NYRSTAR MYRA FALLS
OLD TAILINGS DISPOSAL FACILITY
APA PASTE BERM STABILITY REPORT

Submitted to:

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P.O. Box 8000
Campbell River, BC

Submitted by:

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a Division of Amec Foster Wheeler Americas Limited
Burnaby, BC

26 August 2016

Amec Foster Wheeler File: NX14001D.7.500

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IMPORTANT NOTICE

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1.0 INTRODUCTION

In the 2013 and 2014 dam safety inspection (DSI) reports (AMEC, 2014 and 2015a), AMEC Environment & Infrastructure (now Amec Foster Wheeler Environment & Infrastructure) recommended the following with respect to the Amalgamated Paste Area Berm (APA Berm):

> *2014-01: Investigate Paste Berm stability and seepage conditions. Design remedial drainage and/or buttressing during 2015. The design must incorporate measures to address groundwater discharge at the east abutment. See also recommendation 2013-05.*
>
> *2013-05: Carry out investigation regarding groundwater and seepage conditions at the east abutment of the Paste Berm.*

Nyrstar Myra Falls Ltd. (NMF) informed Amec Foster Wheeler that the BC Ministry of Energy and Mines (MEM) has required that NMF implement all DSI recommendations. The seepage at the east abutment was addressed as part of the Lower Lynx Diversion Ditch project (AMEC, 2015a) which was under construction at the time of this report. This report presents the design of the amalgamated paste area (APA) berm stabilization.

The location of Nystar Myra Falls is shown in Figure 1-1. The general arrangement of the tailings facilities is shown in Figure 1-2. The layout of the Old Tailings Disposal Facility (Old TDF) including the APA Berm and adjacent facilities are shown in Figure 1-3.

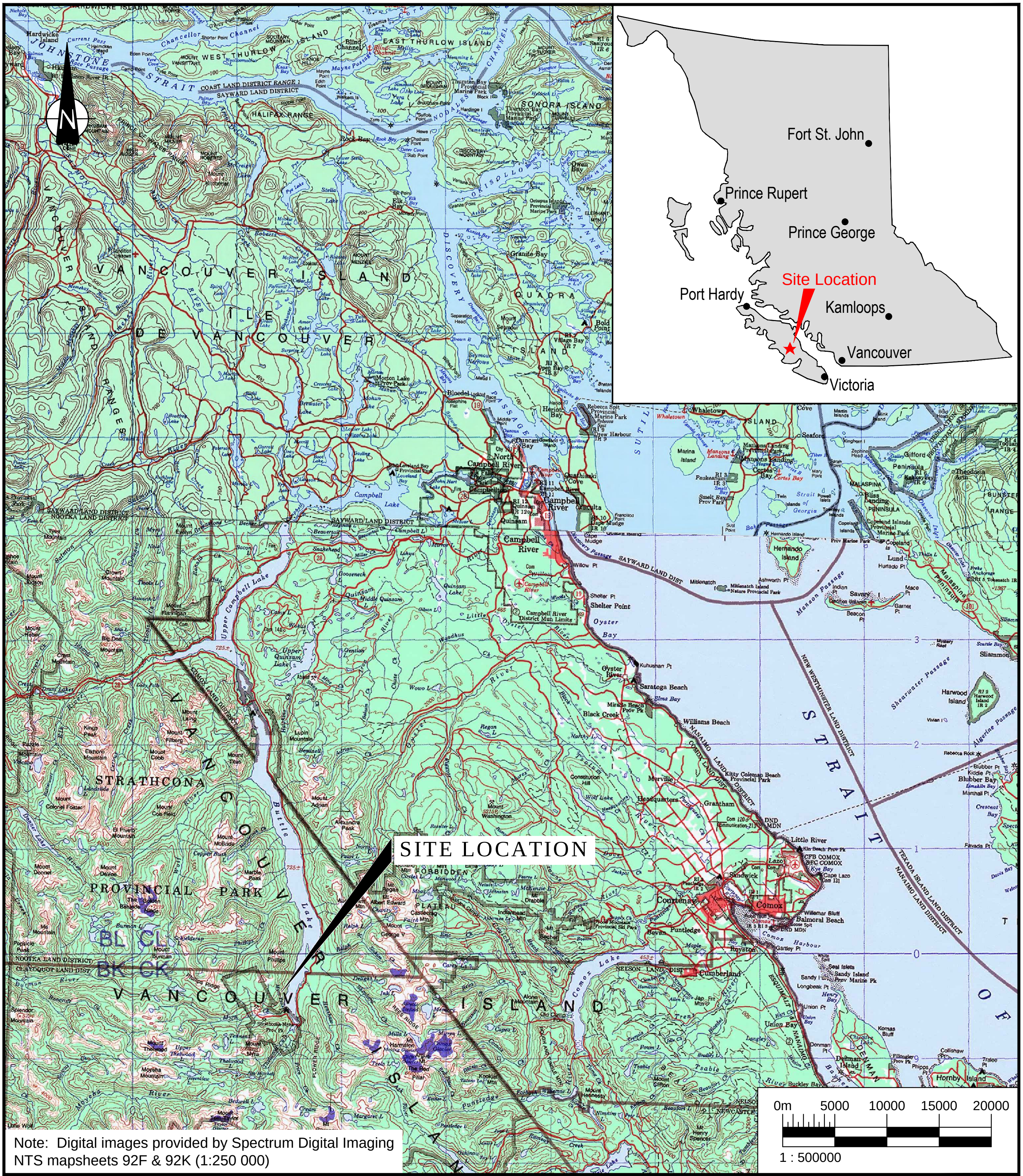
Preliminary slope stability analyses conducted by Amec Foster Wheeler in 2011 to assess the current stability of the APA berm indicated that the *post-seismic* factors of safety against slope stability failure were less than unity (AMEC, 2011; NMF, 2012). The analyses also indicated that the factors of safety could be improved by flattening the downstream slope of the APA berm: however, the analyses indicated that flattening the downstream slope of the APA berm could degrade the overall factors of safety of the Old TDF; similarly for the addition of closure cover.

Analyses completed in the assessment of APA berm stabilization and impact on the overall stability of the Old TDF consisted of the following.

- Two-dimensional, limit equilibrium slope stability analyses (Section 5.0)
- One-dimensional site response analyses (Section 6.2)
- Two-dimensional site response analyses (Section 6.3)

Based on the analyses, post-seismic instability of the APA berm can be mitigated by simply raising the elevation of the existing toe berm instead of flattening the slope as per the preliminary analyses conducted in 2011. The target post seismic factor of safety can be achieved with little or no impact on the overall post-seismic stability of the Old TDF. As well, the two-dimensional site response analyses indicated that the estimated lateral and vertical displacements are significantly less than those estimated during the design of the seismic upgrade berm (Klohn-Crippen, 1999).

Preliminary analyses also were conducted to assess the possible impact of closure cover on the overall stability of the Old TDF. Two closure cover scenarios were assessed – the conceptual closure cover (RGC, 2015) and a 2.5-meter thick conformal cover. The preliminary analyses indicate the addition of closure cover does impact the stability of the Old TDF especially if the closure cover extends to the crest of the seismic berm. The static and post-seismic factors of safety for all modes of failure remain above the targets for closure covers that do not extend over the 6:1 slope and seismic berm. If closure cover is required to extend to the crest of the seismic berm, there are methods to reduce the impact on stability of the Old TDF. Further discussion is provided in Section 7.0.



amec foster wheeler Environment & Infrastructure	CLIENT: nyrstar	DWN BY: YC	PROJECT: MYRA FALLS TAILINGS STORAGE FACILITY AMALGAMATED BERM STABILIZATION	DATE: AUGUST 2016
		CHK'D BY: D. Hughes-Games		PROJECT NO: NX14001D.7.500
	Amec Foster Wheeler Environment & Infrastructure 4385 Boban Drive, Nanaimo, B.C., V9T 5V9 Tel. 250 758-1887 Fax 250 758-1899	DATUM: NAD83	TITLE: SITE LOCATION PLAN	REV. NO: A
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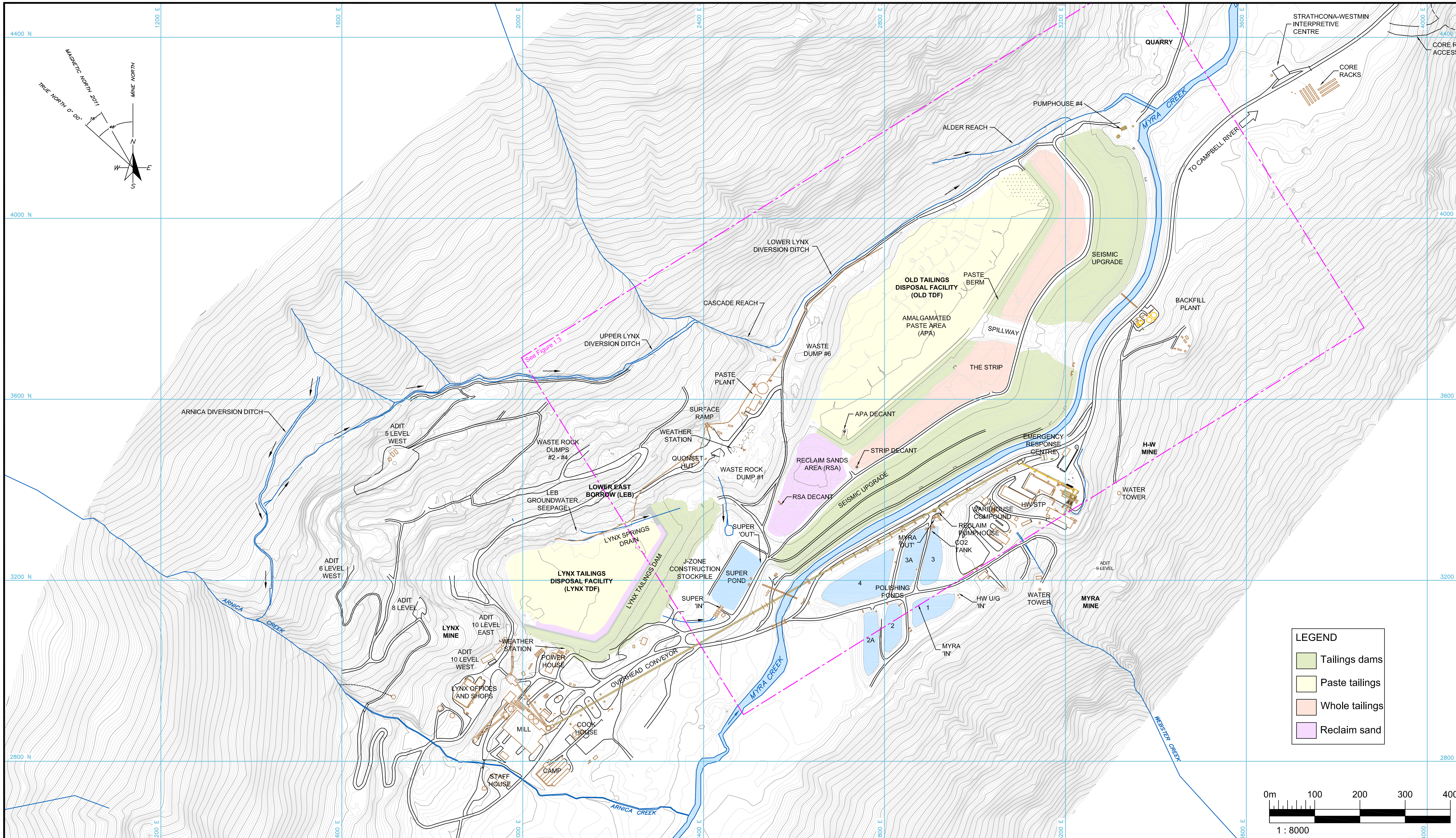

NOTES:
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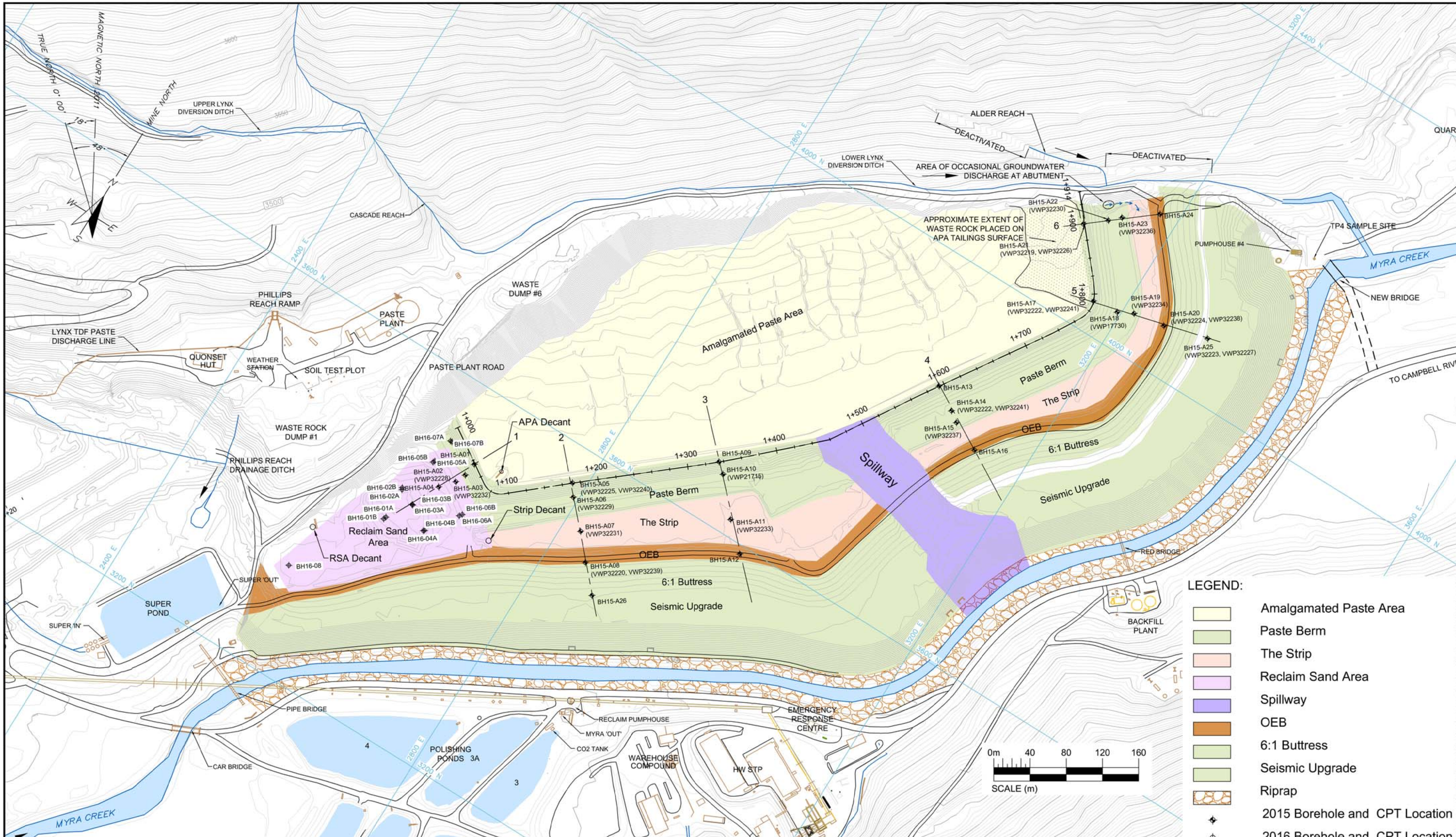
REVIEWED BY:	DAS
SCALE:	1:8000

PROJECT:
MYRA FALLS OLD TAILINGS DISPOSAL FACILITY
AMALGAMATED PASTE AREA BERM STABILIZATION

TITLE:
GENERAL SITE ARRANGEMENT PLAN

PROJECT NO:	NX14001D.7.500
REVISION NO:	
DATE:	AUGUST 2016
FIGURE NO:	1.2
SHEET NO:	1 of 1

S:\PROJECTS\Other Offices\Nanaimo (NX)\NX14001 - ALL Nyrstar Myra Falls\Drawings\2016-03 OLD Stability\140010.7.500\Figure 1.2.dwg - FIGURE 2 - Aug. 26, 2016 12:11pm - yoan.chen



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SCALE: 1:4000

PROJECT: MYRA FALLS TAILINGS DISPOSAL FACILITY AMALGAMATED PASTE AREA BERM STABILIZATION

TITLE: OLD TDF APA BOREHOLE LOCATION PLAN

PROJECT NO.: NX14001D.7.500
REVISION NO.:
DATE: AUGUST 2016
FIGURE NO.: 1.3
SHEET NO.: 1 of 1

2.0 APA BERM DESIGN AND CONSTRUCTION HISTORY

2.1 APA Berm Design

The APA berm was designed in 2001 as part of the Paste Fill Engineering Project (Klohn-Crippen, 2001). The purpose of the APA berm was to increase the capacity of the Old TDF and provide storage for paste tailings until the Lynx Pit became available for use as a tailings disposal facility. At the time the APA berm was designed, the Old TDF was divided into two areas referred to as Area I and Area II with a separator berm or partition embankment between the two areas. Initially, the Paste Fill Engineering Project included storage of paste tailings only in Area II. The APA berm as designed had a crest elevation of 3388.4 m, a crest width of six meters, 2(H):1(V) upstream slope and a 4(H):1(V) downstream slope. The APA berm also included a 27 m wide. 2.5 m high toe berm for stabilization. The downstream slope of the toe berm was 3(H):1(V).

