to the east and west. These areas of downward gradient coincide with areas underlain by mafic/volcanic rocks. This is one indication that the sedimentary rock below the embankment has a lower hydraulic conductivity than the surrounding rock. This is expected given the high plasticity of the sedimentary rock.
- Four observations of artesian pressures in the foundation deposits were made during drilling of the sonic core holes below the failed embankment. These are summarized in Table 5.9. All
occur within the upper and lower glaciofluvial deposits (UGF and LGF) and are likely due to the confinement between the lower permeability LGLU and sedimentary bedrock.

Table 5.9  Post-Failure Observations of Artesian Pressures in Glaciofluvial Foundation Deposits

<table>
<thead>
<tr>
<th>Sonic Core Hole</th>
<th>Observation</th>
</tr>
</thead>
<tbody>
<tr>
<td>SH14-01</td>
<td>Piezometric water pressure 4.6 m above ground surface at embankment toe in VWP tip B.</td>
</tr>
<tr>
<td>CPT14-03</td>
<td>Drillers reported back pressure expelling grout out of hole during completion. End of hole at 20.5 m within UGF.</td>
</tr>
<tr>
<td>CPT14-04</td>
<td>Artesian pressures encountered when drilling out from 17.5 m to 18.5 m in LGF. Two drill casings were added to measure water level above ground. Water level estimated to be 3.5 m above ground level.</td>
</tr>
<tr>
<td>SH14-18</td>
<td>Artesian pressures encountered when drilling in UGF and LGF between depths of 11 m and 16 m. Water pressures estimated to be 2 m above ground surface.</td>
</tr>
</tbody>
</table>

5.10  Post-Failure Inclinometer Readings

Inclinometers, located in Figure 5.34, were installed to record post-failure “creep” movements in the embankment foundation. Such movements would be evidence of the failure plane in the foundation due to large displacements and associated strength loss in the affected soils.

As shown in Figures 5.35 to 5.39, small post-failure movements were recorded by most inclinometers in the UGLU and point to the UGLU as the seat of the embankment failure. This is consistent with the heavily de-structured and folded varves of the UGLU below the embankment, as shown in Figure 5.10b, and the weakened state of the UGLU in the failure zone consistent with the remolding of the clay during the failure movement, as discussed in Section 5.5.1.

Inclinometer SH14-06 installed through the remains of the failed embankment gave movements predominately to the north, towards the downstream toe of the failed embankment. This movement is interpreted to be a remnant from the initial embankment failure.

Inclinometers installed in SH14-09, SH14-04, and SH14-03 display movements predominately to the northwest-west in the UGLU. The direction of these movements is towards the breach opening and this is attributed to the change in direction of the “driving” shear stress within the foundation as the breach developed. No evidence of movements was observed in inclinometer SH14-16 within the breach area, where the post-failure driving shear stresses are lowest. Also, the UGLU thins out in the vicinity of SH14-16 as shown in Figure 5.9.

The last reading from inclinometer SH14-09 indicated a discrete 2 mm shear displacement at El. 904 m within the weathered sedimentary bedrock. This single occurrence should be confirmed by further readings and, if movement is confirmed, should be considered in any future designs or operations of the TSF. Such discrete movements may be reflection of strains on a pre-existing shear within the bedrock.
5.11 Evidence of Large Embankment Displacements

Vertical displacements of the crest in excess of 3.2 m\(^8\) would have been required to cause an overtopping of the embankment. That such vertical displacements occurred during the failure was evident from the field mapping and site investigation. Four of the most important observations are illustrated in Figure 5.40 and are as follows:

1. Vertical and lateral displacements of 6 m and 10 m, respectively, in the undisturbed toe bulge in Zone 1 (Figure 5.40a).

2. The back-tilt of the construction lift interfaces exposed in Zone C in the left abutment of the breach. The back-tilt of 10° from horizontal yields a vertical rotational movements of 8 m over the 45 m width of Zone C exposed in the slope (Figure 5.40b).

3. Upthrust UGLU observed in TP14-01 at the dam toe. As discussed in Section 4.3, the UGLU was vertically upthrust 6.5 m from its original elevation in adjacent sonic core holes (Figure 5.40c).

4. The shear feature encountered in the remnant of the Zones S Till Core. As discussed in Section 4.3, this shear plane was traced to extend over 3.5 m below native ground.

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\(^8\) Based on maximum crest elevation of 970 m of Zone S till core and tailings water pond elevation of 966.8 m at time of failure.
6 BACK-ANALYSIS OF EMBANKMENT FAILURE

6.1 General

The forensic evidence from the field mapping, site investigation, and post-failure movements measured by the inclinometers indicate that the embankment failed by shearing through the UGLU. From all available evidence, the final trigger for the failure would have been the recent excavation at the toe in 2013 followed by raising of the embankment with the steep outer slope of 1.3H:1V. This section presents the results of back-analysis used to assess whether standard modelling techniques using material parameters from the site investigation support this failure hypothesis.

6.2 Conceptual Sequence of Dam Failure

Dam failure would have been initiated by local yielding of the UGLU clay whereby the static shear stresses in the UGLU exceeded the available drained strength defined by the peak effective friction angle, φ. This induced yielding of the clay would have led to rapid straining such that shear-induced pore pressures in the contractive clay would have insufficient time to dissipate, reducing the strength of the clay to its undrained resistance, Su. If the embankment is not stable with the available undrained shear strength, then the embankment would continue to deform until a stable configuration was attained. Such displacements would have strained the clay beyond its peak undrained shear strength and, if displacements were large enough, reduce the undrained strength to its remolded value.

Evidence for the failure process described above includes:

- Buckling and de-structuring of varved clay laminations in the UGLU below the failed embankment (Figure 5.10).
- The reduced undrained strength of the UGLU measured below the failed embankment as a result of shearing (Figures 5.16 and 5.17). This reduced strength approaches the remolded strength of the clay as determined by in situ vane shear testing.
- The upthrust UGLU observed in TP14-01 at the toe of the failed embankment.
- The post-failure movements measured by inclinometers in the UGLU.

