Independent Expert Engineering Investigation and Review Panel

Report on Mount Polley Tailings Storage Facility Breach

Appendix H: Breach Analysis

January 30, 2015

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1.0 DEVELOPMENT OF DETAILED SECTION FOR ANALYSIS

The detailed section utilized for analysis in the vicinity of the breach needed to be developed using a variety of information because, within the background information reviewed, no single section existed that the Panel considered to be representative of the breach area. Therefore, the Panel had to develop a representative detailed section for analysis.

AMEC's Section D, located at about Sta. 3+985 to the right of the breach is the closest AMEC section to the breach area. The dam zoning details at Section D are different than the breach area because Section D is located where a localized downstream cyclone sand trial area was constructed (creating a cyclone sand zone downstream of the core) that is not present within the breach area (see MP00014 and MP00015 for details).

AMEC's Section G located at about Sta. 4+460, is situated to the left of the breach. Recent AMEC documentation shows the same dam zoning on Section D and G. The downstream cyclone zone is also not present at AMEC's Section G (see Figure 3.1 and 3.3 in MP00014 for actual extent of downstream cyclone sand zone). To establish representative dam zoning for AMEC's Section G, particularly for the early stages of dam construction, it is necessary to utilize earlier versions of Knight Piésold's (KP) Section G. The location of KP's Section G, at about Sta. 4+300, is within the breach area and different from AMEC Section G, as AMEC changed the location of Section G when they took over as EOR. Later versions of KP drawings for Section G [1] also contain inconsistencies with KP's earlier versions of drawings. [2]

Dam zoning for the detailed section used in the Panel's analyses was determined by examining the zoning shown on as-built drawings for every construction stage. [3] The constructed works shown for a given stage were each combined to create a detailed section for analysis.

Preconstruction ground lines were estimated from using topographic data available prior to the start of construction of the tailings storage facility (TSF). Topographic data collected prior to the start of construction of the TSF used different datums, either the mine datum or the TSF datum. Preconstruction ground lines were adjusted, where necessary, to the TSF datum, the datum utilized for recently collected LiDAR survey data.

To estimate the downstream slope geometry that existed in the breach area immediately prior to the breach, pre- and post-breach LiDAR data from August 2013 and 2014 were compared. First, the LiDAR data was compared outside the breach area to confirm dam raising in the vicinity of the breach did not include increasing the width of the Zone C, only adding Zone C material at the crest of the Zone C slope that existed at the end of the 2013 construction season. Drawing H1 shows further details.

1 MP00031, MP00033 AND MP00036
2 MP00019, MP00012, MP10032, MP00072 AND MP00038
3 MP00019, MP00012, MP10032, MP00072, MP00038, MP00031, MP00033, MP000034, MP00036, MP00041, MP00047 and MP00044

To estimate the top of Zones C, S and U that existed in the breach area immediately prior to the breach, the post-breach LiDAR data outside and either side of the breach area was projected into the breach area. The average of the projected surfaces was then compared to elevations noted in daily and weekly reports to estimate representative top elevation profiles for each zone within the breach area. Drawing H1 provides a summary of the ground lines utilized to estimate the geometry within the breach area that existed immediately prior to the breach.

At the time of the breach, a shallow toe excavation was present at the toe of the dam along the majority of the length of the breach area. The excavation was completed in November 2013 as part of preparations for construction of a future buttress. An as-built survey of the toe excavation was utilized to establish the geometry at the toe of the dam at the time of the breach. The datum used for the as-built survey did not match the TSF datum and a manual adjustment was required, which comprised matching the excavation surface with a logical location from the August 2013 LiDAR. Drawing H2 shows how the toe excavation geometry varied within the breach area. Drawing H1 shows that the toe excavation within the breach area was close to the estimated pre-construction ground line.

The variation of the dam geometry within the breach area was also reviewed as summarized in Drawing H2. The downstream slope of Zone C is similar throughout the breach area. Downstream of the dam toe, the shallow toe excavation, where present, is also similar. The shallow toe excavation is not present along the entire length of the breach area but where it is absent near the right side of the breach, a drainage ditch (deeper than the shallow toe excavation) is present in relatively close proximity to the dam toe.