At the time of the design of the APA berm, the seismic upgrade berm had not yet been constructed; however, the possible impact of the APA berm on the seismic stability of the Old TDF post construction of the seismic upgrade berm was assessed during the APA berm design phase. The static stability was not assessed. Post-seismic factors of safety were equal to or greater than the target of 1.25. This target was set during design of the seismic upgrade berm (Klohn-Crippen, 1999a) to '*ensure stability and to limit displacements of the proposed berm to less than 3 m*'.

Klohn-Crippen estimated dynamic displacements of the APA berm using approximate methods (Newmark, 1965). Numerical modelling was not completed. Klohn-Crippen concluded that the presence of the APA berm should not increase the dynamic displacement of the seismic upgrade berm from those estimated in the design report (Klohn-Crippen, 1999a), that dynamic displacements of the APA berm crest will be on the order of 1.0 m to 1.2 m and that some lateral spreading of the paste tailings could occur after a seismic event.

A design report for the Area I APA berm was not available to Amec Foster Wheeler; however, it appears that the Area II APA berm design configuration was generally adopted for the Area I APA berm configuration. The design of the toe berm was revised to accommodate gravity drainage in the 'Strip', the area between the APA berm and OEB.

2.2 APA Berm Construction History

Construction of the Area II APA berm starter embankment was originally scheduled to begin after conventional tailings deposition in Area II had reached the maximum allowable elevation of 3381.4 m; however, because of a mine shutdown, the tailings had not yet reached the maximum allowable elevation at the time construction began in 2002 (Klohn-Crippen, 2003a).

In the Fall of 2003, the starter embankment was constructed to an elevation of 3382.9 m with a crest width of 60 m along the western embankment and 40 m along the south and west embankments. The starter embankment was constructed of waste rock placed over at least one layer of non-woven geotextile. In softer areas, up to four layers of overlapping geotextile were required. Initial lift thickness was approximately 1.2 m thick. Significant mud wave formation and subsidence were observed during placement of the initial lift. When assessing a significant mud wave, test pit was excavated at the leading edge of the fill during placement of the initial lift. A fill thickness of 2.5 m to 3.0 m was observed (Klohn-Crippen, 2003a).

An ‘inner perimeter’ berm 10 m-wide, one meter-high berm was constructed along the upstream edge of the starter embankment to accommodate continued deposition of the conventional tailings in Area II to the maximum allowable elevation. The target elevation of the inner perimeter berm was 3383.9 m. At the end of the 2002 construction season, up to 0.5 m of additional fill had to be placed at the crest of inner perimeter berm to achieve the target elevation.

Intermediate berms were constructed between the starter embankment and the outer edge berm (OEB), creating additional tailings deposition areas designated Areas A, B, and C. Area II inside the starter embankment was designated as Area D. At the time the construction of the starter embankment began, the tailings were at approximate elevation 3380.7. Construction of the intermediate berms allowed for deposition of an additional one meter of tailings, approximately, between the APA berm starter embankment and the OEB.

The Area II APA berm starter embankment was raised to elevation of 3386.9 m in 2003 (Klohn-Crippen, 2004). A 10 m to 13 m wide toe berm extension also was added in 2003. The Area II APA berm was raised to the final design elevation of 3388.4 m in 2004 (Klohn-Crippen, 2005a) and to elevation 3389.0 m in 2005 (Klohn-Crippen 2006).

Construction of the Area I starter berm began in 2004 (Klohn-Crippen, 2005a). The target elevation was 3384.6 m; however, fill placement was halted in some areas prior to reaching the target elevation because of excessive settlement and high pore pressures. Similar difficulties with mud waves and subsidence were observed. Mud waves up to 1.5 m high and extending 30 m beyond the leading edge of the fill were observed. Two layers of geotextile were placed prior to placement of the initial lift. In some areas, generally towards the center of the tailings deposit, four to six layers of geotextile were required.

The 2005 construction record for the Area I APA berm appears to be incomplete based on the historical reports made available to Amec Foster Wheeler; however, by the end of the 2005 construction season, the crest of the Area I APA berm had been raised to elevation 3388 m (Klohn-Crippen, 2006). The elevation of the crest and the toe berm were still below the design elevations of 3388.4 m and 3386.5 m, respectively.

Construction reportedly completed in 2006 included a raise of the design crest elevation in both areas to the current elevation of 3391.8 m by placing waste rock (AMEC, 2007). The raise reduced the design crest width from 6 m to about 3 m. The side slopes of the raise were on the order of 1.5(H):1(V).

The design drawings and yearly construction record drawings are provided in Appendix A. The materials used to construct the APA berm generally consisted of waste rock and tailings sand as shown on the drawings.

2.3 APA Berm Construction Instrumentation

Eight vibrating wire piezometers were installed during and shortly after construction of the Area II starter embankment and monitored through construction to the final lines and grades. As expected, pore pressures increased during fill placement and dissipated after fill placement was halted for the season. In general, pore pressures stabilized by the start of each construction season and reportedly showed no significant rise during the 2005 (final) construction season (Klohn-Crippen, 2006).

The maximum calculated pore pressure coefficient (R_u) value was 0.35 which was slightly above the Level I threshold of 0.34 and occurred during the 2003 construction season (Klohn-Crippen, 2004). R_u values were not calculated during the 2002 construction season when the Area II starter embankment was constructed.

Piezometers were not installed specifically to monitor pore pressures during construction of the Area I APA berm starter embankment in 2004; however, there was one existing piezometer within the footprint of the starter embankment. Calculated R_u values exceeded the Level II threshold of 0.5 and reached a peak of 0.57 (Klohn-Crippen, 2005a). Additional piezometers were installed in early 2005. Maximum calculated R_u value was 0.25 (Klohn-Crippen, 2006).

Surface displacement monuments and/or settlement pins also were installed at various stages of construction. They were installed at the end of the each construction season and did not capture settlement occurring during seasonal construction. As well, they were installed on the toe berm and were not in a position to capture the settlement of the APA berm. Most did not survive through to achieving the final design crest and toe berm elevations. As such, the total settlement along the crest of the APA berm is not known.

3.0 CURRENT DESIGN CRITERIA AND CONSIDERATIONS

3.1 Old TDF Classification and Phase

The Old TDF is classified as a high consequence dam in accordance with the Canadian Dam Association's 2014 Technical Bulletin: Application of Dam Safety Guidelines to Mining Dams.

The Technical Bulletin also describes the phases of a mining dam. Currently, the Old TDF is in the Closure – Transition Phase. Once closure cover is in place, the Old TDF will enter the Closure – Active Care Phase.

3.2 Seismic Criteria

The Old TDF is expected to remain in Closure – Active Care Phase for an extended period of time because of the geochemical aspects of the facility. It is also expected that surveillance could become infrequent with time and that the owner or future owners might not have sufficient resources to respond to warning signs or emergencies. The CDA bulletin suggests that under these circumstances the seismic design criteria for Closure – Passive Care should be used in design of the APA berm stabilization; however, the decision to consider Closure – Passive Care seismic criteria for the Old TDF rather than the criteria associated with Closure – Active Care must be made by NMF with concurrence from MEM.

As directed by NMF, Amec Foster Wheeler proceeded with the seismic criteria for Closure – Passive Care. The target levels for earthquake hazard for Closure – Passive Care excerpted from the Technical Bulletin are provided in Table 3-1.

Table 3-1: Target Levels for Earthquake Hazards, Standards-based Assessments for Closure – Passive Care Case

Dam Classification	Annual Exceedance Probability – Earthquakes
Low	1/1,000
Significant	1/2,475
High	**1/2 between 1/2,475 and 1/10,000 AEP or MCE**
Very High	1/10,000 AEP or MCE
Extreme	1/10,000 AEP or MCE

A site-specific probabilistic seismic hazard assessment (PSHA) was completed for the Myra Falls mine in 2016 (AMEC, 2016a). The dominant source contributing hazard to the Old TDF is a M9 (magnitude nine) Cascadia Subduction Zone interface earthquake. The maximum credible earthquake (MCE) was based on the 84th percentile deterministic spectra for this interface event. The target elastic response spectrum, halfway between the 1:2475 AEP spectrum (uniform hazard response spectrum [UHRS]) and that of the MCE for soft rock (V_{s30}=450 m/s), is provided in Table 3-2 and shown graphically in Figure 3-1. The reader is referred to the site-specific hazard evaluation for a detailed discussion of the seismicity and seismic hazards of the Nyrstar Myra Falls area.

Table 3-2: Target Elastic Response Spectrum

Period (seconds)	0.01	0.03	0.05	0.1	0.15	0.2	0.3	0.5	1	2	5	10	PGA
Spectral Acceleration (g)	0.653	0.657	0.600	1.028	1.270	1.356	1.400	1.275	0.771	0.323	0.092	0.028	0.653

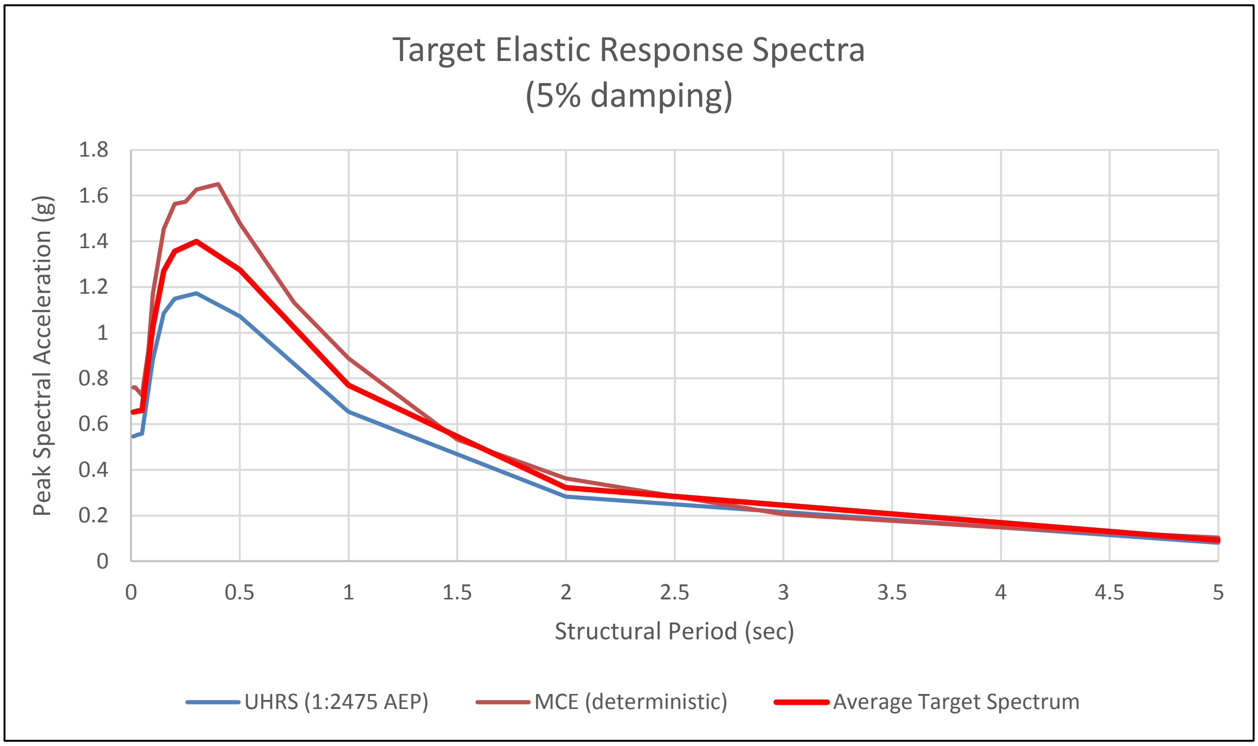


Figure 3-1: Target Elastic Response Spectra

3.3 Input Earthquake Ground Motion Time Histories

The seismic hazard to the site for the 1:2475 AEP to MCE earthquake hazard is dominated by contributions from offshore, large magnitude subduction events with horizontal Epicentral distances of 40 to 70 km from the site based on the PSHA. As well, the longer duration shaking associated with large magnitude subduction earthquakes is expected to cause the largest seismically induced deformations of the Old TDF. Accordingly, a suite of six earthquake records representative of large magnitude subduction earthquakes was selected for consideration in seismic analysis of the Old TDF.

The six subduction earthquake records were selected based on distance from the source location and the recording station to provide a reasonable match to the results from the PSHA. Records were also selected such that spectra computed from the unfiltered accelerograms only required linear scaling in the range of 0.5 to 2 times to approximately match the target spectrum. Each record represents the maximum horizontal component of shaking recorded during the particular earthquake.

The subduction zone earthquake records were baseline corrected, filtered and linearly scaled using the proprietary software SeismoSignal (v5.1.0) by SeismoSoft srl. Records 1 through 4, which were obtained during the 2011 Tohoku Earthquake in Japan, had the highest Arias Intensity and longest significant durations of shaking after baseline correction, filtering and scaling. These records, identified in Table 3-3 as earthquake records 1 through 4, were selected for use in seismic analysis of the Old TDF. It was anticipated they would produce the highest seismic deformations of the Old TDF. Earthquake record 5 was recorded on a 5 m thick surface layer of clay overlying sand. This record was not considered representative of dense Site Class C soils and subsequently not used in the analyses.

Spectral matching to the target spectrum shown in Figure 3-1 was performed using RSPMATCH09 (Al Atik and Abrahamson, 2010). The characteristics of each record, after baseline correction, filtering, linear scaling and spectral matching, are summarized in Table 3-3. Comparison spectral acceleration plots of the four filtered records versus the target UHRS as well as acceleration, velocity and displacement time histories are shown in Appendix B.

Table 3-3: Summary of Earthquake Accelerogram Characteristics for Hard Rock Conditions

Earthquake Record	Earthquake	Recording Station	Moment Magnitude M	Horizontal Epicentral Distance - R (km)	Directional Component	Peak Ground Acceleration after Filtering and Scaling - PGA (g)	Arias Intensity after Filtering and Scaling - AI (m/s)	Significant Duration after Filtering and Scaling - T (s)	Peak Ground Acceleration after Spectral Matching - PGA (g)	Arias Intensity after Spectral Matching - AI (m/s)	Significant Duration after Spectral Matching - T (s)
1	2011 Tohoku, Japan	MYGH09	9	≈ 60	EW2	0.41	14.07	104.87	0.60	22.9	105.9
2	2011 Tohoku, Japan	IWT011	9	≈ 60	EW	0.45	10.90	90.96	0.62	17.7	91.5
3	2011 Tohoku, Japan	MYG009	9	≈ 60	EW	0.48	8.00	102.58	0.61	12.7	86.6
4	2011 Tohoku, Japan	MYG009	9	≈ 60	NS	0.67	6.80	104.52	0.63	16.9	84.7
5	2011 Tohuku, Japan	MYG015	9	≈ 50	EW	0.17	3.96	113.76	n/a	n/a	n/a
6	2010 Maule, Chile	PUENTE ALTO	8.8	≈ 70	NS	0.45	7.43	36.23	0.60	14.3	37.0

3.4 Geotechnical Criteria

For both Closure – Active Care and Passive Care, the geotechnical criteria for slope stability assessments provided for Construction, Operation and Transition Phases are applicable. The target levels for static and post-earthquake slope stability assessments excerpted from the Technical Bulletin are provided in Table 3-4 and Table 3-5. The full or partial rapid drawdown loading condition is not applicable to the static assessment of the Old TDF because the condition cannot occur.

Table 3-4: Target Factors of Safety for Slope Stability in Construction, Operation and Transition Phases - Static Assessment

Loading Condition	Minimum Factor of Safety	Slope
During or at end of construction	>1.3 depending on risk assessment during construction	Typically downstream
Long term (steady state seepage, normal reservoir level)	1.5	Downstream
Full or partial rapid drawdown	1.2 to 1.3	Upstream slope where applicable

Amec Foster Wheeler proposes that end of construction be defined as the time the Old TDF enters the Closure – Active Care Phase and instrumentation indicates that all construction-induced pore pressures have dissipated including those generated by placement of closure cover and that steady state pore pressures have been achieved. The static undrained shear strength of the tailings was used in slope stability analyses prior to the defined end of construction. The target factor of safety of 1.3 applies to the end of construction case after which the frictional strength will be used and the target of 1.5 applies.

Table 3-5: Target Factors of Safety for Slope Stability in Construction, Operations, and Transition Phases - Seismic Assessment

Loading Condition	Minimum Factor of Safety
Pseudo-static	1.0
Post-earthquake	1.2

Post-seismic target factors of safety for previous analyses were different than those shown in Table 3-5. Previously, a post-seismic target factor of safety of 1.25 was used for failure surfaces that might impact the outer toe drain. For all other failure surfaces, the post-seismic target factor of safety was 1.1. These targets are not consistent with the Technical Bulletin which was published by CDA in 2014. Amec Foster Wheeler proceeded with design of the APA berm stabilization using the target factors of safety as recommended in the Technical Bulletin.

3.5 Other Design Considerations

The design of the APA berm should provide a uniform crest elevation and width and, reduce existing over-steepened slopes. The design criteria for the APA berm are as follows:

- Crest elevation – 3392 m maximum
- Crest width – 6 m minimum
- Slopes – 2(H):1(V) maximum

The toe berm will be raised and/or extended as required to achieve the target static and post-seismic factors of safety as provided in Table 3-4 and Table 3-5.