Following from the above, KCB developed a conceptual sequence of dam failure as a framework to evaluate the failure process of the embankment. This hypothesized failure sequence is set out in five stages as listed in Table 6.1. Each stage corresponds to the strength state of the UGLU and considers the influence of other contractive clayey soils and fills on the failure process. The stages are as follows:

- Stage 1 considers the static stability of the embankment under fully drained conditions, using the peak effective friction angle of the UGLU. Local yielding of the foundation clay could occur if the limit equilibrium Factor of Safety (FoS) is low, typically less than 1.3, using the drained strength of the clay.
Stage 2 considers the embankment stability assuming that local yielding triggers the undrained strength of clay. If the FoS is less or close to unity, then failure of the embankment would occur. Mobilization of the peak undrained shear strength occurs at shear strains of 5% based on laboratory testing. For a 2 m maximum thickness of the UGLU, movements in the UGLU would be in the order of 0.1 m or less. Such movements would not be detectable by observation at the crest of the embankment.

Stage 3 considers failure of the embankment is occurring and the undrained shear strength of the UGLU is reduced to a post-peak strength due to the accumulation of strain within the clay. At 20% shear strain in the UGLU, maximum movements in the UGLU would now be in the order of 0.4 m. At these larger displacements, triggering of undrained shear strength in the compacted clay core and underlying UGT is now assumed.

Stage 4 represents the advanced state of failure with the undrained shear strength of the UGLU reduced to its remolded value.

Stage 5 represents the failed embankment coming to rest on the weakened foundation at a FoS of 1. This final state was achieved by the reduction of driving force as the crest of the embankment dropped and the increase in resisting force with the buckling and mounding of displaced soils at the embankment toe.

Breach of the embankment followed Stage 5 as the impounded water pond overtopped the down-dropped crest of the embankment. Schematically, this is shown in Figure 3.12.

**Table 6.1 Conceptual Sequence of Dam Failure**

<table>
<thead>
<tr>
<th>Stage</th>
<th>Description</th>
<th>Approximate Movement in UGLU (m)</th>
<th>Shear Strength in UGLU</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Static conditions using peak effective angles and piezometric conditions prior to failure.</td>
<td>0</td>
<td>( \phi'_p = 22^\circ )</td>
</tr>
<tr>
<td>2</td>
<td>Peak undrained shear strength mobilized at 5% strain within the UGLU as a result of local yielding. Peak drained strength in all embankment fills and other foundation soils.</td>
<td>0.1</td>
<td>( Su = 50 + 0.13 \sigma_{vo}' )</td>
</tr>
<tr>
<td>3</td>
<td>Post-peak undrained shear strength achieved in the UGLU at 20% strain due to continued movement. Peak undrained strength triggered in other contractive embankment fills and foundation soils.</td>
<td>0.4</td>
<td>( Su = 36 + 0.11 \sigma_{vo}' )</td>
</tr>
<tr>
<td>4</td>
<td>Remolded undrained shear strength in the UGLU and peak undrained shear strengths in contractive fills and foundation soils.</td>
<td>&gt; 1</td>
<td>( Su = 22 + 0.03 \sigma_{vo}' )</td>
</tr>
<tr>
<td>5</td>
<td>Failed embankment at equilibrium in post-failure configuration with a factor of safety close to 1.0</td>
<td>&gt; 3</td>
<td>( Su = 22 + 0.03 \sigma_{vo}' )</td>
</tr>
</tbody>
</table>

Note:
1. \( \phi'_p \) = Effective friction angle; \( Su \) = Undrained shear strength; \( \sigma_{vo}' \) = Initial vertical effective stress (kPa) prior to failure. Refer to Figure 5.22 for selected Su values.
2. Maximum movements based on a 2 m thick UGLU below the embankment toe.
6.3 Analysis Sections

The embankment sections selected for the analyses are Sections C and D, shown on Figure 6.1. Figure 6.2 and Figure 6.3 show the geometry and zonation at these sections.

Section C is located near the center of the failure zone and was used to evaluate the stability of the embankment during Stages 1 to 4. The configuration of the model incorporates: the foundation soil and bedrock profiles as presented in Figure 5.13; the as-constructed embankment section as presented in Figure 2.6; and the pre-failure topography from the aerial LiDAR survey recorded in August 2013.

A 2 m deep excavation at the embankment toe (to approximately El. 930 m) was included in the model to reflect the site stripping in preparation for construction of a future toe buttress. The model domain extends 225 m upstream of the embankment set-out-line, with a transition from coarse-grained sandy tailings and fine-grained tailings “slimes” assumed 150 m upstream of the set-out-line.

Section D passes through the intact remnants of the failed embankment and was used to assess the stability of the embankment in Stage 5. The configuration of the model incorporates: the foundation soil and bedrock profiles as presented in Figure 5.14, the as-constructed embankment section as presented in Figure 2.6, and the post-failure geometry at Section D from the aerial LiDAR survey recorded in August 2014. The model extends 160 m upstream and downstream of the set-out-line.

A substantial portion of the upstream embankment fill at Section D was eroded during the tailings breach. The configuration of the embankment following initial embankment failure, but prior to breaching, was approximated based on an area-mass balance between the upthrust toe bulge and the crest downdrop, as shown in Figure 6.3.

6.4 Material Properties for Analyses

Material properties for the compacted till core (Zone S), foundation soils and bedrock units were discussed in Section 5. Material properties for other fill materials and impounded tailings are presented in Table 6.2. These properties were derived from as-built records, including: grain size distribution, in situ water content, and in situ dry density. Properties were also based on soil classification and index properties when other information was not available.
### Table 6.2 Summary of Material Properties for Failure Analysis – Fill Materials and Impounded Tailings