The pond elevation shown on Drawing H1 was determined from daily reports. The water table elevation within and behind the dam was reviewed using piezometer data available at KP's Section G (See Drawing F2 for piezometer locations). The water levels measured in the piezometers in July of each year between 2007 and 2014 are shown on Drawing H3. The piezometer data suggests the water table is reduced by the presence of the upstream toe drain.

2.0 CONSOLIDATION ANALYSIS

2.1 OVERVIEW

Consolidation analyses to examine dissipation of pore pressures during staged construction were completed in support of the triggering analysis and evaluation. Key inputs into the consolidation analyses included soil stratigraphy, loading rate and soil consolidation parameters.

Soil parameters to be considered for the consolidation analyses include both the vertical and horizontal coefficients of consolidation (c_v and c_h respectively) and the coefficient of consolidation varies with vertical stress level.

2.2 SOIL STRATIGRAPHY

Soil stratigraphy is presented in Appendix D. Selecting a soil stratigraphy for consolidation analyses requires careful consideration of drainage paths and drainage boundary conditions. The soil profile utilized for the consolidation analyses was generally consistent with that shown on Drawing D13 in Appendix D and is noted on the consolidation analyses outputs. The presence of the Glaciofluvial layer, within the Lower Tills, and its relative continuous distribution is a key element in the consolidation analyses as it provides a lower drainage boundary. CPT dissipation data in this layer confirms it has sufficient permeability to act as a drainage layer.

2.3 LOADING RATE

Consolidation analyses were conducted for staged embankment loading. Understanding the loading sequence is critical for the analysis. In this case, the embankment loading rate is further complicated by variable loading rates between different materials within the Dam, namely the core (Zone S), rockfill shell (Zone C), upstream fill (Zone U) and water table. Zone S, Zone C and Zone U were often constructed to different elevations at each stage.

Estimated as-built elevations within the breach area (between approximately Sta. 4+200 and 4+300) were obtained from:

- as-built drawings
- daily and weekly construction reports by AMEC and Mount Polley Mining Corporation
- construction photographs
- nuclear densometer test summary sheets

The approximate time (in months) used to delineate the start and end of construction for each stage was obtained from the construction reports.

Elevations within the breach area prior to Stage 7 were based on reported elevations on the as-built drawings. From the information provided, there was insufficient information to determine the rate of rise of the individual zones on a month-to-month basis prior to Stage 7. A point was set about mid-way between the start and end of the stage to reflect the rate of rise as a step-wise increase, rather than a constant rate measured from start to end of construction.

Elevations within the breach area from Stage 7 to August 3, 2014 were based on elevations and chainages noted on daily and weekly reports, construction photographs and nuclear densometer test summaries.

The general rate of loading utilized in the consolidation analyses is shown in **Figure 3.2.3** of the main report. Figures H.A1-1 through H.A1-5 in Attachment H1 of this Appendix contain the detailed rate of loading utilized for the consolidation analyses.

2.4 COEFFICIENT OF CONSOLIDATION

2.4.1 Coefficient of Vertical Consolidation, C_V

Values of c_V were obtained through lab testing on undisturbed samples using oedometer and triaxial test data. The value of c_V for a given soil typically decreases with increasing confining stress, commonly when the preconsolidation pressure is reached.

Estimates of c_V obtained from oedometer and triaxial testing are contained in Appendix E, Attachment 2. The attachment also includes summary plots comparing the variation in c_V with confining stress grouped by inferred soil unit.

Data obtained for the Upper Till and Lower Tills indicated that c_V does not vary with confining stress. This may be the result of the relative difference in size between the specimen being tested and the gravel particles present in these samples. The c_V values estimated in Upper Till material during triaxial testing were consistently less than the values measured in the oedometer test. Average c_V values from oedometer testing for the Upper Till and Lower Tills obtained from the oedometer testing were utilized in the consolidation analyses.

Data obtained for the Upper GLU shows a general decrease in c_V at the average preconsolidation pressure of about 400 kPa. The c_V values estimated in Upper GLU material during triaxial testing were within the range of values measured during oedometer testing, but generally below the average. Average c_V values for a confining stress less than and greater than 400 kPa were utilized in the consolidation analyses for the Upper GLU.

2.4.2 Coefficient of Horizontal Consolidation, c_h

Values of c_h can be obtained by conducting dissipation tests during CPT testing. CPT dissipation curves and a summary of the c_h values estimated using the CPT testing are provided in Appendix D, Attachment 3. The summary includes an average c_h value measured for different inferred soil units.