The design of the APA berm stabilization must accommodate the existing decants and spillways as currently designed and/or constructed. The APA berm stabilization design must also accommodate or be a part of surge pond currently under construction at the west end of the Old TDF. The design of final closure cover must accommodate the APA berm stabilization and also not adversely impact the overall stability of the Old TDF. Design of closure cover was not yet in process at the time of this report.

Amec Foster Wheeler understands that the outer toe drain is considered essential to the long-term performance of the Old TDF. Based on the information provided in 1999 seismic berm design report (Klohn-Crippen, 1999), it appears that survival of the outer toe drain intact and fully functional was not a design criteria when considering the maximum design earthquake ground motions. Remedial measures could be required to maintain the integrity of the outer toe drain after the design earthquake.

4.0 GEOLOGICAL MODELS

Geological models were developed for six cross-sections, subsequently referred to as 'planes', for use in slope stability analyses. The geological models were developed using stratigraphy based on borehole logs from historical (Knight Piesold,1982,1996; Klohn-Crippen, 1998, 1999, 2003b) and more recent subsurface investigations (Amec Foster Wheeler, 2015b, 2016a) and, on drawings that show the configuration of the Old TDF at 100-meter interval stationing along the toe of the Old TDF (AMEC, 2011). The drawings were based on construction as-built drawings as well as other information available at the time the final as-built report and drawings were prepared.

Discussions of the foundation soils stratigraphy, geotechnical material properties and the pore pressure regime are provided in Sections 4.2 through 0. The Planes 1 through 6 are located at approximately at Stations 1+040, 1+170, 1+340, 1+600, 1+1800 and 1+900, respectively, as shown in Figure 1-3. The geological models for the six planes are shown in Figure 4-1 through Figure 4-6.

4.1 APA Berm Configuration

The surface elevation of the APA berm was based on the design considerations for final crest elevation, width and slope as well as the as-built drawings. The bottom elevation was determined based on data from boreholes advanced through the APA berm (Amec Foster Wheeler, 2015).

The information and descriptions of observations contained in the construction reports suggest that significant settlement and/or intrusion into the underlying tailings occurred during construction of the APA Berm. As well, up to 1 m of tailings were deposited between the APA berm and the OEB suggesting that the bottom of the APA berm should not be modelled as a horizontal surface at the elevation of the tailings between the APA berm and the OEB, especially in Area II where additional tailings were added between the APA berm and the OEB to improve drainage towards the western decant (Klohn-Crippen, 2005).

4.2 Foundation Soils Stratigraphy

The same unit designations for the foundation soils that were used in previous descriptions of the stratigraphy have been adopted for consistency. Brief descriptions of the foundation soils from original ground surface down are as follows:

- Unit 2b: Colluvial silty sand and gravel.
- Unit 2a: Glacial fluvial soils consisting of sand and gravel with cobbles and boulders.
- Unit 3: Silty sand considered to be a 'transition' zone between the glaciolacustrine soils and the overlying glaciofluvial soil.
- Unit 4: Highly laminated fine sand and nonplastic to low plasticity silt and clay of glaciolacustrine origin.
- Bedrock and/or glacial till.

4.3 Geotechnical Material Parameters

The geotechnical material properties used in the current slope stability analyses are summarized in Table 4-1. Further discussions of the geotechnical material properties of the foundation materials and tailings are provided in Appendices C and D.

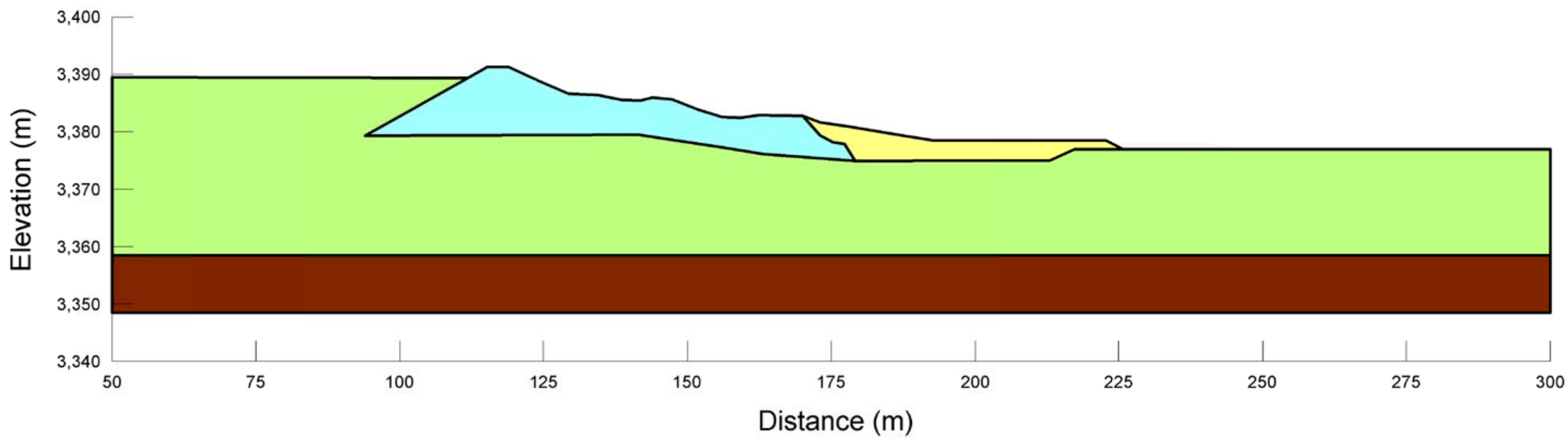
Table 4-1: Geotechnical Material Properties

Material type	Unit Weight γ (kN/m^{3})	Static Strength Φ, S_u	Post-seismic Residual Strength Φ, S_r
Seismic Berm	22	38°	38°
Embankment Raises	22	34°	34°
Tailings	22.5	28°, S_u=0.20 σ'_{v0}	S_u=0.14σ'_{v0}
Starter Embankment	22	36°	36°
Dense Glaciofluvial Sand and Gravel with loose layers (Unit 2a)	24	34°	34°
Medium Dense Sand and Gravel (densified)	24	36°	36°
Colluvial Silty Sand and Gravel (Unit 2b)	24	34°	S_u=0.28σ'_{v0}
Transitional Silty Sand (Unit 3)	22	32°	S_u=0.35σ'_{v0}
Glaciolacustrine Silt and Clay (Unit 4)	20	S_u=0.22σ'_{v0}	S_u=0.13σ'_{v0}

4.4 Pore Pressure Regime

The pore pressure regime within and beneath the Old TDF is strongly and favourably influenced by the presence of the coarse granular soils present at the original ground surface and the system of drains installed within the coarse granular soils at the time of initial construction and also during the seismic upgrade, as well as the overall balance between net annual precipitation and drainage. The pore pressure at the original ground surface was assumed to be essentially zero. This assumption is supported by the results of pore pressure dissipation (PPD) tests conducted at the base of the tailings deposit during the 2015 APA berm investigation (Amec Foster Wheeler, 2015b) and in the foundation soils during the 2015 glaciolacustrine field investigation (Amec Foster Wheeler, 2016b) as well as data from the numerous hydrogeological wells that have been installed at the site (Robertson, 2014). Data from the hydrogeological wells also indicates that the pore pressure in the foundation soils is essentially hydrostatic, possibly with a very slight upward gradient.

The PPD tests and piezometers within the tailings deposit indicate that the pore pressures were zero near the surface, increased to a maximum around mid-deposit and then decreased to zero at the base of the tailings (original ground surface). The maximum, mid-deposit pore pressures ranged between 1.0 m and 9.2 m with an average of around 3.5 m. Pore pressures within the tailings should be expected to vary with the seasons. Rapid increases have been observed at the start of the wet season, generally late October or early.



MATERIALS LEGEND

SEISMIC BERM FILL

EMBANKMENT RAISES

TAILINGS

UNIT 2b COLLUVIUM

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	Amec Foster Wheeler Environment & Infrastructure Suite 600 - 4445 Lougheed Highway, Burnaby, BC V5C 0E4 Tel. 604-294-3811 Fax 604-294-4664	DATUM: --	TITLE: APA Plane 1 - Geological Cross-section	REV. NO: A
		PROJECTION: --		
		SCALE: AS SHOWN		FIGURE NO: 4.1

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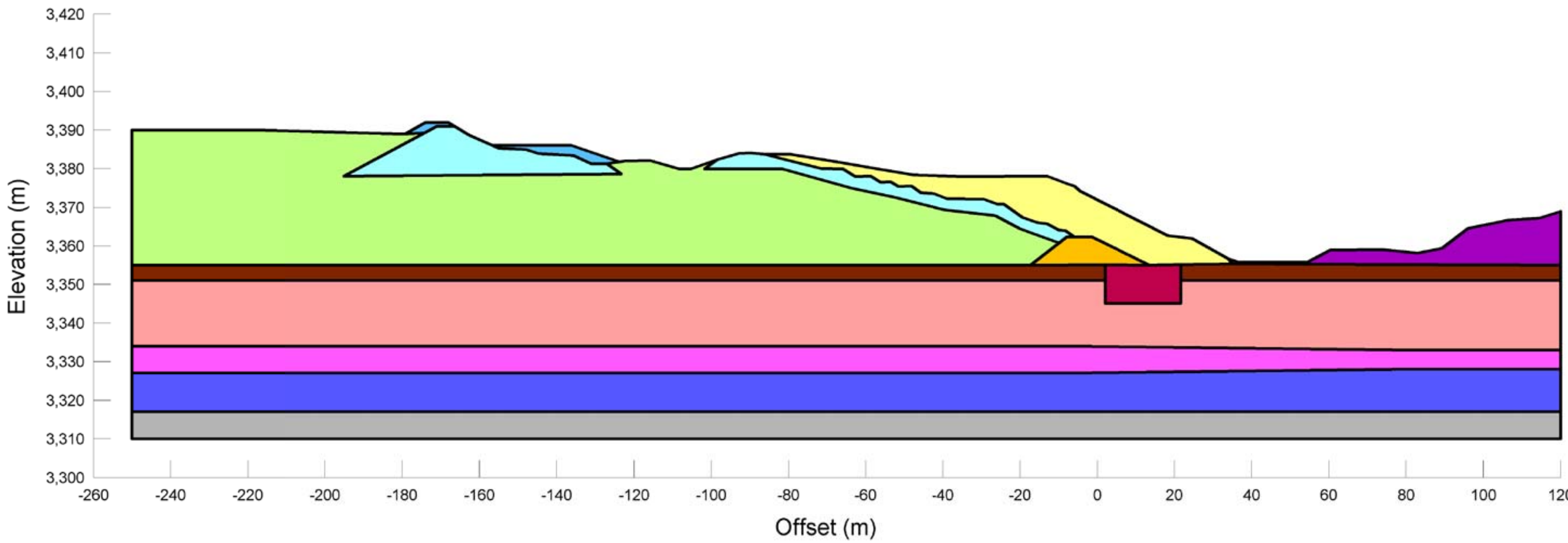

MATERIALS LEGEND

- SEISMIC BERM FILL
- EMBANKMENT RAISES
- STARTER EMBANKMENT
- MINE ROAD FILL
- UNIT 2a GLACIO FLUVIAL
- EMBANKMENT RAISES - ADDITION
- GLACIAL TILL
- TAILINGS
- UNIT 3 TRANSITIONAL GLACIOFLUVIAL
- UNIT 4 GLACIOLACUSTRINE
- UNIT 2b COLLUVIUM
- DENSIFIED ZONE

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	Amec Foster Wheeler Environment & Infrastructure Suite 600 - 4445 Lougheed Highway, Burnaby, BC V5C 0E4 Tel. 604-294-3811 Fax 604-294-4664	CHK'D BY: DAS	TITLE: APA Plane 2 - Geological Cross-section	PROJECT NO: NX14001D.7.500
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		PROJECTION: --		FIGURE NO: 4.2
		SCALE: AS SHOWN		

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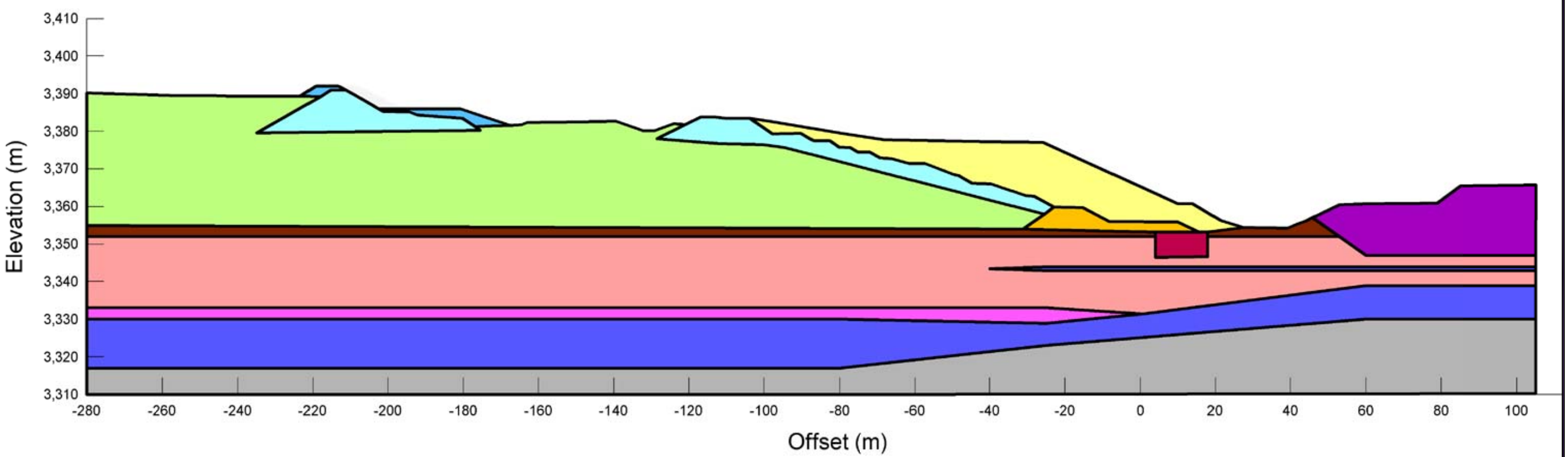

MATERIALS LEGEND

SEISMIC BERM FILL	GLACIAL TILL
EMBANKMENT RAISES	TAILINGS
STARTER EMBANKMENT	UNIT 3 TRANSITIONAL GLACIOFLUVIAL
MINE ROAD FILL	UNIT 4 GLACIOLACUSTRINE
UNIT 2a GLACIO FLUVIAL	UNIT 2b COLLUVIUM
EMBANKMENT RAISES - ADDITION	DENSIFIED ZONE

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		DATUM: --	TITLE: APA Plane 3 - Geological Cross-section	REV. NO: A
		PROJECTION: --		
		SCALE: AS SHOWN		FIGURE NO: 4.3

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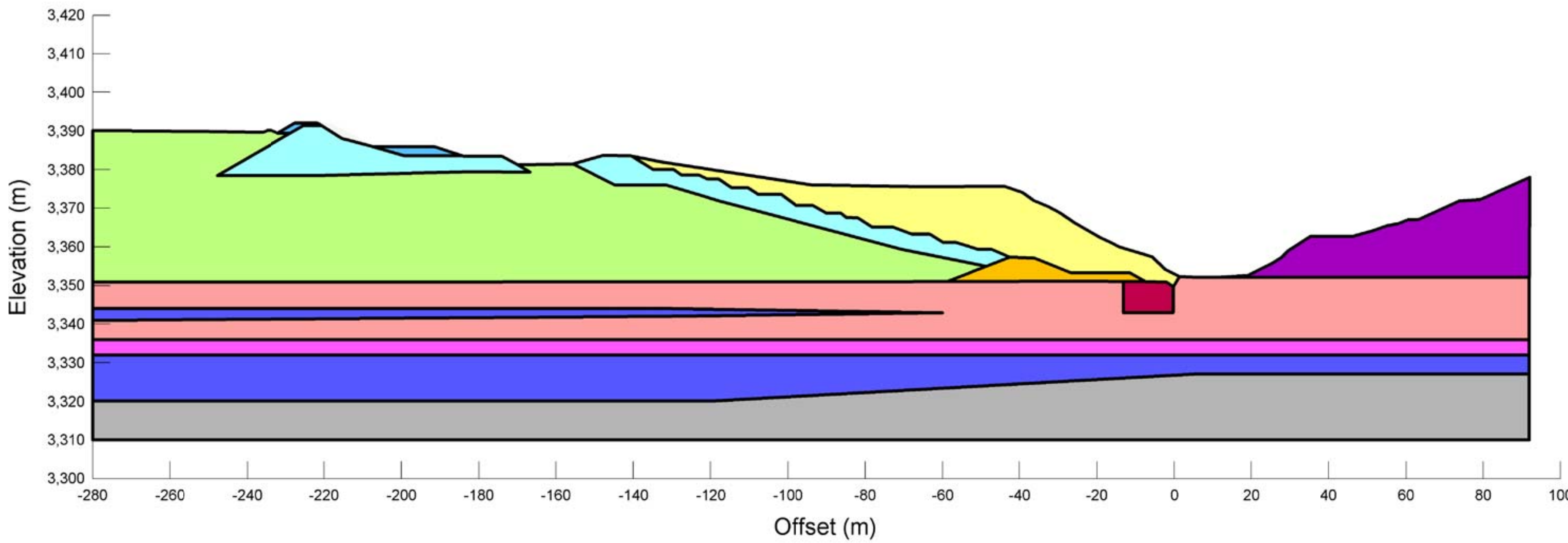