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Till Core Zone S</th>
<th>Fill Zone B</th>
<th>Rockfill Zone C</th>
<th>Fine Rockfill Zone T</th>
<th>Sand and Gravel Zone F</th>
<th>Coarse Bearing Layer Zone CBL</th>
<th>Cycloned Sand Zone CS</th>
<th>Random Fill Zone U</th>
<th>Upstream Drain</th>
<th>Coarse Tailings</th>
<th>Fine Tailings (Slimes)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Water content (%)</td>
<td>10.5</td>
<td>11.0</td>
<td>2.5</td>
<td>4.4</td>
<td>8.4</td>
<td>19.9</td>
<td>11.6</td>
<td>19.9</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Specific gravity</td>
<td>2.73</td>
<td>2.73</td>
<td>2.70</td>
<td>2.70</td>
<td>2.70</td>
<td>2.70</td>
<td>2.70</td>
<td>2.70</td>
<td>-</td>
<td>2.70</td>
<td>2.70</td>
</tr>
<tr>
<td>Saturation (%)</td>
<td>100</td>
<td>100</td>
<td>20</td>
<td>35</td>
<td>50</td>
<td>100</td>
<td>100</td>
<td>100</td>
<td>-</td>
<td>100</td>
<td>100</td>
</tr>
<tr>
<td>Average porosity</td>
<td>-</td>
<td>-</td>
<td>0.25</td>
<td>0.25</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Average void ratio</td>
<td>-</td>
<td>-</td>
<td>0.33</td>
<td>0.34</td>
<td>0.45</td>
<td>0.54</td>
<td>0.60</td>
<td>0.54</td>
<td>0.35</td>
<td>0.90</td>
<td>1.00</td>
</tr>
<tr>
<td>Unit weight (kN/m³)</td>
<td>22.8</td>
<td>22.7</td>
<td>20.4</td>
<td>20.6</td>
<td>19.8</td>
<td>20.7</td>
<td>18.1</td>
<td>20.7</td>
<td>-</td>
<td>18.6</td>
<td>18.1</td>
</tr>
<tr>
<td>Peak effective friction angle</td>
<td>33°</td>
<td>33°</td>
<td>40°</td>
<td>35°</td>
<td>34°</td>
<td>33°</td>
<td>32°</td>
<td>30°</td>
<td>32°</td>
<td>32°</td>
<td>28°</td>
</tr>
<tr>
<td>Undrained shear strength</td>
<td>0.38σ&lt;sub&gt;vo&lt;/sub&gt;</td>
<td>0.30σ&lt;sub&gt;vo&lt;/sub&gt;</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Horizontal hydraulic conductivity (k&lt;sub&gt;h&lt;/sub&gt;) (m/s)</td>
<td>2x10&lt;sup&gt;-8&lt;/sup&gt; (S-1)</td>
<td>2x10&lt;sup&gt;-9&lt;/sup&gt; (S-2)</td>
<td>2x10&lt;sup&gt;-8&lt;/sup&gt; (S-1)</td>
<td>2x10&lt;sup&gt;-9&lt;/sup&gt; (S-2)</td>
<td>4x10&lt;sup&gt;-2&lt;/sup&gt;</td>
<td>5x10&lt;sup&gt;-4&lt;/sup&gt;</td>
<td>1x10&lt;sup&gt;-3&lt;/sup&gt;</td>
<td>2x10&lt;sup&gt;-5&lt;/sup&gt;</td>
<td>1x10&lt;sup&gt;-3&lt;/sup&gt;</td>
<td>1x10&lt;sup&gt;-7&lt;/sup&gt;</td>
<td>1x10&lt;sup&gt;-9&lt;/sup&gt;</td>
</tr>
<tr>
<td>Vertical hydraulic conductivity (k&lt;sub&gt;v&lt;/sub&gt;) (m/s)</td>
<td>5x10&lt;sup&gt;-9&lt;/sup&gt; (S-1)</td>
<td>5x10&lt;sup&gt;-9&lt;/sup&gt; (S-2)</td>
<td>4x10&lt;sup&gt;-10&lt;/sup&gt;</td>
<td>5x10&lt;sup&gt;-2&lt;/sup&gt;</td>
<td>5x10&lt;sup&gt;-4&lt;/sup&gt;</td>
<td>1x10&lt;sup&gt;-4&lt;/sup&gt;</td>
<td>1x10&lt;sup&gt;-3&lt;/sup&gt;</td>
<td>2x10&lt;sup&gt;-6&lt;/sup&gt;</td>
<td>2.5x10&lt;sup&gt;-6&lt;/sup&gt;</td>
<td>1x10&lt;sup&gt;-3&lt;/sup&gt;</td>
<td>5x10&lt;sup&gt;-8&lt;/sup&gt;</td>
</tr>
<tr>
<td>Anisotropy ratio (k&lt;sub&gt;h&lt;/sub&gt;/k&lt;sub&gt;v&lt;/sub&gt;)</td>
<td>4</td>
<td>10</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>10</td>
<td>4</td>
<td>1</td>
<td>10</td>
<td>10</td>
</tr>
</tbody>
</table>

| Note:                                              |                  |             |                  |                      |                        |                              |                       |                  |                |                |                      |
|                                                   |                  |             |                  |                      |                        |                              |                       |                  |                |                |                      |
| 1. Specific gravity from as-constructed records. A value of 2.7 was adopted when other data was not available. Tailings specific gravity varies from 2.65 to 2.78, a median 2.7 was used. |                  |             |                  |                      |                        |                              |                       |                  |                |                |                      |
| 2. Average porosity from soil classification.     |                  |             |                  |                      |                        |                              |                       |                  |                |                |                      |
| 3. Unit weight calculated based on median water content, specific gravity, and percentage saturation. |                  |             |                  |                      |                        |                              |                       |                  |                |                |                      |
| 4. Hydraulic conductivity shown in table reflects adjusted values from calibration analysis conducted during the seepage analysis. |                  |             |                  |                      |                        |                              |                       |                  |                |                |                      |
| 5. Coarse tailings properties were established assuming consolidated peripheral-discharged beach sands with up to 30% fines and deposited mostly above water. Fine tailings (slimes) were assumed based on typical values for copper slimes deposited mostly below water. |                  |             |                  |                      |                        |                              |                       |                  |                |                |                      |
6.5 Pore Pressure Conditions Prior to Failure

A knowledge of the pore pressures in the tailings embankment and foundation soils is essential when assessing the stability of a tailings embankment because the in situ effective stresses and strength of the embankment fills and its underlying foundation are directly dependent on them.

Steady state seepage analysis was used to evaluate the pore pressure regime caused by seepage from the tailings pond through the embankment and foundation soils. This analysis is reported in Appendix III. Results were compared to pre-failure piezometric elevations recorded by piezometers installed in the till core (Zone S), upstream Zone U, and tailings beach. Seepage records from the Perimeter Embankment were also used to calibrate the model.

Additional pore pressures could have been induced in the UGLU by raising of the embankment. No piezometers in the UGLU were available to record such pore pressures. In absence of field data, consolidation analyses of the UGLU were undertaken to estimate the potential magnitude of the excess pore pressures. These analyses are reported in Appendix VI.