The values were estimated using the methods outlined by Teh & Houlsby (1991). The Rigidity Index (IR) was determined using interpreted shear modulus and undrained shear strength estimates from interpretation of CPT data. It should be noted that estimation of c_h values using the CPT cannot provide information related to the change in c_h as the confining stress changes.

Values for c_h in the Basal Till and Glaciofluvial layers were highly variable. Based on field observations of soil conditions and comparison to lab data, lower values of c_h are believed to be more representative for the Basal Till.

For the consolidation analyses, anisotropy was not considered for the Upper Till and Lower Tills but was considered for the Upper GLU. Based on a comparison between the c_h values from the CPT dissipation testing in Upper GLU materials and c_v values from oedometer testing on Upper GLU, the ratio of c_h/c_v was estimated to be about 30 to 40. A value of 30 was utilized in consolidation analyses where anisotropy was considered.

2.4.3 Summary

Table D.4.3 provides a general summary of the c_v and c_h values estimated.

TABLE D.4.3 – SUMMARY OF C_V AND C_H VALUES

SOIL UNIT	AVERAGE C_V (CM/SEC^2)	AVERAGE C_H (CM/SEC^2)
Upper Till	3×10^{-3} (oedometer) 8×10^{-4} (triaxial)	4×10^{-2}
Upper GLU	2×10^{-3} (average) 3×10^{-3} for < 400 kPa 7×10^{-4} for > 400 kPa	8×10^{-2}
Basal Till	7×10^{-3}	4×10^{-4} (min) 2×10^{-1} (average)
Lower GLU	6×10^{-3}	6×10^{-2}
Glaciofluvial	N/A	5×10^{-1}

2.5 CONSOLIDATION ANALYSES

2.5.1 One-Dimensional Consolidation Analysis

One-dimensional consolidation analyses were completed using the program Settle3D. The program calculates three-dimensional elastic stress distributions with depth using applied surface loads. The program analyses one dimensional consolidation in the vertical direction.

The complex loading sequence shown in Figure H.A1-5 (Attachment H1) was converted to equivalent surface loads and input into the model using the loading curve in Figure H.A1-1 (Attachment H1). The results of the Settle3D analyses are presented in Figure H.A1-6 (Attachment H1) for a point within the middle of the Upper GLU layer. The results indicate that an average excess pore pressure of 50 kPa may have been present within the Upper GLU layer at the time of failure.

2.5.2 Two-Dimensional Consolidation Analysis

Two-dimensional consolidation analyses were completed using the program Plaxis2D. Plaxis2D is a finite element program capable of capturing complex stress strain soil behavior in parallel with complex pore pressures effects.

The soil profile, drainage boundary conditions, consolidation parameters and reference points utilized in the Plaxis2D analyses were consistent with the Settle3D analyses. The soft clay model within Plaxis2D was used to complete the consolidation analyses. Material strengths utilized for the soft clay model were selected to prevent generation of shear induced pore pressure by setting them high enough that embankment failure would not occur in the model. The model was reviewed during each loading increment to confirm shear induced pore pressure and/or zones of significant shear strain were not occurring. The complex loading sequence shown in Figure H.A1-5 (Attachment H1) was utilized for the model along with the loading curve in Figure H.A1-1 (Attachment H1).

Both isotropic and anisotropic cases were examined to determine the impact of higher rates of pore pressure dissipation in the horizontal direction in the Upper GLU.

The results of these analyses (isotropic and anisotropic) are presented in Figure H.A1-7 (Attachment H1) for a reference point within the middle of the Upper GLU layer. The results are similar to the Settle3D analyses but the pore pressure dissipation is different during some stages, particularly under the embankment slope and toe, due to the more accurate representation of the embankment load on the foundation. The stresses from later stages placed at or near the crest of the embankment are more widely distributed when they reach the Upper GLU layer which results in higher pore pressures toward the toe of the embankment. The results indicate that consideration of higher rates of pore pressure dissipation in the lateral direction has a relatively small impact.

Appendix H: Drawings

- **Drawing H1**: Basis for Geometry Used for Stability Analysis
- **Drawing H2**: Toe Geometry Comparison Water Table Variation
- **Drawing H3**: Water Table Variation

Appendix H: Attachments

- **Attachment H1**: Consolidation Analysis