MATERIALS LEGEND

SEISMIC BERM FILL	GLACIAL TILL
EMBANKMENT RAISES	TAILINGS
STARTER EMBANKMENT	UNIT 3 TRANSITIONAL GLACIOFLUVIAL
MINE ROAD FILL	UNIT 4 GLACIOLACUSTRINE
UNIT 2a GLACIO FLUVIAL	DENSIFIED ZONE
EMBANKMENT RAISES - ADDITION	

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PROJECT:
MYRA FALLS TAILINGS DISPOSAL FACILITY
AMALGAMATED PASTE AREA BERM STABILIZATION

TITLE:
APA Plane 4 - Geological Cross-section

DATE: AUGUST 2016
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FIGURE NO: 4.4

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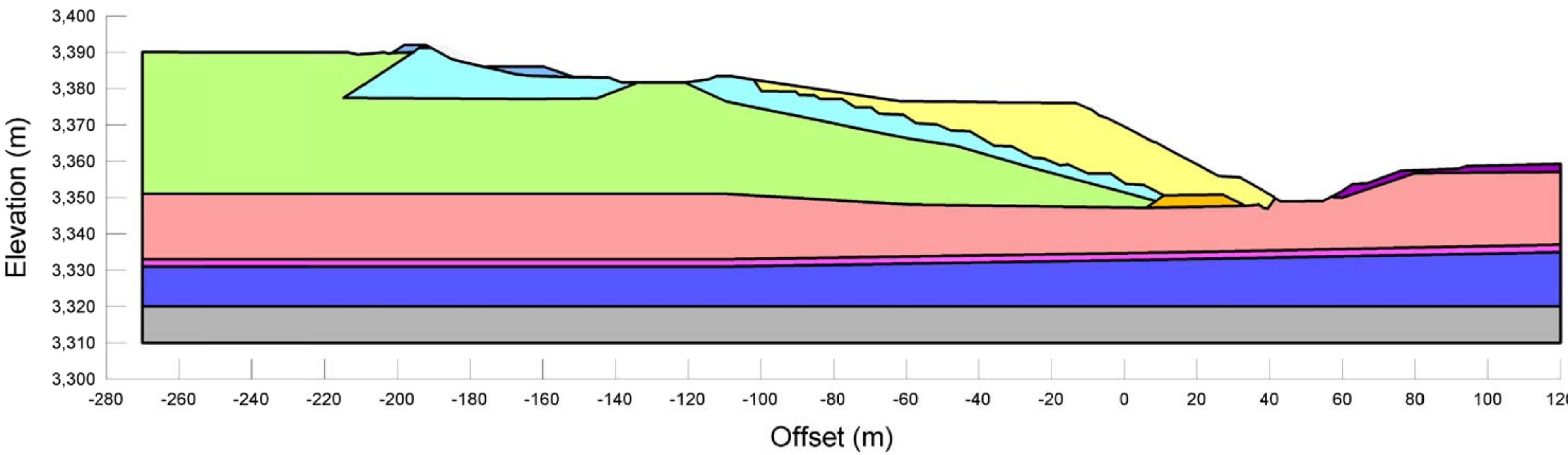

MATERIALS LEGEND

SEISMIC BERM FILL	GLACIAL TILL
EMBANKMENT RAISES	TAILINGS
STARTER EMBANKMENT	UNIT 3 TRANSITIONAL GLACIOFLUVIAL
MINE ROAD FILL	UNIT 4 GLACIOLACUSTRINE
UNIT 2a GLACIO FLUVIAL	EMBANKMENT RAISES - ADDITION

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MYRA FALLS TAILINGS DISPOSAL FACILITY
AMALGAMATED PASTE AREA BERM STABILIZATION

TITLE:
APA Plane 5 - Geological Cross-section

DATE: AUGUST 2016
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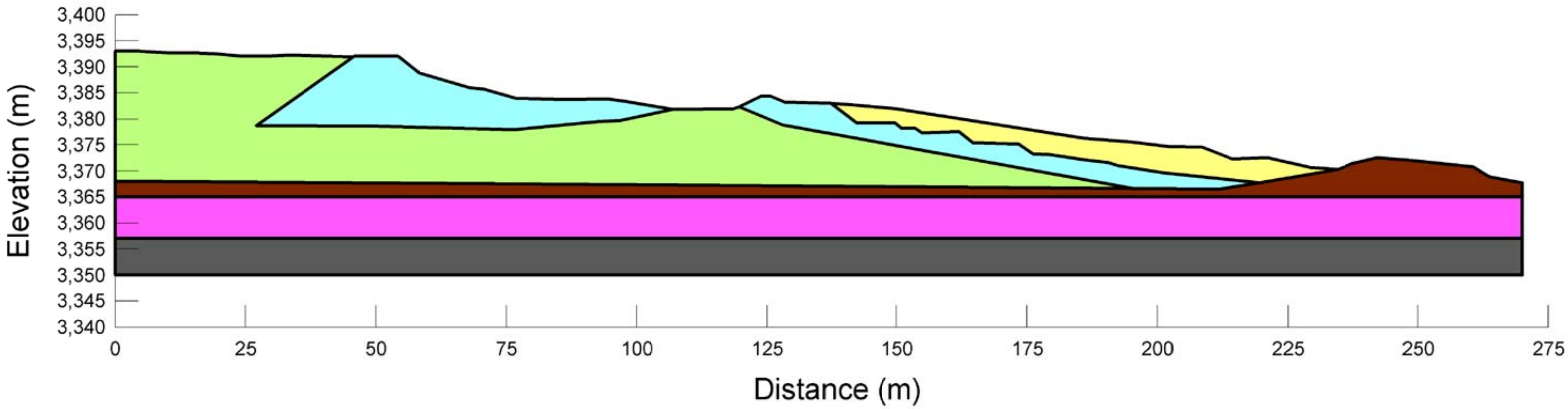
FIGURE NO: 4.5



MATERIALS LEGEND

SEISMIC BERM FILL

EMBANKMENT RAISES

UNIT 2b COLLUVIUM

GLACIAL TILL

TAILINGS

UNIT 3 TRANSITIONAL GLACIOFLUVIAL

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SCALE:	AS SHOWN

PROJECT:
MYRA FALLS TAILINGS DISPOSAL FACILITY
AMALGAMATED PASTE AREA BERM STABILIZATION

TITLE:
APA Plane 6 - Geological Cross-section

DATE:	AUGUST 2016
PROJECT NO:	NX14001D.7.500
REV. NO:	A
FIGURE NO:	4.6

5.0 LIMIT EQUILIBRIUM SLOPE STABILITY ANALYSIS

5.1 Methodology

Two dimensional limit equilibrium slope stability analyses were completed at six planes using the proprietary software Slope/W (2012) by GeoSlope International. The Morgenstern-Price half-sine function was used to resolve interslice forces.

Six failure modes generally consistent with previous analyses were evaluated at each of the six planes described in Section 4.0. The modes are briefly described as follows and shown graphically, using Plane 5 as an example, in
Figure 5-1:

- Mode A: Failure surfaces entirely within the seismic upgrade berm.
- Mode B: Failure surfaces beginning in the 6(H):1(V) slope of the seismic upgrade berm and exiting at or near the toe of the seismic upgrade berm.
- Mode C: Failure surfaces beginning at the near the crest of the seismic upgrade berm and exiting at or near the toe of the seismic upgrade berm.
- Mode D: Failure surface beginning near the Outer Edge Berm (OEB), extending into the foundation soils, and exiting at or near the toe of the seismic upgrade berm.
- Mode E: Failure surfaces beginning within the crest of the seismic upgrade berm, extending into the foundation soils, and exiting at or near the toe of the seismic upgrade berm.
- Mode F: Failure surfaces beginning behind or within the crest of the APA berm and exiting at or near the toe of the seismic berm.
- Mode G: Failure surfaces beginning behind or within the crest of the APA berm, extending into the foundation soils and exiting at or near the toe of the APA berm.

Modes A through F do not apply to Plane 1 because of the presence of the surge pond, shown in Figure 1-3. The APA berm in this area was stabilized by the addition of the surge pond east slope buttress (Amec Foster Wheeler, 2016d).

Static and post-seismic stability of the APA Berm initially were evaluated at a crest elevation of 3392 m and a crest width of six meters as per the design criteria discussed in Section 3.0. The design crest width was achieved at each plane by extending the APA berm over the existing tailings, essentially an upstream raise. Achieving the design crest elevation and width by extending the APA berm over the existing tailings reduced the required volume of material and eliminated the need for benching the downstream slope. Benching is normally recommended in order to avoid placement of 'sliver' fills and to provide a horizontal work surface for compaction equipment. However, benching requires excavation and replacement of existing berm material and, where only a relatively minor thickness of fill is required, benching could extend a meter or more into the existing APA berm depending on the width of compaction equipment and safety concerns for equipment working adjacent to slopes. Some settlement of the ground surface should be expected where the fill has been placed over the existing tailings.

The existing toe berm at each plane was then raised and/or widened as required to achieve the static and post-seismic target factors of safety for all modes of failure. The results of the analyses were then assessed and a toe berm elevation and width were selected such that when applied to all planes, the target factors of safety were achieved and/or exceeded. This process resulted in a uniform toe berm elevation of 3386 m and a minimum width of 15 m for the length of the APA berm.

Preliminary limit equilibrium slope stability analyses were also completed on Plane 5 to assess the possible impact that closure cover might have on the stability of the Old TDF. Three closure cover scenarios were assessed – the conceptual cover that provides a smooth transition between the crest of the APA berm and the crest of the OEB as per the conceptual cover configuration (RGC, 2015), a conformal cover with a thickness of 2.5 m that extends to the crest of the seismic berm and, a 2.5-m-thick conformal cover that only extends to the crest of the OEB.

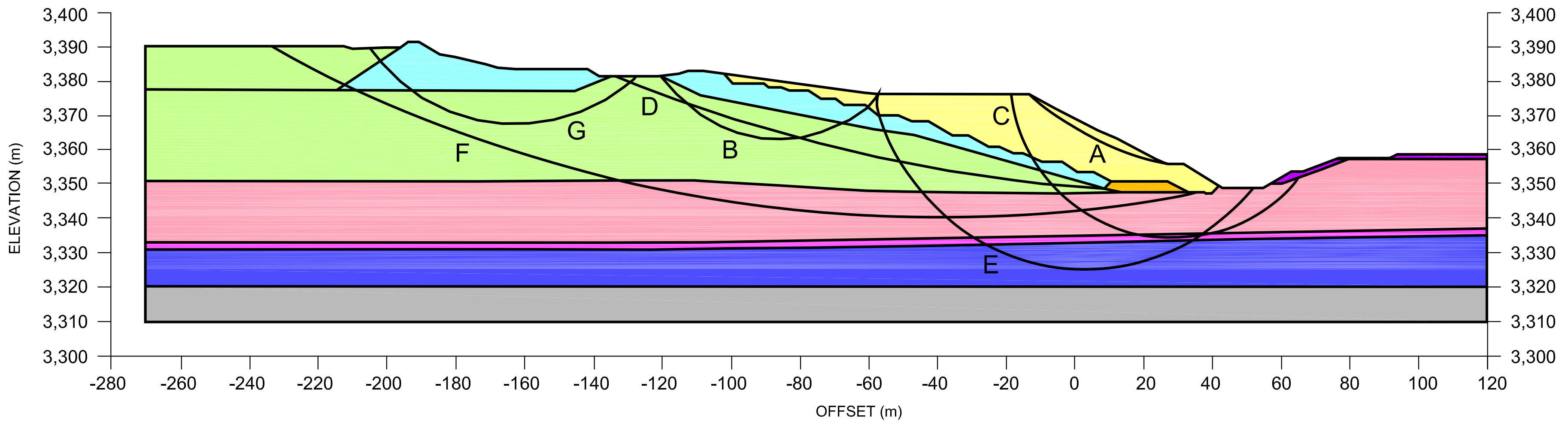
5.2 Limit Equilibrium Analysis Results

A summary of computed factors of safety as compared to all three targets – long-term, short-term and post seismic is provided in Table 5-1. The graphical results of the limit equilibrium slope stability analyses for the short-term and post-seismic analyses for the APA berm stabilization are provided in Appendix E. Appendix E and Table 5-1 do not include the preliminary analyses for closure cover.

The results of the preliminary analyses of the closure cover scenarios indicate that the closure cover could impact the overall stability of the Old TDF depending on details. The conceptual cover design indicates a 9% slope from the crest of the APA berm generally to the crest of the OEB and then a conformal cover from the OEB to the crest of the seismic berm. If the cover only extends to the crest of the OEB, the post-seismic target factor of safety of 1.2 is achieved. If a conformal cover extends from the OEB crest to the seismic berm crest, the post seismic factor of safety for the Mode D failure is somewhat less than target. The static short-term and long-term factors of safety are all greater than the targets.

Table 5-1: Summary of Factors of Safety

Plane	Failure Surface	Long-term Static Factor of Safety, Target Factor of Safety = 1.5	Short-term Static Factor of Safety, Target Factor of Safety = 1.3	Post Seismic Factor of Safety; Target Factor of Safety = 1.2
1	A	n/a	n/a	n/a
	B	n/a	n/a	n/a
	C	n/a	n/a	n/a
	D	n/a	n/a	n/a
	E	n/a	n/a	n/a
	F	n/a	n/a	n/a
	G	3.8	1.9	1.2
2	A	1.5	1.5	1.5
	B	3.7	2.1	1.7
	C	1.7	1.7	1.6
	D	2.5	1.9	1.4
	E	2.4	2.2	1.7
	F	2.6	2.3	1.6
	G	3.1	1.7	1.3
3	A	1.7	1.7	1.7
	B	4.7	2.9	2.4
	C	1.8	1.8	1.8
	D	2.7	2.2	1.4
	E	2.2	2.0	1.8
	F	2.7	2.5	1.8
	G	3.0	1.7	1.3
4	A	1.5	1.8	1.8
	B	4.4	2.6	2.1
	C	1.7	1.5	1.5
	D	2.2	2.0	1.5
	E	2.1	2.1	1.6
	F	2.3	2.2	1.5
	G	3.4	1.5	1.4
5	A	1.5	1.5	1.5
	B	4.9	2.7	2.2
	C	1.7	1.7	1.6
	D	2.0	1.4	1.2
	E	1.8	1.7	1.3
	F	2.3	1.7	1.4
	G	3.8	2.0	1.5
6	A	4.5	2.5	2.1
	B	5.2	3.4	2.8
	C	5.1	5.1	3.6
	D	4.6	2.7	2.3
	E	4.9	3.7	2.3
	F	5.6	2.7	2.1
	G	2.2	1.7	1.2



MATERIALS LEGEND

SEISMIC BERM FILL	GLACIAL TILL
EMBANKMENT RAISES	TAILINGS
STARTER EMBANKMENT	UNIT 3 TRANSITIONAL GLACIOFLUVIAL
MINE ROAD FILL	UNIT 4 GLACIOLACUSTRINE
UNIT 2a GLACIO FLUVIAL	

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SCALE:	AS SHOWN

PROJECT:
MYRA FALLS TAILINGS DISPOSAL FACILITY
AMALGAMATED PASTE AREA BERM STABILIZATION

TITLE:
LIMIT EQUILIBRIUM SLOPE STABILITY FAILURE MODES

DATE:	AUGUST 2016
PROJECT NO:	NX14001D.7.500
REV. NO:	A
FIGURE NO:	5.1

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6.0 DYNAMIC DEFORMATION MODELING

6.1 Introduction

A two dimensional (2D), nonlinear, dynamic finite element (FE) model using the proprietary software LSDYNA v.978 by Livermore Software Technology was developed to estimate potential static and seismic deformations of the Old TDF. The cross-section considered in the model was based on Plane 5. Plane 5 was chosen for more detailed dynamic analysis because the tailings and GLU are thickest at this location compared to other planes as well as the lowest static and post-seismic factors of safety which were for Mode D. The Plane 5 cross-section and soil stratigraphy is shown in Figure 4-5.

Geotechnical material properties are provided Table 4-1. Additional properties used in the dynamic deformation modeling are provided in Table 6-1 and discussed in Appendix F.

6.2 One Dimensional Seismic Wave Propagation Analysis

One dimensional (1-D), seismic wave propagation analysis was completed in order to perform initial calibrations and checks required for the 2-D LSDYNA modeling. The computer program DESRA-2C and a later version of the program referred to as DESRAMOD (Lee and Finn, 1978) were used for this purpose. A summary of the 1-D analyses is provided in the following paragraphs and also is discussed in detail in Appendix F.

Two 1-D soil columns referred to as Soil Profile 1 and Soil Profile 2 on Plane 5 were considered in the 1-D analyses. The locations are shown in Figure 4-5. The DESRAMOD analysis was run for both soil profiles using the four earthquake records discussed in 3.3. The LSDYNA analysis was run for only one soil profile and two of the earthquake records.

Computed peak cyclic stress ratios (CSR) versus depth from both models are shown in Figure 6-1. There is a relatively tight band of computed peak CSR for all input ground motions with values in the range of 0.15 to 0.2, indicating a close agreement between the two models. Computed peak CSRs in the range of 70 to 100% of the peak undrained strengths of the tailings and GLU develop during shaking.