6.5.1 Steady-State Seepage Pore Pressures

Figure 6.4 shows the predicted total head contours and pore pressure conditions prior to failure within the embankment fills, tailings, and foundation soils. Salient observations pertinent to embankment stability include:

- Steady-state piezometric conditions in the foundation soils below the embankment are influenced by the strong seepage gradients underneath the till core of the embankment and the presence of the permeable glaciofluvial deposits (UGF/LGF) that are confined by the relatively impermeable LGLU, UGLU, and till units. Due to these factors, the predicted piezometric levels in the upper glaciolacustrine are 4 m to 6 m above the piezometric surface in the embankment.

- Predicted piezometric levels in the middle of the upper glaciofluvial unit are 6 m to 8 m above the phreatic surface in the embankment. These elevated levels are consistent with artesian conditions in a piezometer installed in SH14-01 (piezometric level 4.6 m above ground) and other observations of artesian conditions in the UGF during the post-failure site investigation (Section 5.9).

- The Upstream Toe Drain reduces the piezometric pressures in the tailings upstream of the core, thereby reducing the seepage gradients in the foundation below the core. A parametric analysis excluding the drain effect showed that piezometric levels upstream of the till core increased to nearly hydrostatic conditions. Seepage gradients beneath the till core increased accordingly as did piezometric levels within the UGLU, which increased to 7 m to 9 m above the piezometric surface in the embankment.
6.5.2 Transient Pore Pressures in the UGLU

One-dimensional consolidation analyses were conducted in two representative soil columns located along Section C near the crest (Column 1) and below the embankment mid-slope (Column 2). Figure 6.5 shows the excess pore pressure predicted at the middle of the UGLU at the two soil columns. The estimated pore pressures in the UGLU at the time of failure range from 97 kPa to 158 kPa below the embankment mid-slope and below the crest, respectively.

These construction-induced pore pressures are considered in parametric sensitivity analysis of the embankment stability.

6.6 Limit Equilibrium Stability Analysis

6.6.1 General

Slope stability analyses were conducted for Section C and Section D using the software Slope/W. The objective was to obtain the Factors of Safety (FoS) at the five hypothesized stages of dam failure in Table 6.1. Details of the analyses are reported in Appendix IV.

“Base Case” analyses were conducted for Stages 1 to 4 using the predicted steady state pre-failure seepage pore pressures from Figure 6.4. A number of additional sensitivity analyses were then conducted to assess: the detrimental effect of the 2 m deep excavation at the embankment toe, the beneficial influence of the Upstream Toe Drain, the detrimental effects of the construction-induced pore pressures in the UGLU, and the stability of the embankment with a flattened 2H:1V exterior slope. For comparison, the stability of the embankment for a hypothetical slip surface in the deeper overconsolidated LGLU was also assessed.

6.6.2 Stability Results for Section C

Figures 6.6a to 6.6d shows the “base case” analyses results for Stages 1 to 4 at Section C using the predicted steady state pre-failure seepage pore pressures from Figure 6.4. Figures showing the results from the sensitivity analyses can be found in Appendix II-B and II-C. Table 6.3 summarizes the calculated FoS.

<table>
<thead>
<tr>
<th>Case</th>
<th>Description</th>
<th>Factor of Safety (FoS)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Stage 1</td>
</tr>
<tr>
<td>Base Case</td>
<td>Steady-state seepage pore pressures.</td>
<td>1.27</td>
</tr>
<tr>
<td>1</td>
<td>No stripping and excavation at the embankment toe.</td>
<td>1.34</td>
</tr>
<tr>
<td>2</td>
<td>Without Upstream Toe Drain and elevated piezometric levels in tailings and foundation soils.</td>
<td>1.12</td>
</tr>
<tr>
<td>3</td>
<td>Inclusion of construction induced pore-pressures in the UGLU.</td>
<td>1.19</td>
</tr>
<tr>
<td>4</td>
<td>Effect of a flattening the downstream slope to 2H:1V.</td>
<td>1.59</td>
</tr>
<tr>
<td>5</td>
<td>Slip surfaces through LGLU using peak friction angle.</td>
<td>1.38</td>
</tr>
<tr>
<td>6</td>
<td>Slip surfaces through LGLU using residual friction angles.</td>
<td>1.19</td>
</tr>
</tbody>
</table>
Salient observations are:

- In the base case, with steady state pore water pressures, the FoS for Stage 1 was 1.27. With triggering of the undrained strength of the UGLU, the FoS reduced close to unity for Stage 2 and below unity for Stages 3 and 4.
- With the addition of excess pore pressures in the UGLU, FoS for Stage 1 dropped from 1.27 to 1.19. FoS for Stage 2 dropped from 1.02 to 0.98.
- Without the 2 m deep excavation at the embankment toe, the FoS increased from 1.27 to 1.34 for Stage 1 and from 1.02 to 1.10 for Stage 2, and only the FoS for Stage 4 fell below unity. The increases in FOS, especially for Stage 2, indicates that embankment failure may not have occurred on August 4, but this would require further analyses.
- The stability of the tailings embankment is very sensitive to pore pressures in the upstream tailings and embankment fills, and the corresponding seepage gradients at the foundation soils. The FoS dropped significantly from 1.27 to 1.11 for Stage 1 when the beneficial effect of the Upstream Toe Drain was removed.
- Flattening the slope to 2H:1V improves FoS for all the stages, with only Stage 4 FoS below unity. Embankment failure on August 4 is not predicted.
- The FoS for a hypothetical slip surface through the LGLU was 1.38 and 1.19 with peak and residual drained strengths, respectively (see Figures IV-C-1 and IV-C-2). Failure through the deeper LGLU unit is not predicted. This agrees with observations from inclinometers installed post-failure which did not show any post-failure movements in the LGLU.

6.6.3 Stability Results for Section D

Figure 6.6e shows the stability result for Stage 5 on the re-constructed Section D. FoS slightly above unity is calculated. This is consistent with the failed embankment coming to rest as the driving forces reduced (with drop in embankment crest) and resisting forces increased (with the upthrusted toe bulge).

The stability analyses for Section D rely on simplified but reasonable assumptions of post-failure pore water pressures and embankment geometry. The results are intended only to demonstrate the validity of our failure hypothesis.

6.7 Numerical Stress Analysis

6.7.1 General

As the behaviour of the UGLU is of primary concern in the assessment of the dam failure mechanism, numerical stress and deformation analyses using the 2D Finite difference modelling software, FLAC, were conducted to provide insights into the following questions:

- What was the global Factor of Safety (FoS) of the dam corresponding to peak drained shear strength and peak undrained shear strengths in the UGLU?
- What was the local FoS within the UGLU corresponding to drained strengths and were they low enough to cause local yielding of the clay and trigger of rapid undrained shearing?
- If yielding occurred, would continued displacement or straining have led to significant strain softening or even remolding of the UGLU?
- What would be the nature of the dam displacements caused by the failure of the dam?

To answer these questions, the numerical analyses were conducted as follows:

- Step 1: Conduct FoS analysis using the strength reduction technique in FLAC/Slope module to calculate the global FoS of the dam using peak drained and peak undrained shear strengths for the UGLU.
- Step 2: Conduct static stress analysis using drained strength parameters in the UGLU to establish in situ stresses and FoS against local yielding within the UGLU just prior to failure.
- Step 3: Invoke undrained behaviour in the UGLU, as shown in Figure V-1, to assess the stability of the dam and predict its deformation.

Details of the FLAC analyses are reported in Appendix V.

### 6.7.2 Modelling of the UGLU

Figure 6.7 shows the assumed shear stress-strain behaviour of the UGLU under drained and undrained conditions based on the results of the advanced laboratory test work. The peak friction angle is reached under drained loading conditions at 15% shear strain. Peak undrained shear strength is attained at approximately 5% shear strain, and with further straining, the undrained strength drops to its post-peak value at about 20% shear strain. The remolded undrained shear strength is reached at approximately 60% shear strain.

### 6.7.3 Global Factor of Safety

The factor of safety (FoS) against slope failure is computed by FLAC using a procedure known as the “strength reduction technique” (Dawson et al. 1999). In this technique, the shear strength of the material is progressively reduced to bring the slope to the state of limiting equilibrium. The factor, by which the strengths are reduced to the state of limiting equilibrium, is defined as the FoS.

Figure 6.8 shows the global FoS for the embankment with peak drained and peak undrained strengths for the UGLU and assuming hydrostatic pore pressure conditions. The global FoS of the dam is 1.21 with the peak drained strength in the UGLU and reduces to less than unity with the peak undrained shear strength. The FoS with peak strength is relatively low and could have allowed some portions of the UGLU to locally strain beyond the available peak drained strength, thereby initiating a progressive undrained failure mechanism in the UGLU. Under such conditions, failure is predicted by the strength reduction analyses (FoS = 0.94). These results are consistent with the findings from the limit equilibrium analyses in Section 6.6.
6.7.4 Local Factors of Safety and Yielding

Two-dimensional plane-strain analyses were conducted with FLAC to estimate in situ stresses in the embankment and foundation soils. The analyses were carried out in six loading stages as shown in Figure V-8 in Appendix V. Figure 6.9 shows the results for the embankment configuration at Stage 6B completed in 2010 and the final Stage 9 completed just prior to failure.

The results for Stage 9 show that:

- Vertical effective stresses in the UGLU, range from 100 kPa to 760 kPa beneath the dam.
- The average pre-consolidation stress of 400 kPa was exceeded below the lower third of the dam slope. Hence, the majority of the UGLU below the dam was normally consolidated and contractive during shear.
- The local FoS_D, corresponding to the drained shear strength, were predicted to be close to unity within portions of the UGLU. The location of yielding is predicted to occur near the dam toe where the shear stresses induced by the steep dam slope are high.
- The local FoS_Up within the UGLU, corresponding to peak undrained shear strength, is substantially less than 1 under virtually the entire dam slope. This result predicts rapid dam failure would occur following an initiation of local yielding.

By comparison for Stage 6B, the local FoS_D within the UGLU was greater than 1.2 and the local FoS_Up was above unity. While these values are low, failure would not be predicted by the model at the end of Stage 6B and this supports the validity of the model.

For Stage 9, the drop in the global FoS following yield of the UGLU and the associated displacements within the UGLU could have triggered the spread of undrained behaviour across the UGLU and further strain softening in zones that had already reached their peak undrained shear strengths. If unchecked, the progression of shearing would have ultimately led to mobilization of remolded shear strengths within the UGLU as the dam failed. A deformation analysis is reported in Section 6.7.5 to check this possible failure mechanism.

Figures V-12 and V-13 summarize the results for a parametric sensitivity case without the UGLU for the ends of Stage 6B and final Stage 9. In this case, where the UGLU was replaced by UGT, the results show that the local FoS_D strength is greater than 2.0 in Stage 6B and greater than 1.5 in Stage 9. Because the strength of UGT is much higher than the strength of UGLU, higher local FoS were obtained in this sensitivity analysis and dam failure is not predicted by the model.

6.7.5 Embankment Deformation Analysis

A simplified two-dimensional plane-strain static deformation analysis was conducted using FLAC to demonstrate that triggering of undrained shear in the UGLU would have caused the failure of the dam and to predict the mode and patterns of the dam deformations. This analysis was conducted in two steps: in the first step, in situ stresses within the dam prior to the failure were estimated using drained strength parameters for all materials including the UGLU; and in the second step, the
undrained behaviour of the UGLU was invoked and the strain dependent behaviour of UGLU shown in Figure 6.7 was modelled.

Figure 6.10 show the deformed shape and mobilized shear strain within the UGLU for two time intervals selected to represent early and final stages of failure. Salient observations are:

- For the early stages of failure, when the maximum horizontal displacement within the UGLU was approximately 0.3 m, the shear strain within the UGLU exceeded 5% with a number of zones experiencing more than 60% strain, indicating straining of the UGLU to its remolded strength had occurred in these areas.

- For the final stages of failure, when the maximum horizontal displacement within the UGLU was approximately 6 m, the settlement upstream of the dam was approximately 5 m and the upthrust near the toe reached about 4 m. Rotation on the backscarp is also predicted. The shear strain within the entire width of the UGLU exceeds 60% indicating the UGLU has been fully reduced to its remolded shear strength.

These results illustrate the progressive weakening of the UGLU and provide an indication of the mode of deformation and magnitude of movements.

The simplified analysis demonstrates that the dam would have undergone a sudden and rapid failure following triggering of undrained shear in the upper UGLU. The predicted sliding of the dam and discrete shear deformations in Figure 6.10 are consistent with the actual post-failure observations of upthrusting at the dam toe and the shear plane found through the lower portion of the till core. The rotation on the backscarp of the failure is also consistent with the back-tilting of the construction lifts observed in the abutments of the failed dam.