Peak shear strains versus depth are shown Figure 6-2. Both models indicate similar trends of shear strain versus depth. The largest computed shear strains (in the range of 0.5 to 1.5%) were in the tailings and GLU, suggesting that largest contributions to downslope lateral movements of the Old TDF will occur due to cyclic straining in these materials. Smaller strains and lateral movements are computed within the higher strength glaciofluvial materials.

A comparison of lateral displacement versus time relative to the input base displacement between the DESRAMOD and LSDYNA models is shown in Figure 6-3. Agreement is excellent between the two models in terms of peak displacement amplitudes, in frequency content of the displacement time history, and in residual displacements at the end of shaking.

Figure 6-1 through Figure 6-3 indicate close agreement between the two 1-D models.

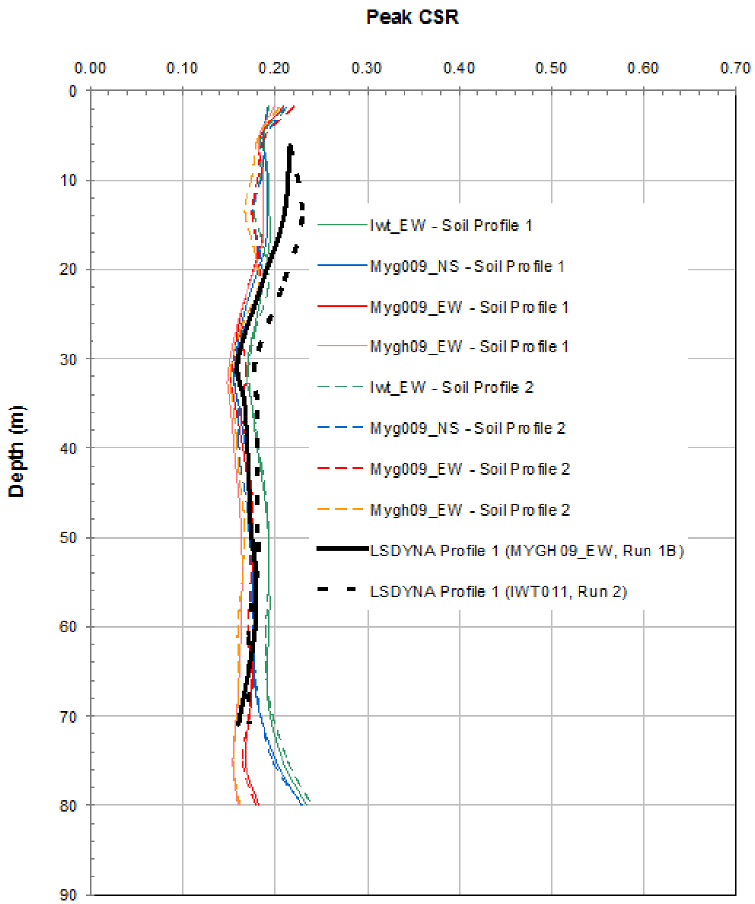

Figure 6-1: Comparison of Computed Peak CSR's Versus Depth from 1-D DESRAMOD and LSDYNA Models

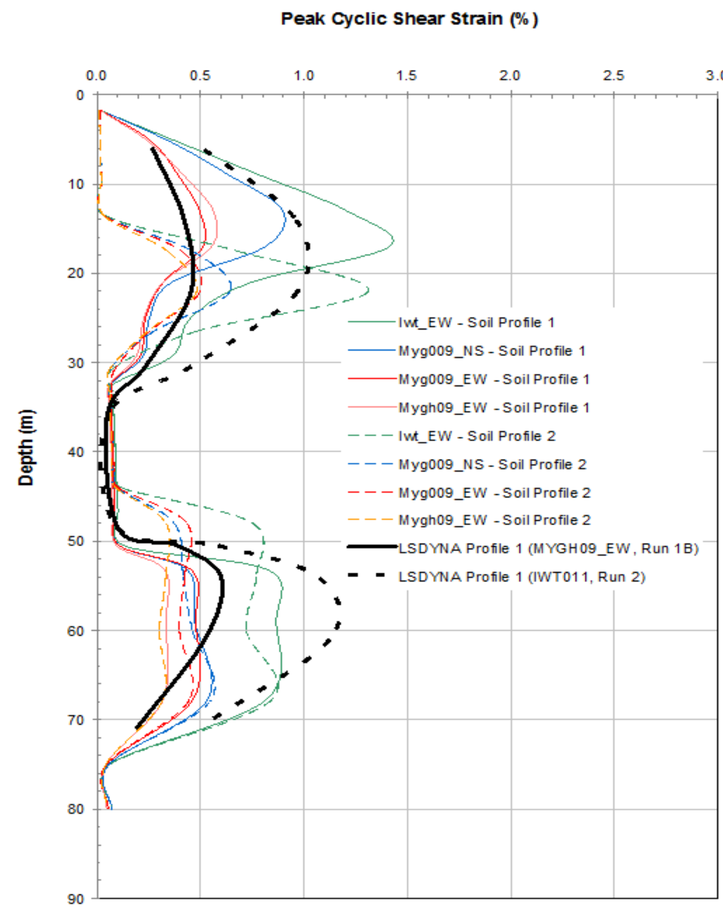

Figure 6-2: Comparison of Computed Peak Shear Strains Versus Depth from 1-D DESRAMOD and LSDYNA Models

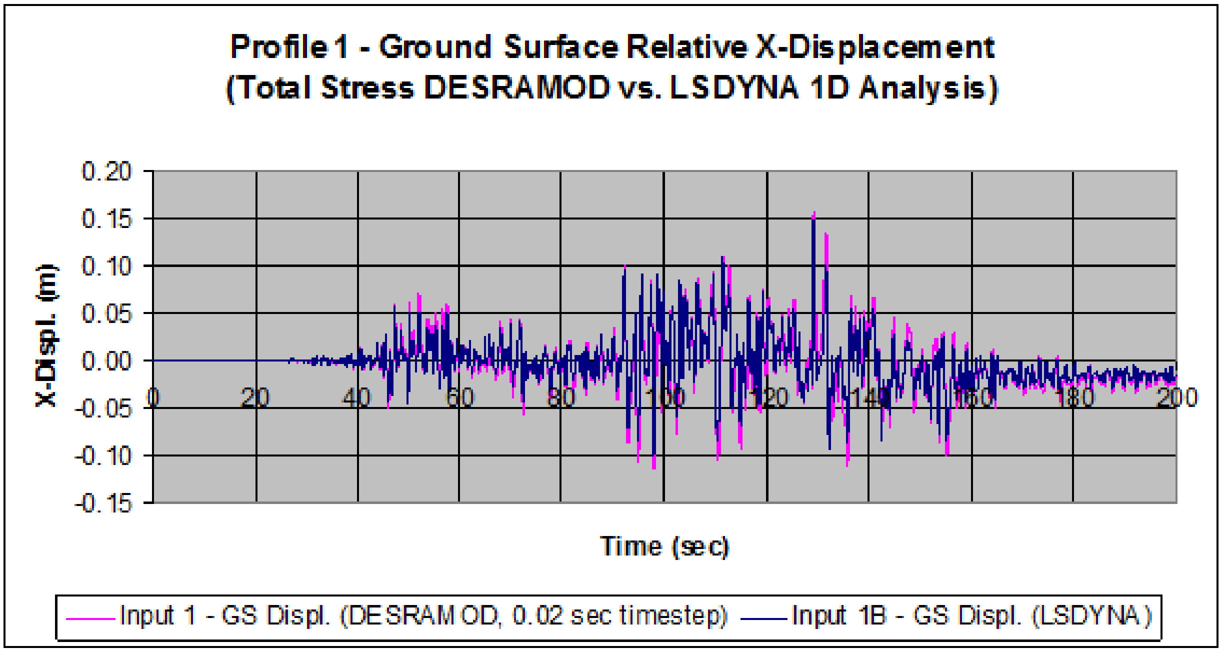

Figure 6-3: Comparison of Computed Relative Lateral Displacement Versus Time at Ground Surface for Soil Profile 1 Considering Earthquake Record 1 Input from 1-D DESRAMOD and LSDYNA Models

Table 6-1: Summary of Geotechnical Properties Used in Dynamic Modeling

Material Type	Total Unit Weight, γ (kN/m³)	Stress Normalized Seismic Shear Wave Velocity, V_{s1} (m/sec)	Seismic Shear Wave Velocity, V_s (m/sec)	Small Strain Shear Modulus, G_{max} (MPa)	K_0	Elastic Bulk Modulus, B_{elas} (MPa)	Static		Cyclic		
							Peak Friction Angle, Φ' (deg.)	S_u/σ'_{vo}	Equivalent Peak Friction Angle, Φ_{equiv} (deg.)	S_u/σ'_{vo}	R
Seismic Berm	22	270	221-291	110-190	0.38	71-123	38	n/a	38(5)	n/a	2(5)
Embankment Raises	22	270	244-336	133-253	0.44	87-143	34	n/a	34(5)	n/a	2(5)
Tailings	22.5	195	160-330(2)	60-250(2)	0.75	600-2500	13.2	0.20	11.9	0.18	150
Starter Embankment	22	280	383	329	0.41	183	36	n/a	36(5)	n/a	2(5)
Dense Glaciofluvial Sand and Gravel with loose layers (Unit 2a)	24	210	350-370	300-330	0.44	3000-3300	34	n/a	34(5)	n/a	150
Transitional Silty Sand (Unit 3)	22	203	359(3)	290 (3)	0.47	2900	32	n/a	29	0.35	150
Glaciolacustrine Silt and Clay (Unit 4)	20	175	285-335(2)	165-230(2)	0.75	1650-2300	14.5	0.22	14.5(5)	0.22(5)	150
Dense Glacial Till	24	≈230-240	450	495	0.75	4950	35(1)	n/a	35(1)	n/a	2(5)

Φ_{equiv} = equivalent peak friction angle designed to match target shear strength ratio

(1) Shear strength calculated assuming pore water cavitation
(2) V_s (and G_{max}) depends on stress level according to relationship with V_{s1}
(3) Average V_s, G_{max} and B_{elas} properties across all zones were selected for the Unit 3 transitional glaciofluvial materials which were assumed to have a 2m thickness across the model.
(4) Average V_s, G_{max} and B_{elas} properties across all zones were selected for the Unit 2A glaciofluvial materials which was assumed to have a thickness of 18 m across the model except at the far RHS of the model where the thickness increased by approximately 7 m. Depth dependence of these material parameters was considered.
(5) No change from initial static properties
n/a = not applicable

6.3 Two Dimensional Seismic Wave Propagation Analysis

A two dimensional (2-D) finite element model of the Plane 5 cross-section was developed using LSDYNA. The 2-D model is shown in Figure 6-4. A summary of the 2-D analyses is provided in the following paragraphs. A detailed technical discussion is provided in Appendix F.

The model was divided into seven zones to account for differing overburden stress conditions and to permit stress level variations in small strain shear modulus, G_{max} and elastic bulk modulus, B_{elas} to be considered. Soil zone numbering increased from left to right across the model with Zone 1 on the left and Zone 7 on the right of the model. The boundaries of each zone are indicated by the vertical lines shown in Figure 6-4. Also indicated in Figure 6-4 are five nodes. The nodes are located at critical points on the model where estimates of horizontal displacement are desired. From left to right, the nodes represent the crest of the APA berm (4718), the toe of the APA berm (4656), the crest of the seismic upgrade berm (4823) and the toe of the seismic upgrade berm (2965).

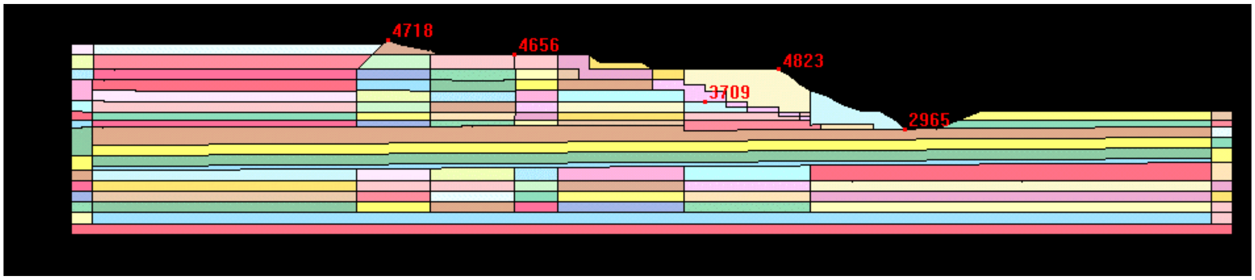

Figure 6-4: 2-D LSDYNA Model of Plane 5 Cross-Section, Showing Locations of Nodal Output

The left hand and right hand side of the model has a height of 90 m and 58 m, respectively. The width of the model is 550 m. Elastic elements are placed on the left hand, right hand and bottom boundaries of the model.

6.3.1 Nonlinear Stress-Strain Response Using Geologic Cap Model

The various soil zones were assumed to have nonlinear, hysteretic response during cyclic shearing with limiting shear strength calculated using a Mohr-Coulomb failure criterion. Nonlinear material response for both static and cyclic loading was modelled using a nonlinear “geologic cap model” or GCM (Livermore Software Technology, 2001). For small changes in shear stress, response is linear elastic and elastic strains occur. For larger changes in shear stress, plastic strains occur resulting in nonlinear shear stress – shear strain response. Shear stresses are limited according to the Mohr-Coulomb failure criterion.

6.3.2 Stages of 2-D Modeling

The 2-D modeling was completed in three stages. Brief descriptions of the three stages are as follows:

- Stage 1: Initial self-weight gravity loading of the dam (seismic berm, APA berm and embankment fills) and its foundations to calculate pre-earthquake effective stresses throughout the model.

- Stage 2: Dynamic modeling of dam response with no cyclic strain accumulation considered from cyclic pore pressure generation.

- Stage 3: Post-seismic stability analysis of dam response assuming cyclic shaking results in strength reduction relative to pre-earthquake strengths within the tailings, GLU and Unit 3 glaciofluvial soils.

6.3.3 Two Dimensional Dynamic Deformation Modeling Results

The following results are presented:

- Contours of pre-earthquake vertical effective stresses (Stage 1) throughout the model are shown in Figure 6-5.

- Contours of maximum shear strain at the end of seismic shaking using input Earthquake Record 1 with no consideration of cyclic degradation (Stage 2) in the GLU, tailings and Unit 3 materials, i.e. based on the pre-earthquake "static" soil properties are shown in Figure 6-6. Examination of this figure indicates a critical slip surface (zone of highest shear strains) extending from upstream of the seismic berm and exiting near the toe of the seismic berm. This mode of deformation is similar to the critical potential failure surface identified by Surface D from SLOPE-W modeling which indicated a post-seismic factor of safety of 1.2 for this mode of potential failure. The shear strains output from LSDYNA (termed "Green's strains") are ½ of engineering shear strains used in normal engineering practice.

- Contours of maximum shear strain at the end of seismic shaking using input Earthquake Record 1 with consideration of cyclic degradation (Stage 3) in the GLU, tailings and Unit 3 materials, i.e. based on the "cyclic" soil properties are provided in Figure 6-7. The broadening of the zones of maximum shear strain are apparent using the cyclic soil properties when comparing Figure 6-6 and Figure 6-7.

- Plots of peak cyclic shear stress ratio versus depth are provided in Figure 6-8 within various soil zones (Zones 2, 4, 5 and 6) considering input Earthquake Records 1 through 4. The CSR's show a reasonably tight band versus depth considering the various input earthquake records. A reasonable average peak CSR within the GLU, Unit 3 glaciofluvial and tailings is 0.2 with values as high as 0.25 due to locally higher K_0 values during shaking. The peak CSR's indicate that the maximum undrained shear strengths within the tailings and GLU are reached during shaking.

- A summary of computed permanent lateral displacements at the end of shaking considering five input ground motions is given in Table 6-2. The model was run with and without the effects of cyclic degradation. As expected, including the effects of cyclic degradation increased post-seismic displacements by up to 35%. Maximum displacements were computed using input Earthquake Record 1 which has the highest Arias Intensity of all records considered. ***Maximum lateral displacements at the crest of the seismic berm of 0.96 m were computed which is considered to be within acceptable limits.*** The relatively limited lateral displacements computed are considered to be the result of the relatively flat slopes and buttressing effect of the seismic berm, as well as the large amount of damping considered in the model due to cyclic hysteresis within the tailings and GLU.