The observed fully remolded strengths at final stages of the failure are also consistent with observations of low undrained shear strengths in CPT soundings and VST testing below the failed embankment.
7 CONCLUSIONS

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 Perimeter Embankment. 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 final trigger for the failure was the recent excavation at the toe in 2013 and raising of the embankment with the steep outer slope of 1.3H:1V.

At the time of failure, the Factor of Safety (FoS) of the dam was calculated using limit equilibrium methods to be 1.27 using the peak drained strength of the UGLU and the pre-failure pore pressures estimated by seepage analyses. The FoS reduces 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 upthrust 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 initiate the subsequent dam breach.

A sliding plane in the UGLU is 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.

The existence of a pre-existing shear plane in the UGLU was considered as a possible factor in the failure. Samples retrieved from outside the failed dam were examined for the presence of shear planes or other distortions of the varved clay structure 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.
8 COMPARISON TO AZNALCOLLAR TAILINGS DAM FAILURE

It is notable that the Aznalcollar Tailings Dam near Seville, Spain was also constructed with a 1.3H:1V exterior slope and failed at a height of 28 m in 1998. It is likely the closest direct comparable in the mining industry to the Mount Polley dam failure.

This dam failed by sliding on a heavily overconsolidated, high plastic, marl clay foundation. While overconsolidated and dilative in shear, the marl clay displayed remarkable “brittle” behavior with substantial loss of frictional strength once the peak strength was exceeded. Failure was initiated by local yielding of the clay, which initiated a progressive failure within the dam foundation. Contributing factors to the local yielding were the steep dam slope and excess pore pressures generated in the clay during the dam construction.

The Aznalcollar tailings facility stored pyrite and pyroclastic tailings in two adjacent cells. At the time of failure, the tailings pond in the pyroclastic cell was maintained against the dam crest to keep the stored tailings saturated to prevent oxidation. Failure of the dam released the entire 5.5 million m³ of stored pond water and 1.5 million m³ of tailings slurry from both the pyrite and pyroclastic cells.

Table 8.1 compares some of the features of the Mount Polley and Aznalcollar tailings dams. Ultimately, both dams failed because the exterior slopes were too steep for the foundation conditions.

Table 8.1 Comparison of Mount Polley and Aznalcollar Tailings Dam Failures

<table>
<thead>
<tr>
<th>Feature</th>
<th>Mount Polley Tailings Dam</th>
<th>Aznalcollar Tailings Dam</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dam Height</td>
<td>40 m</td>
<td>28 m</td>
</tr>
<tr>
<td>Dam Construction Method</td>
<td>Modified upstream method with “core” of compacted till and filter zones</td>
<td>Downstream method with lining of compacted soil on the upstream dam face and filter zones</td>
</tr>
<tr>
<td>Exterior Dam Slope</td>
<td>1.3H:1V in upper slope 1.8H:1V at toe</td>
<td>1.3H:1V</td>
</tr>
<tr>
<td>Foundation Clay Involved in Failure</td>
<td>Glaciolacustrine Varved Clay</td>
<td>Marine Marl Clay</td>
</tr>
<tr>
<td>Clay Thickness</td>
<td>2 m maximum</td>
<td>Over 25 m</td>
</tr>
<tr>
<td>Clay Properties</td>
<td>Liquid Limit = 40 – 70% Clay Fraction = 40 – 70%</td>
<td>Liquid Limit = 55 – 75% Clay Fraction = 45 – 75%</td>
</tr>
<tr>
<td>Consolidation State</td>
<td>Lightly Overconsolidated Liquidity Index = 0.5</td>
<td>Heavily Overconsolidated Liquidity Index = 0.13</td>
</tr>
<tr>
<td>Behavior Under Shear</td>
<td>Contractive</td>
<td>Dilative</td>
</tr>
<tr>
<td>Construction Excess Pore Pressures</td>
<td>Low based on post-failure pore pressure analyses</td>
<td>High based on piezometers installed after failure event</td>
</tr>
<tr>
<td>Rate of Failure</td>
<td>&lt; 2 hours 10 m of dam displacement</td>
<td>&lt; 2 hours Up to 50 m of dam displacement</td>
</tr>
<tr>
<td>Water Pond at Time of Failure</td>
<td>High pond level against dam crest following high spring runoff</td>
<td>High pond level against dam crest to keep pyroclastic tailings saturated</td>
</tr>
<tr>
<td>Consequences of Failure</td>
<td>Release of 17 million m³ of water (including interstitial water) and 8 million m³ of tailings solids</td>
<td>Release of 5.5 million m³ of water and 1.5 million m³ of tailings slurry</td>
</tr>
</tbody>
</table>
9 CLOSING

The work presented in this report was managed and performed under the direction of Howard Plewes, M.Sc., P.Eng.

This report is an instrument of service of Klohn Crippen Berger Ltd. The report has been prepared for the use of BC Ministry of Energy and Mines for the specific application to the Mount Polley Dam Failure investigation. In this report, Klohn Crippen Berger has endeavoured to comply with generally-accepted professional practice common to the local area. Klohn Crippen Berger makes no warranty, express or implied.

KLOHN CRIPPEN BERGER LTD.

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Geotechnical Engineer

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Principal, Geotechnical Engineer
REFERENCES


Itasca. 2014. FLAC, Fast Lagrangian Analysis of Continua. Itasca Consulting Group, Inc. Minneapolis, MN.


FIGURES (CONT’D)

Figure 5.15 Summary of Consolidation Test Results on UGLU and LGLU
Figure 5.16 Undrained Shear Strength (Su) Nkf from CPT Soundings in UGLU and LGLU
Figure 5.17 Undrained Shear Strength (Peak and Remolded) from Vane Shear Tests in UGLU
Figure 5.18 Geologic Section C Undrained Shear Strength Data
Figure 5.19 Geologic Section D Undrained Shear Strength Data
Figure 5.20 Peak and Residual Shear Strengths of Thin-Walled Piston, Sonic Core, and Block Specimens from Direct Shear Tests on UGLU Unit
Figure 5.21 Stress Paths from Triaxial Compression Tests on UGLU Unit
Figure 5.22 Summary of Monotonic Direct Simple Shear Tests on UGLU Unit and Proposed Undrained Shear Strength Functions
Figure 5.23 Normalized Stress-Strain from Monotonic Direct Simple Shear Tests on UGLU
Figure 5.24 Representative Photographs of LGLU
Figure 5.25 Top Elevation of LGLU Encountered in Sonic Core Holes
Figure 5.26 Empirical Strength Relationships for Clays
Figure 5.27 Top Elevation of Bedrock Encountered in Sonic Core Holes
Figure 5.28 Geological Section F with Sonic Core Holes and Geophysical Survey Data
Figure 5.29 Post-Failure Piezometer Location Plan
Figure 5.30 Section G (AMEC) with Piezometer Data
Figure 5.31 Section C (KCB) with Piezometer Data from Section G’ (Knight Piesold)
Figure 5.32 Section D (AMEC) with Piezometer Data
Figure 5.33 Seepage Gradients in Foundation Soil and Bedrock
Figure 5.34 2014 Inclinometers Showing Movements Measured in UGLU
Figure 5.35 Inclinometer Reading SH14-03
Figure 5.36 Inclinometer Reading SH14-04
Figure 5.37 Inclinometer Reading SH14-06
Figure 5.38 Inclinometer Reading SH14-09
Figure 5.39 Inclinometer Reading SH14-16
Figure 5.40 Evidence of Large Embankment Displacements
Figure 6.1 Location of Sections C and D Use in Analyses
Figure 6.2 Section C Pre-Failure Geometry and Zonation
Figure 6.3 Section D Post-Failure Geometry and Zonation
Figure 6.4 Steady-State Seepage Analyses Results for Pre-Failure Conditions
Figure 6.5 Predicted Excess Pore Pressure at the Middle of UGLU
Figure 6.6 Slope Stability for Stages 1 to 5
Figure 6.7 Shear Stress-Strain Behaviour of UGLU Under Drained and Undrained Conditions
Figure 6.8 Global Factors of Safety for Peak Drained and Undrained Shear Strengths in UGLU
Figure 6.9 Mobilized Stresses and FoS Against Yielding at the End of Stage 6B and Stage 9
Figure 6.10 Deformed Shape of the Embankment and Shear Strain in UGLU at Maximum Horizontal Displacements of 0.3 m and 6 m
NOTES:
1. Refer to reports referenced in Table 3.1.
2. Measurements were based on available instrumentation data from 2013-2014.
3. Data used is from the Environment Canada Point Inlet weather station.
4. Water quality data was obtained from the Environment Canada Point Inlet weather station.
5. The data shown is representative of the equipment used in the observation period of 2013-2014.
a) View of upthrust toe bulge in Zone 1
   (looking north west)

b) View of washed rockfill debris on upthrust toe bulge in Zone 2
   (looking north east)

c) Water washed rock debris on dam slope in Zone 2

d) View of water erosion of upthrust toe bulge in Zone 2
   (looking south east)
a) View of eroded toe bulge in zones 3 and 4 (looking south west)

b) View of exposed clay core in zone 8 (looking north east)

c) Close-up view of exposed clay core in zone 8 (looking south)

d) View of exposed construction lifts in zone 7 (looking north east)
a) Close-up view of exposed horizontal construction lifts in Zone 7

b) View of exposed construction lifts in Zone 9 (looking north west)

c) View of relaxation scarps in left abutment of dam breach in Zone 6
TIME INTERVAL I - PRE-FAILURE

TIME INTERVAL II - DAM FAILURE AND INITIAL OVERTOPPING

TIME INTERVAL III - DOWNCUTTING OF EMBANKMENT BY CONCENTRATED WATER FLOW

TIME INTERVAL IV - DOWNCUTTING OF EMBANKMENT IN BREACH COMPLETE

LEGEND:

- [Legend items]

NOTES:
1. [Note 1]
2. [Note 2]
3. [Note 3]

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MT. PERRY DAM FAILURE
ASSESSMENT OF FAILURE MECHANISM

HYPOTHESES SEQUENCE OF EMBANKMENT FAILURE

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[Drawing details]
a) 3D Model of Till Core Exposed in Right Abutment of Breach

b) Projection ofMapped Shear Plane Surface in Till Core

c) Shear Plane – Note Color Change in Till Core at Shear Zone

d) Shear Zone with Tailings Sand Infill

e) Till Core Separating Preferentially Along Shear Plane

f) Softening and Disturbance Within the Shear Zone
a) INCLINED GLACIOLACUSTRINE CLAY EXPOSED IN TRENCH SIDEWALL

b) HAND-EXCAVATED BLOCK SAMPLE OF CLAY WITH LAMINATIONS INCLINED AT APPROXIMATELY 40°
a) UGT - FREE FIELD DOWNSTREAM OF FAILED EMBANKMENT

SH14-22 (Top layer of UGT) Approximate El. 927.5 m to 921.3 m
Water content = 14.2%; Liquid limit = 26%; Plastic index = 13%; Clay fraction = 16%

b) UGT - BELOW FAILED EMBANKMENT AND BREACH AREA

SH14-03 (Top layer of UGT) Approximate El. 927.6 m to 921.5 m
Water content = 11.8%; Liquid limit = 24%; Plastic index = 12%; Clay fraction = 19%
BC MINISTRY OF ENERGY AND MINES

MT. POLLEY DAM FAILURE ASSESSMENT OF FAILURE MECHANISM
CPT SHEAR WAVE VELOCITY MEASUREMENTS IN GLACIAL TILLS

Upper Glacial Till (UGT)
Middle Glacial Till (MGT)
Lower Glacial Till (LGT)

Shear wave velocity (m/s) vs. elevation (m)

- Below the Failed Embankment and Breach Area
- Free Field Downstream of Failed Embankment
a) UGLU - FREE FIELD DOWNSTREAM OF FAILED EMBANKMENT

Sub-horizontal varved bedding. Water content = 35.7%; Liquid limit = 60%; Plastic index = 20%; Clay fraction = 58%

b) UGLU - BELOW FAILED EMBANKMENT AND BREACH AREA

Folded laminations/Soft consistency. Water content = 37%; Liquid limit = 44%; Plastic index = 17%
### BC MINISTRY OF ENERGY AND MINES