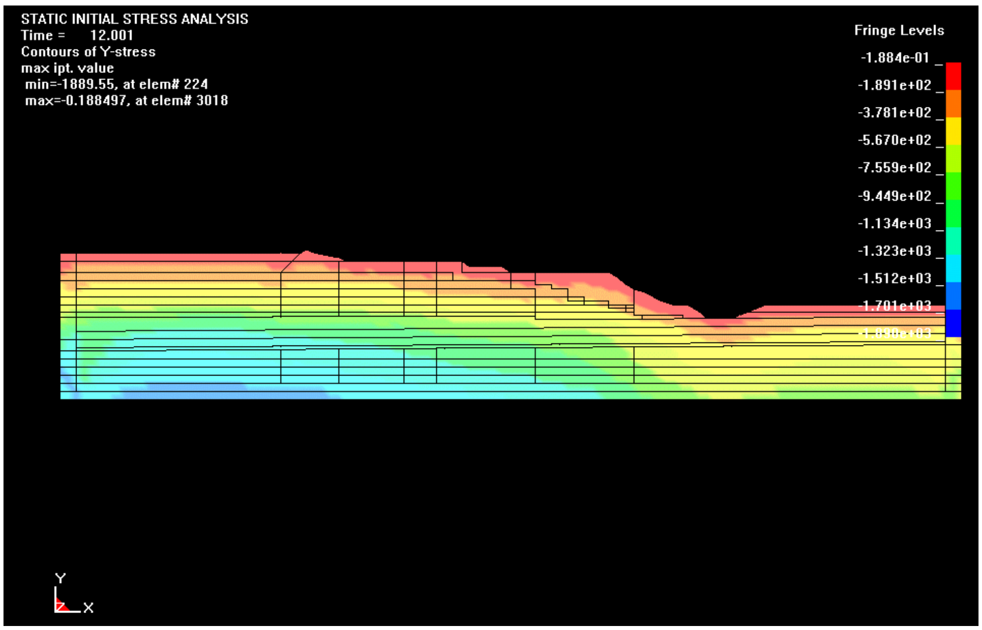

Figure 6-5: Contours of Pre-Earthquake Vertical Effective Stress

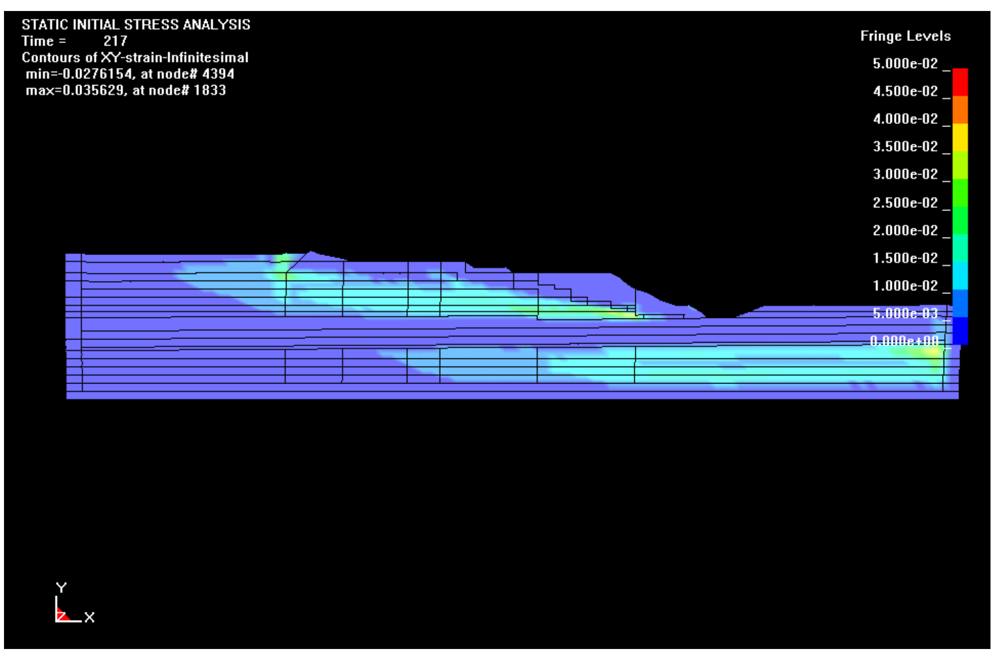

Figure 6-6: Contours of Post-Earthquake Maximum Shearing Strains with No Effects of Cyclic Degradation

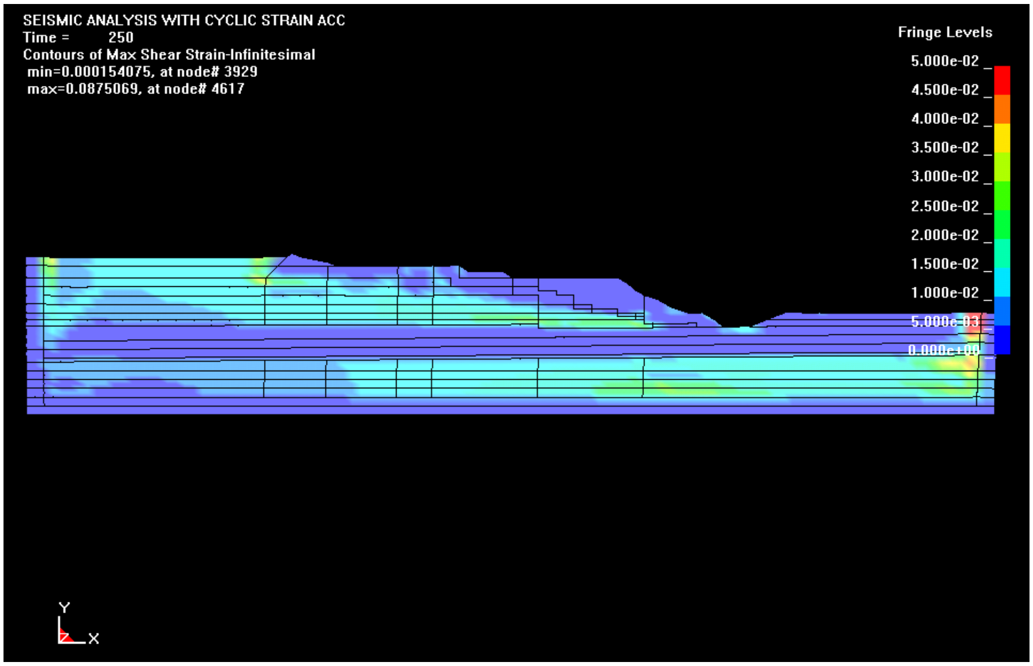

Figure 6-7: Contours of Post-Earthquake Maximum Shearing Strains with Effects of Cyclic Degradation

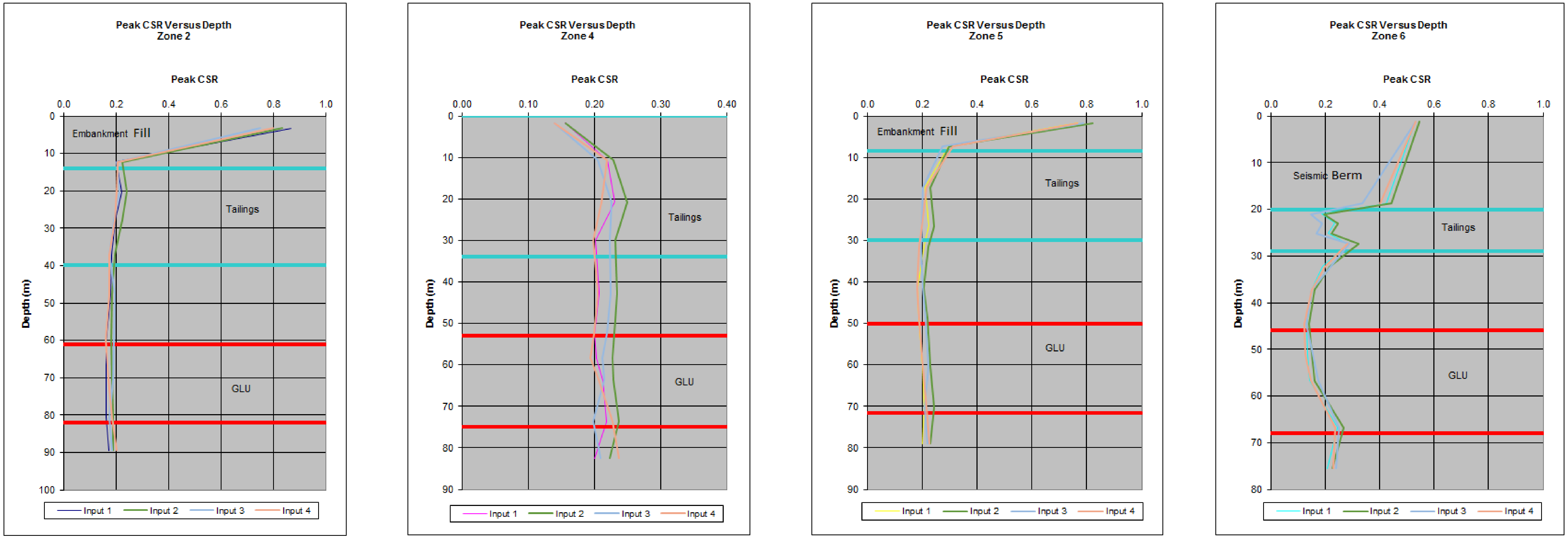

Figure 6-8: CSR for Earthquake Records 1 through 4

6.3.4 Approximate Methods to Estimate Seismic Displacements

Approximate methods were used to estimate post-seismic dam displacements to provide a check against the LSDYNA model results using the following approaches:

- **Bray and Travasarou (2007).** The Bray and Travasarou (B&T) model is based on a statistical analysis of a simplified 1-D nonlinear slope model considering a variety of seismic input motions over a range of earthquake magnitudes. The B&T displacements are sensitive to yield acceleration coefficient (k_y) and effective horizontal acceleration time history at the centroid of the slide mass. The latter is indirectly related to the input ground motion spectral acceleration characteristics which need to be specified at a structural period equal to 1.5 times the small strain fundamental period of the dam. The latter has been previously estimated to be approximately 0.9 seconds under the crest of the seismic berm. Using the B&T empirical equations and using a critical yield acceleration coefficient of 0.10 g determined from SLOPE-W analyses for Profile 5, the mean post-seismic displacements at the crest of the seismic berm for a M9 design earthquake are computed to be 1.0 m.

- **Newmark (1965)** subjected to an input base acceleration time history. An in house computer program was used to compute cumulative displacements of the rigid block which occur progressively whenever the base acceleration exceeds the yield acceleration of the soil mass above a critical slip surface. The horizontal acceleration time history computed by LSDYNA at node 3709 using input Earthquake Record 1 with no consideration of cyclic degradation effects was used in the Newmark analysis. Node 3709 is located at the approximate centroid of the soil mass above a critical slip surface determined from SLOPE-W analysis (Surface D). A k_y of 0.10 also was used in the modeling. The Newmark model predicted a post-seismic lateral displacement of 1.09 m.

- **Idriss and Boulanger (2008)** which involves estimating post-seismic shear strains at different depths in a 1-D soil profile and integrating these strains versus depth to compute X-displacements at the soil surface. Considering a 1-D soil profile under the crest of the seismic berm (Zone 6), shear strain potentials were estimated within the tailings and GLU based on effective CSR's of 0.10 in these layers and using available cyclic lab test data. Maximum strain potentials of 5% and 1% were used in the tailings and GLU, respectively Shear strain potentials in the Unit 3 glaciofluvial were estimated to be equal to 2% as described in Section 5.2.1 (c). The shear strain profile assumed is shown in Figure 6-9. Integration of this strain potential predicted a horizontal ground displacement at the crest of the seismic berm equal to 0.91 m. The horizontal shear strains computed using the 2D LSDYNA for this soil profile and considering input Record (1), which predicted maximum post-seismic X-displacement at the crest of the seismic berm, are also shown in Figure 5.15. The latter indicate lower strain development in the tailings due to the buttressing effect of the adjacent seismic berm, and larger strain development in the deeper GLU than estimated using the strain potential method.

The above simplified estimates of post-seismic lateral displacement at the crest of the seismic berm are in close agreement with the 2D finite element model results.

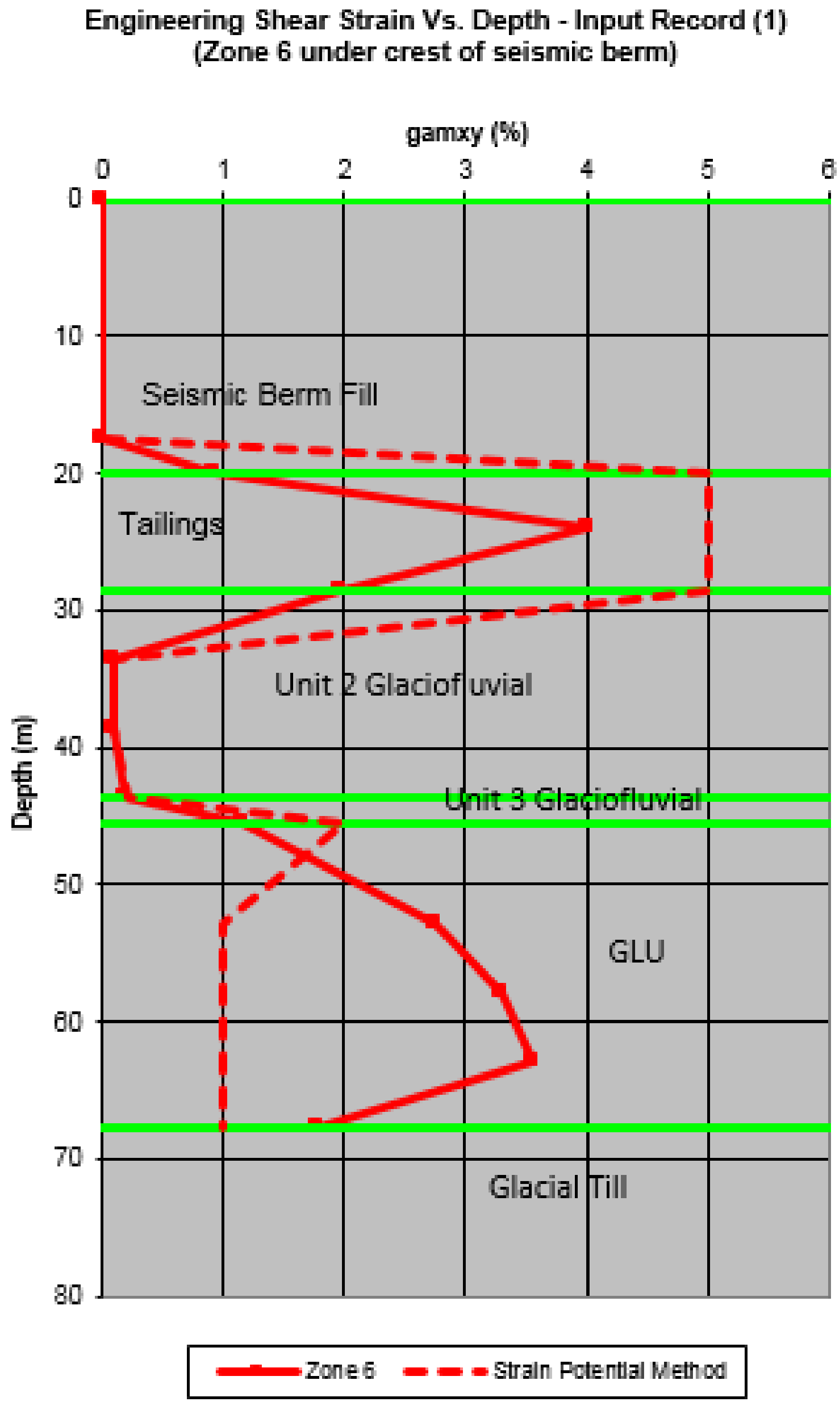

Figure 6-9: Comparison of Post-Seismic Shear Strain from LSDYNA and Strain Potential Method

6.3.5 Post-Seismic Stability Assessment

Cyclic straining under design levels of seismic shaking is considered to result in post-seismic strength reduction in the tailings, GLU and Unit 3 transitional glaciofluvial materials. The following post-seismic residual strength ratios were considered:

- Tailings S_r/σ'_{v0}= 0.14
- GLU S_r/σ'_{v0}= 0.13
- Unit 3 glaciofluvial (silty sand) S_r/σ'_{v0}= 0.35

K_0 values of 1.0 were assigned to the tailings and GLU, considered representative of post transient liquefaction conditions, giving equivalent friction angles of 8° and 7.5°, respectively. A K_0 value of 0.47 was assigned to the Unit 3 transitional glaciofluvial since transient liquefaction is not expected within these materials under design levels of shaking. An equivalent friction angle of 29° was assigned to the Unit 3 materials.

It was also assumed that following seismic shaking drained deformations would occur in the Unit 2A glaciofluvial and Unit 3 transitional glaciofluvial deposits. Because of the low permeability and long drainage path lengths in the tailings and GLU, it was assumed that deformations following seismic shaking would be essentially undrained.

The 2D model was then re-run following the end of seismic shaking considering input Earthquake Record 1, which predicted the maximum seismic lateral displacements of all the records considered. Computed post-seismic lateral and vertical displacements are presented in Table 6-2 and Table 6-3, respectively. Using the reduced post-seismic strengths, the Old TDF is predicted to be stable and no additional lateral and vertical displacement results following the end of seismic shaking at the ground surface points output. Excess pore pressures are expected to be induced in the tailings and GLU which will dissipate slowly years following a major seismic event. This will lead to additional long term settlements of the dam which have not been considered in the present evaluation.