#### ASSESSMENT OF FAILURE MECHANISM

**SUMMARY OF CONSOLIDATION TEST RESULTS ON UGLU AND LGLU**

**CLIENT PROJECT TITLE**

**PROJECT NO.**

**TO BE READ WITH KLOHN CRIPPMEN BERGER REPORT DATED **

---

**SAMPLE DEPTHS (m)***

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Depth (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>RH14-03A SS03</td>
<td>12.2</td>
</tr>
<tr>
<td>RH14-32A SS02</td>
<td>9.3</td>
</tr>
<tr>
<td>RH14-10 SS01</td>
<td>10.1</td>
</tr>
<tr>
<td>RH14-10 SS02</td>
<td>10.7</td>
</tr>
<tr>
<td>RH14-22 SS02</td>
<td>9.3</td>
</tr>
<tr>
<td>SH14-10A 17 (LGLU)</td>
<td>14.4</td>
</tr>
</tbody>
</table>

---

**VOID RATIO**

- **VERTICAL EFFECTIVE STRESS (kPa)**
  - RH14-03A SS03 (UGLU)
  - RH14-10 SS01 (UGLU)
  - RH14-22 SS02 (UGLU)
  - RH14-10 SS02 (UGLU)
  - TP14-01 BS2 (UGLU)
  - RH14-22A SS02 (UGLU)
  - SH14-10A 17 (LGLU)

---

**COEFFICIENT OF CONSOLIDATION, Cc (cm²/sec)**

- **VERTICAL EFFECTIVE PRESSURE a’ (kPa)**
  - RH14-03A SS03 (UGLU)
  - RH14-10 SS01 (UGLU)
  - RH14-22 SS02 (UGLU)
  - RH14-10 SS02 (UGLU)
  - TP14-01 BS2 (UGLU)
  - RH14-22A SS02 (UGLU)
  - SH14-10A 17 (LGLU)

---

**HYDRAULIC CONDUCTIVITY, k (cm/s)**

- **VERTICAL EFFECTIVE PRESSURE a’ (kPa)**
  - RH14-03A SS03 (UGLU)
  - RH14-10 SS01 (UGLU)
  - RH14-22 SS02 (UGLU)
  - RH14-10 SS02 (UGLU)
  - TP14-01 BS2 (UGLU)
  - RH14-22A SS02 (UGLU)
  - SH14-10A 17 (LGLU)
a) UGLU - INTERPRETED UNDRAINED STRENGTH FROM CPT TESTING

![Graph showing undrained shear strength (Su) against elevation (m) for UGLU samples.](image1)

- Below the Failed Embankment and Breach Area
- Free Field Downstream of Failed Embankment

b) LGLU - INTERPRETED UNDRAINED STRENGTH FROM CPT TESTING

![Graph showing undrained shear strength (Su) against elevation (m) for LGLU samples.](image2)

- Below the Failed Embankment and Breach Area
- Free Field Downstream of Failed Embankment

---

To be read with Klohn Crippen Berger Report dated May 01, 2015.
MT. POLLEY DAM FAILURE
ASSESSMENT OF FAILURE MECHANISM
UNDRAINED SHEAR STRENGTH (PEAK AND REMOLDED)
FROM VANE SHEAR TESTS IN UGLU

Date & Time: 01/05/2015 12:33

BC MINISTRY OF ENERGY AND MINES

PROJECT No. M09954A01

May 01, 2015
PEAK SHEAR STRENGTH

RESIDUAL SHEAR STRENGTH

* Specimen pre-cut with wire prior to conducting residual shear strength test
UGLU THIN-WALLED SAMPLES

Failed across bedding planes

UGLU BLOCK SAMPLES

Failed along bedding planes

TO BE READ WITH KLOHN CRIPPEN BERGER REPORT DATED May 01, 2015

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May 01, 2015

CLIENT
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PROJECT
MT. POLLEY DAM FAILURE

TITLE
ASSESSMENT OF FAILURE MECHANISM

STRESS PATHS FROM TRIAXIAL COMPRESSION TESTS ON UGLU

PROJECT No.
M09954A01

No.
5.21
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SUMMARY OF MONOTONIC DIRECT SIMPLE SHEAR TESTS ON UGLU AND PROPOSED UNDRAINED SHEAR STRENGTH FUNCTIONS

TO BE READ WITH KLOHN CRIPPEN BERGER REPORT DATED ___________________

May 01, 2015
a) UGLU - MONOTONIC DIRECT SIMPLE SHEAR TESTING

Note:

$\tau$ - Shear Stress

$\sigma_{vc}'$ - Consolidation stress
a) LGLU - FREE FIELD DOWNSTREAM OF FAILED EMBANKMENT

SH14-10 (LGLU layer) Approximate El. 917.4 m to 915.5 m
Wavy/undulating bedding; Water content = 20.7%; Liquid limit = 31%; Plastic index = 17%; Clay fraction = 24%

b) LGLU - BELOW FAILED EMBANKMENT AND BREACH AREA

SH14-04 (LGLU layer) Approximate El. 917.1 m to 916.7 m
Wavy/inclined bedding; Water content = 26%
a) Lateral and vertical displacements of upthrust toe bulge

b) Back-tilt of construction lift interfaces in Zone C exposed in left abutment

c) UGLU upthrust 6.5 m vertically in TP14–01 at embankment toe

d) Shear plane through remnant till core exposed in right abutment (looking southeast)
a) Excess pore water pressure with time in the middle of UGLU column 1 below dam crest

b) Excess pore water pressure with time in the middle of UGLU column 2 below mid-slope

NOTES:
1. This figure should be read in conjunction with Appendix V.

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NOTES:
1. Drained and undrained shear strengths were taken as follows:
   - Drained effective friction angle, \( \phi = 22 \) degrees
   - Peak undrained shear strength, \( S_u = 50 + 0.13\gamma_s' \) kPa
   - Post-peak undrained shear strength, \( S_u = 36 + 0.11\gamma_s' \) kPa
   - Remolded undrained shear strength, \( S_u = 22 + 0.07\gamma_s' \) kPa
2. The drained and undrained strengths are based on stresses on the horizontal plane. UGLU was divided into four layers and the parameters corresponding to each layer are plotted.
1) Deformed shape magnified by 5 times (maximum horizontal displacement within UGLU: ~0.3 m)

2) Deformed shape not magnified (maximum horizontal displacement within UGLU: ~6 m)

a) Deformed shape of the dam

b) Shear strain in the upper glaciolacustrine unit

Shear strain in UGLU:

- Red: 0
- Dark red: 50m/°
- Light red: 35m/°
- Yellow: 20m/°
- Green: 10m/°
- Blue: 5m/°
- Purple: 2.5m/°