Table 6-2: Summary of Computed Post-Seismic Horizontal Displacements

Earthquake Record	Comments	Lateral Displacement (m)			
		Crest of APA Berm (Node 4718)	Toe of APA Berm (Node 4656)	Crest of Seismic Berm (Node 4823)	Toe of Seismic Berm (Node 2965)
1	No cyclic degradation	0.65	0.69	0.73	0.60
2	No cyclic degradation	0.60	0.65	0.71	0.56
3	No cyclic degradation	0.39	0.44	0.50	0.40
4	No cyclic degradation	0.53	0.56	0.66	0.52
1	With cyclic degradation	0.85	0.95	0.96	0.79
2	With cyclic degradation	0.85	0.94	0.94	0.76
3	With cyclic degradation	0.48	0.56	0.63	0.50
4	With cyclic degradation	0.62	0.71	0.79	0.65
6	With cyclic degradation	0.58	0.65	0.68	0.53
1	Post-seismic residual strengths	0.85	0.95	0.96	0.79

Table 6-3: Summary of Computed Post-Seismic Vertical Displacements

Earthquake Record	Comments	Vertical Displacements (m)			
		Crest of APA Berm (Node 4718)	Toe of APA Berm (Node 4656)	Crest of Seismic Berm (Node 4823)	Toe of Seismic Berm (Node 2965)
1	Post-seismic residual strengths	0.21	0.28	0.09	-0.06

7.0 CONCLUSIONS AND RECOMMENDATIONS

The current post-seismic instability of the APA berm can be mitigated by raising the elevation of the toe berm to a uniform elevation of 3386 m with a uniform width of 15 m. Post-seismic factors of safety are equal to or exceed the target of 1.2 for all modes of failure assessed using limit equilibrium methods. The lowest computed post-seismic factor of safety was for the Mode D failure on Plane 5. Limit equilibrium modelling indicates that the relatively minor amount of additional fill required to improve the post-seismic stability of the APA berm does not adversely impact the overall stability of the Old TDF including the Mode D failure. This is in line with the conclusion made during the design of the APA berm that it would not have an adverse impact because it was sufficiently distant from the seismic upgrade berm (Klohn-Crippen, 2001). Permit-level drawings for the APA berm stabilization are provided in Appendix G.

The models used in the static and dynamic deformation modeling included the additional APA berm stabilization fill. The estimated deformation of the crest of the seismic upgrade berm and at the crest of the stabilized APA berm are on the order of one meter. Checks using three approximate methods also indicated post-seismic deformations at the crest of the seismic berm of similar magnitudes. Deformations of this magnitude are generally considered within tolerable limits for facilities such as the Old TDF. In fact, the estimated post-seismic deformations are significantly less than those estimated during the design of the seismic upgrade berm. The limited post-seismic lateral deformations are considered to be the result of the relatively flat slopes and buttressing effect of the seismic berm as well as the large amount of damping considered in the LSDYNA model resulting from cyclic hysteresis within the tailings and GLU.

As discussed in Section 5.0, preliminary limit equilibrium slope stability analysis were conducted on Plane 5 to assess the impact that closure cover might have on the stability of the Old TDF. The results of the preliminary analyses suggest that the closure cover cannot extend over the 6:1 slope to the crest of the seismic berm. Approximately 1.5 m of clean fill was placed over the 6:1 slope and the seismic berm during the final stages of construction and, depending on the specific requirements for closure cover, these areas could be considered essentially covered. If extension of the cover over the seismic berm is required, the 6:1 slope and seismic berm can be subcut to the thickness of closure cover such that there is no net increase in elevation when closure cover is placed. The stability also can be improved by the addition of a toe berm by raising the elevation of the road currently located near the toe of the seismic berm or, by flattening the 6:1 slope as discussed in the draft closure plan (NMF, 2011).

The analyses were only conducted for Plane 5. Planes 1 through 4 and 6 will need to be assessed during the design of the closure cover. It is also important to note that all closure cover scenarios generally improve the stability of the APA berm.

The conceptual closure cover involves placing fill between the APA berm and the OEB to provide a smooth transition between the two berms. This could involve placing a significant thickness of fill. The fill essentially is an additional load at the top of the Old TDF and there is no off-setting load added to the toe. A conformal closure cover will minimize the amount of fill to be placed between the APA berm and the OEB.

It is important to note that the closure cover briefly described in the seismic upgrade design report does not extend over the seismic berm (Klohn-Crippen, 1999). This would suggest that additional loading caused by extending closure cover the 6:1 slope and the crest of the seismic berm might not have been considered in the design of the seismic berm.

Post-seismic deformations at the toe of the seismic upgrade berm are on the order of 0.8 m at Plane 5 which would suggest that post-seismic deformation of the outer perimeter drain might be of similar magnitude. This magnitude is significantly less than that estimated during design of the seismic upgrade berm. Design-phase estimates of deformations at the toe of the seismic upgrade berm during a major seismic event the design phase were around 2.5 m (Klohn-Crippen, 1999, 2005b). The design report was silent on the allowable deformations of the outer perimeter drain during a major seismic event or maximum design earthquake; however, the design criteria for maximum horizontal displacement during an operational basis earthquake was 0.3 m (Klohn-Crippen, 1999, 2005b). This would suggest that the outer perimeter drain might not survive the current design earthquake ground motions for Closure – Passive Care.

8.0 LIMITATIONS AND CLOSURE

This report has been prepared for the exclusive use of Nyrstar Myra Falls Ltd. for specific application to the area within this report. Any use which a third party makes of this report, or any reliance on or decisions made based on it, are the responsibility of such third parties. Amec Foster Wheeler accepts no responsibility for damages, if any, suffered by any third party as a result of decisions made or actions based on this report. It has been prepared in accordance with generally accepted soil and foundation engineering practices. No other warranty, express or implied, is made.

Respectfully submitted,
Amec Foster Wheeler Environment & Infrastructure,
a Division of Amec Foster Wheeler Americas Limited

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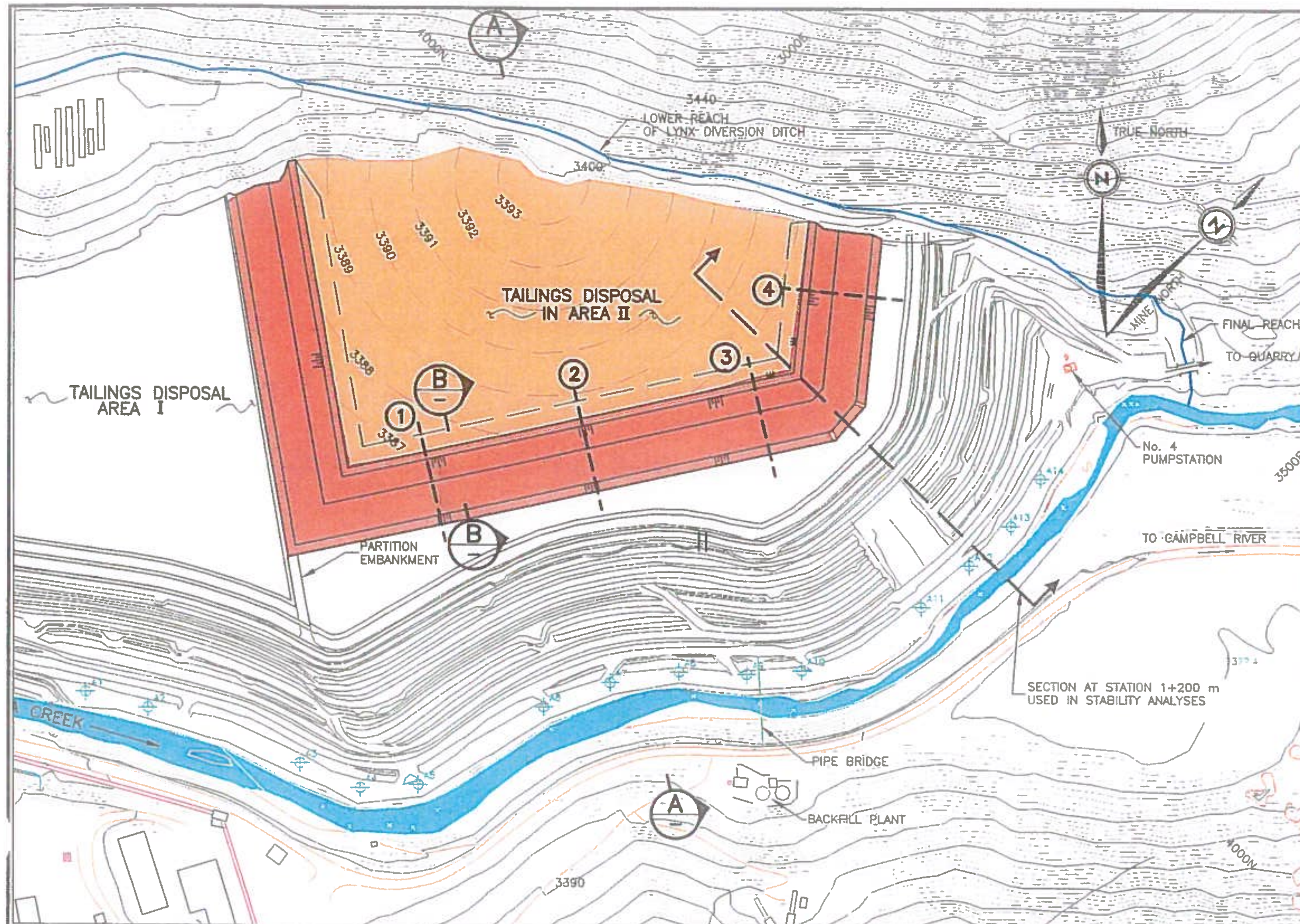
APPENDIX A

Design Drawings and Yearly Construction Record Drawings



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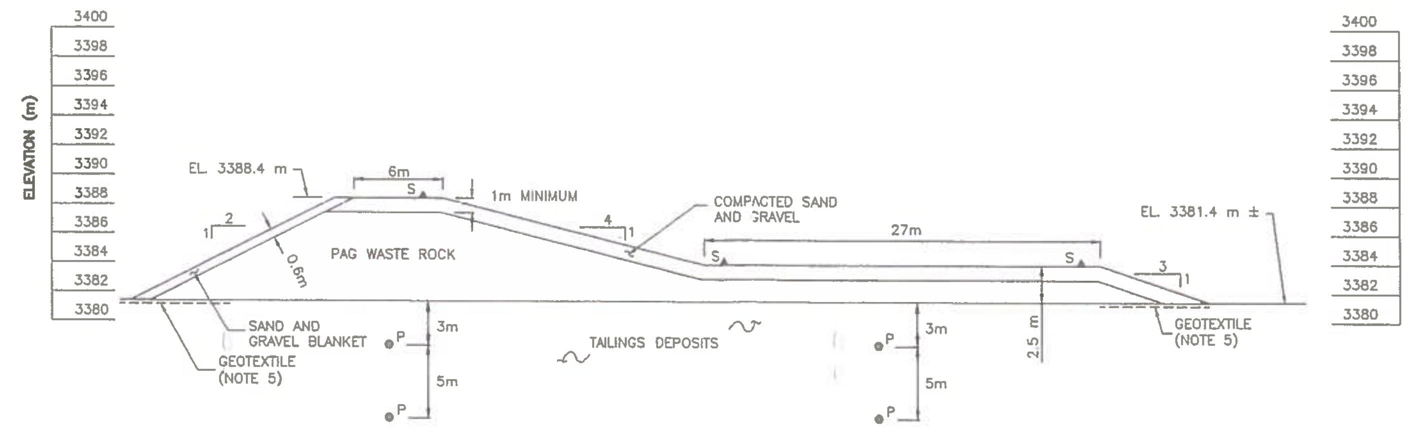


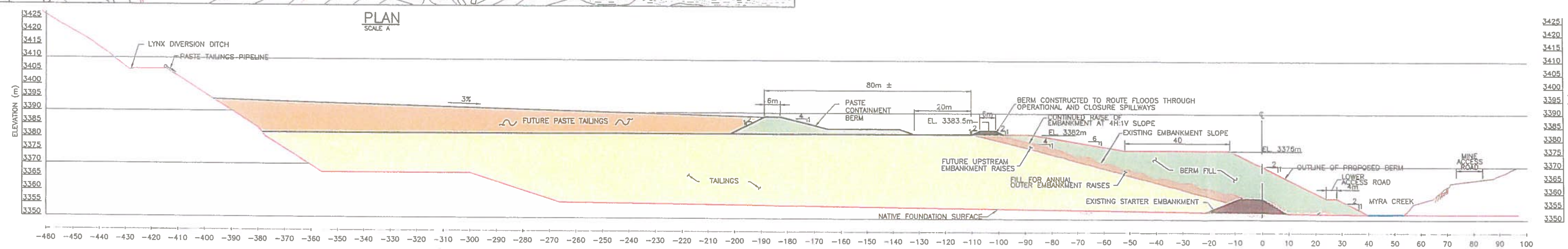
SECTION (B) CONTAINMENT BERM
SCALE B

LEGEND:

(1) — — —	INSTRUMENTATION SECTION
P	PIEZOMETER
S	DISPLACEMENT MONUMENT SURFACE

NOTES:

1. ALL ELEVATIONS AND DIMENSIONS ARE IN METRES.
2. TOPOGRAPHY BASED ON AERIAL SURVEY FLOWN IN SEPT. 1992, EXCEPT EMBANKMENT FACE PLANIMETRY WHICH IS BASED ON 1996 AND 1997 LAND SURVEY BY PARALLEL GEO-SERVICES. INC.
3. EMBANKMENT CREST AND TAILINGS ELEVATION FROM DECEMBER 1998 AND DECEMBER 1999 SURVEYS BY BOLIDEN.
4. DETAILS OF 1999 BERM CONSTRUCTION FOR SEISMIC UPGRADE PROJECT NOT SHOWN.
5. HEAVY DUTY NON-WOVEN GEOTEXTILE ADDED BENEATH OUTER SLOPES, IF REQUIRED.



SCALE C: 0 — 50m
SCALE B: 0 — 10 m
SCALE A: 0 — 100 m

NOTES:

1. SECTION A IS LOCATED AT STATION OF 0+900 m OF THE SEISMIC UPGRADE PROJECT.
2. DETAILS OF RIPRAP ARMOURING, CUTOFF WALL AND DRAINAGE WALL OMITTED FOR CLARITY.

SECTION (A)
SCALE C
TYPICAL SECTION OF PASTE TAILINGS DISPOSAL IN AREA II

TO BE READ WITH KLOHN-CRIPPEN REPORT DATED FEB 19, 2001

AS A MUTUAL PROTECTION TO OUR CLIENT, THE PUBLIC AND OURSELVES, ALL REPORTS AND DRAWINGS ARE SUBMITTED FOR THE CONFIDENTIAL INFORMATION OF OUR CLIENT FOR A SPECIFIC PROJECT AND AUTHORIZATION FOR USE AND/OR PUBLICATION OF DATA, STATEMENTS, CONCLUSIONS OR ABSTRACTS FROM OR REGARDING OUR REPORTS AND DRAWINGS IS RESERVED PENDING OUR WRITTEN APPROVAL.

NO.	DATE	ISSUE/REVISION	DRAWN	CHK'D	DESIGN	APP'D
A	JUL 13, 2001	FINAL	CYW		FL	

CLIENT: BOLIDEN WESTMIN (CANADA) LTD. MYRA FALLS OPERATION

KLOHN CRIPPEN

PROJECT: PASTE FILL PROJECT PASTE DISPOSAL IN TDF

TITLE: PLAN AND TYPICAL SECTIONS OF PASTE TAILINGS DISPOSAL IN AREA II

PROJECT No. PM8976 04 | DWG. No. D-1408 | Rev. A

CANCEL PRINTS BEARING PREVIOUS

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Xrefs: ... MYRA_PR1

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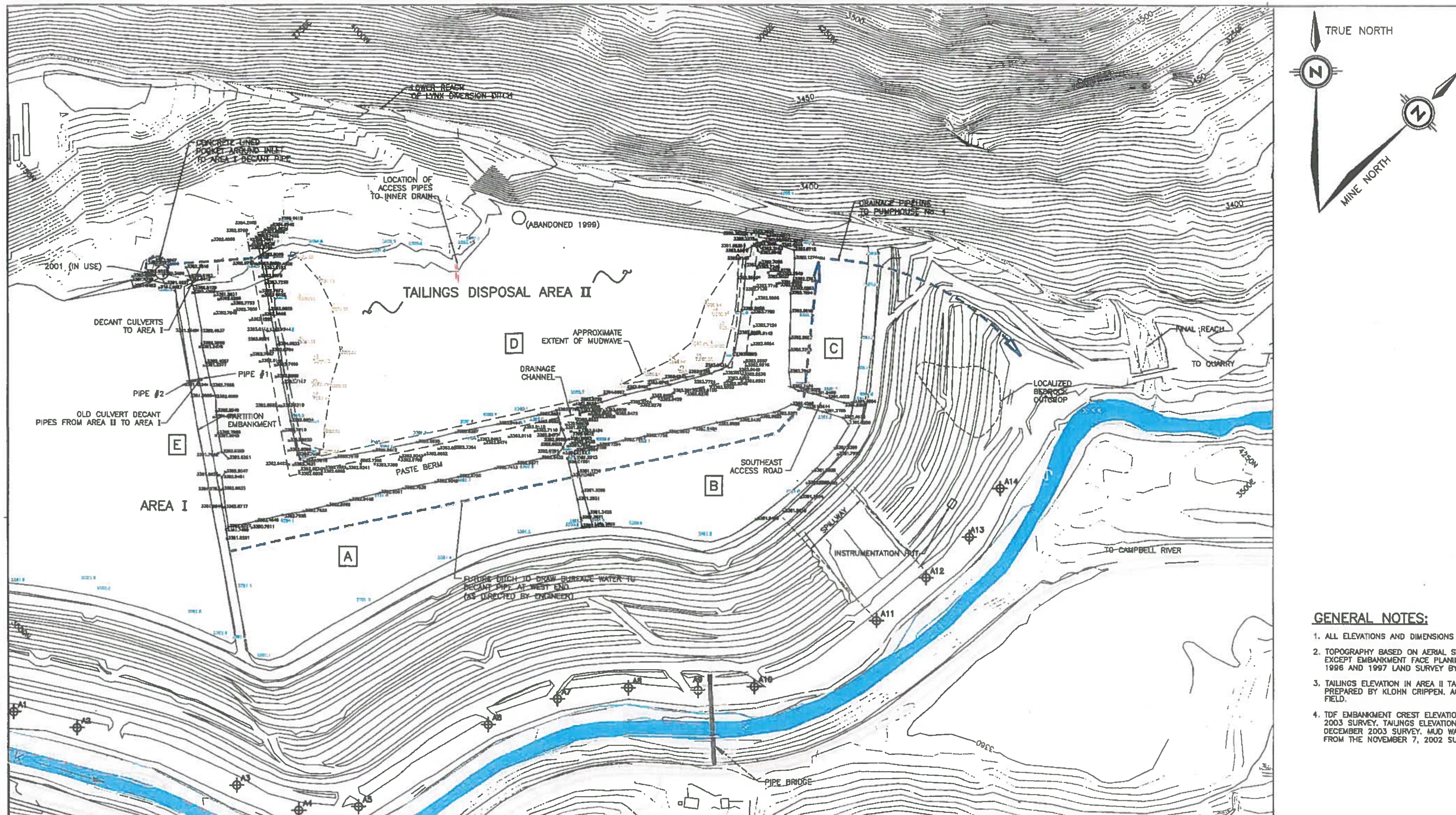

GENERAL NOTES:

1. ALL ELEVATIONS AND DIMENSIONS ARE IN METRES.
2. TOPOGRAPHY BASED ON AERIAL SURVEY FLOWN IN SEPT. 1992. EXCEPT EMBANKMENT FACE PLANIMETRY WHICH IS BASED ON 1996 AND 1997 LAND SURVEY BY PARALLEL GEO-SERVICES. INC.
3. TAILINGS ELEVATION IN AREA II TAKEN FROM THE 2001 ANNUAL REPORT PREPARED BY KLOHN CRIPPEN. ACTUAL ELEVATIONS WILL VARY IN THE FIELD.
4. TDF EMBANKMENT CREST ELEVATIONS UPDATED FROM THE JANUARY 2003 SURVEY. TAILINGS ELEVATIONS WERE UPDATED FROM THE DECEMBER 2003 SURVEY. MUD WAVE ELEVATIONS WERE PROVIDED FROM THE NOVEMBER 7, 2002 SURVEY.

LEGEND:

- MUDWAVE
- 2001 ○ DECANT TOWERS
- A-1 ACCESS CHAMBER FOR OUTER DRAIN
- 3378.7 TAILINGS BEACH ELEVATIONS SURVEYED DECEMBER 2002
- A DEPOSITION CELL
- MUDWAVE SURVEY POINT
- TAILINGS SURVEY POINT
- x3383.6612 AS-BUILT PASTE BERM SURVEY POINT

PLAN
SCALE A

SCALE A: 0 — 100m

PROFESSIONAL ENGINEER
PROVINCE OF BRITISH COLUMBIA
B.A. HOFMANN
#17987

June 30 2003

NO.	DATE	ISSUE / REVISION	DRAWN	CHK'D	DESIGN	APP'D
B	JUNE 30, 2003	ISSUED WITH PASTE BERM CONSTRUCTION SUMMARY FINAL REPORT	SS			
A	MAY 9, 2003	ISSUED WITH PASTE BERM CONSTRUCTION SUMMARY DRAFT REPORT				

DRAWING NO.	REFERENCE DRAWING

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CLIENT: BOLIDEN WESTMIN (CANADA) LTD. MYRA FALLS OPERATION

KLOHN CRIPPEN

PROJECT: PASTE FILL PROJECT
PASTE BERM STARTER EMBANKMENT

TITLE: AS-BUILT SURVEY OF PASTE BERM STARTER EMBANKMENT

SCALE	PROJECT No.	DWG. No.	REV.
AS SHOWN	PM 8976 04	D-4007	B

CANCEL PRINTS BEARING PREVIOUS REVISION

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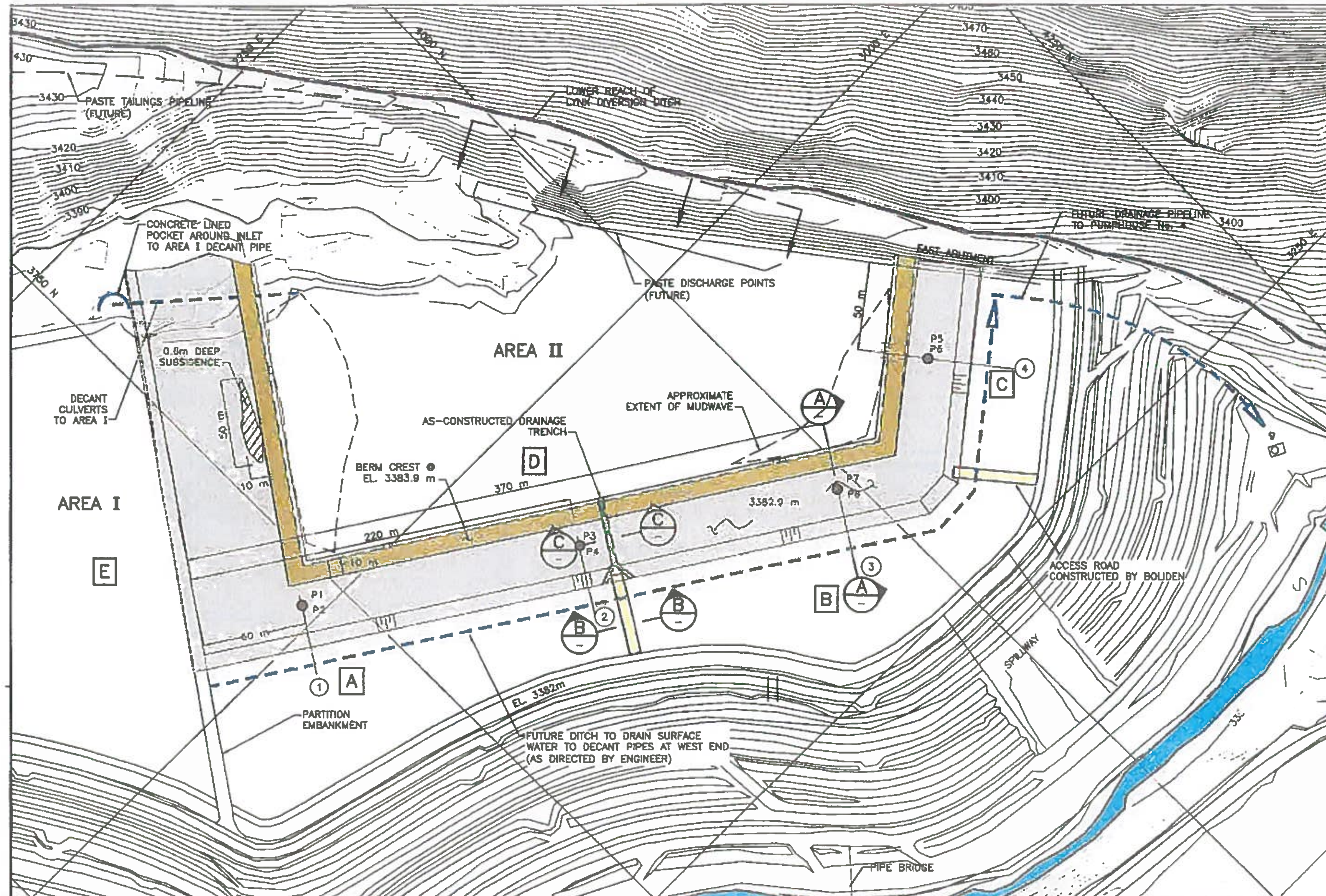

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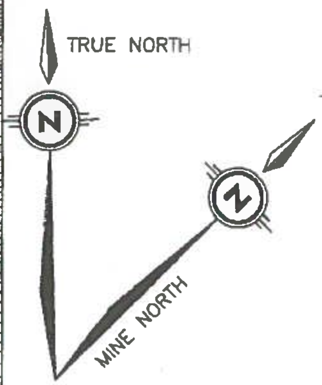
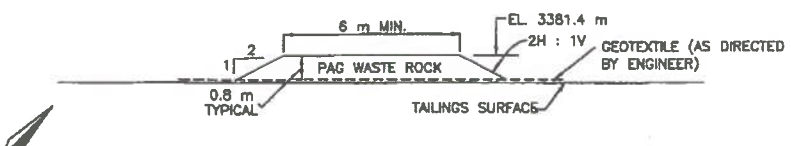
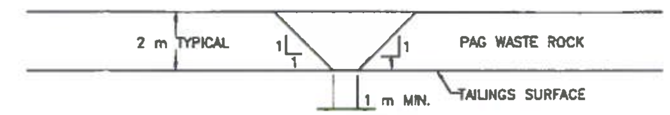
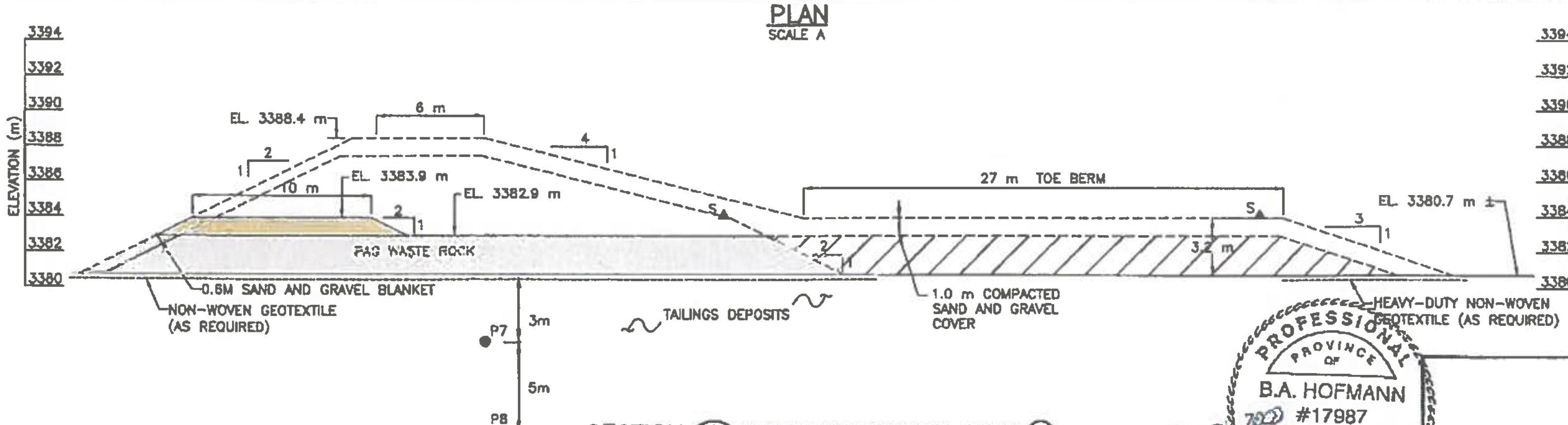




SECTION B TYPICAL INTERMEDIATE TAILINGS BERM
SCALE C

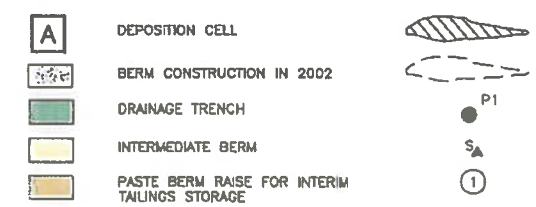


SECTION C DRAINAGE TRENCH
SCALE C



SECTION A INSTRUMENTATION LINE 3
SCALE B

LEGEND:



GENERAL NOTES:

1. ALL ELEVATIONS AND DIMENSIONS ARE IN METRES.
2. TOPOGRAPHY BASED ON AERIAL SURVEY FLOWN IN SEPT. 1992, EXCEPT EMBANKMENT FACE PLANIMETRY WHICH IS BASED ON 1996 AND 1997 LAND SURVEY BY PARALLEL GEO-SERVICES, INC.
3. TAILINGS ELEVATION IN AREA II TAKEN FROM THE 2001 ANNUAL REPORT PREPARED BY KLOHN CRIPPEN. ACTUAL ELEVATIONS WILL VARY IN THE FIELD.

SCALE C: 0 – 5 m
SCALE B: 0 – 10 m
SCALE A: 0 – 100 m

PROFESSIONAL ENGINEER — PROVINCE OF BRITISH COLUMBIA — B.A. HOFMANN #17987 — June 30 2003

NO.	DATE	ISSUE / REVISION	DRAWN	CHK'D	DESIGN	APP'D
B	JUNE 30, 2003	ISSUED WITH PASTE BERM CONSTRUCTION SUMMARY FINAL REPORT				
A	MAY 9, 2003	ISSUED WITH PASTE BERM CONSTRUCTION SUMMARY DRAFT REPORT				

DRAWING NO.	REFERENCE DRAWING

CLIENT: BOLIDEN WESTMIN (CANADA) LTD. MYRA FALLS OPERATION

KLOHN CRIPPEN

PROJECT: PASTE FILL PROJECT PASTE BERM STARTER EMBANKMENT

TITLE: PLAN AND SECTIONS PASTE BERM STARTER EMBANKMENT

SCALE	PROJECT No.	DWG. No.	REV.
AS SHOWN	PM 8976 04	D-4006	B

CANCEL PRINTS BEARING PREVIOUS REVISION

Scale: 1=10(PS) Drawing File: M:\M08976 – MFO Paste Fill\04 – Paste Berm Specs & Drawings\400 Design\410 Drawings\Paste Berm Con. Report-30Jun2003\D-4006-RB.dwg (asuzuki) Xrefs: XREF1, MYRA_PR1

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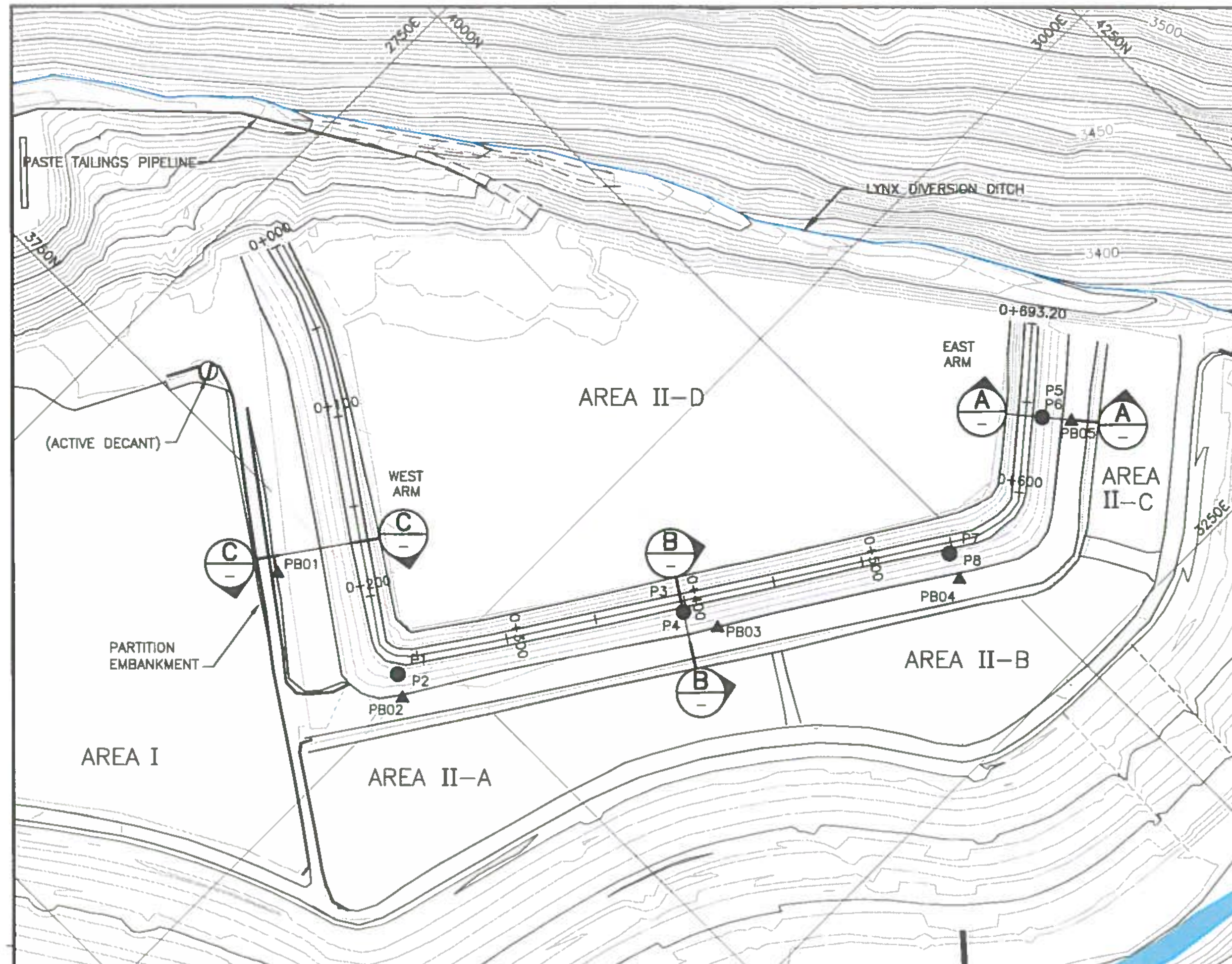
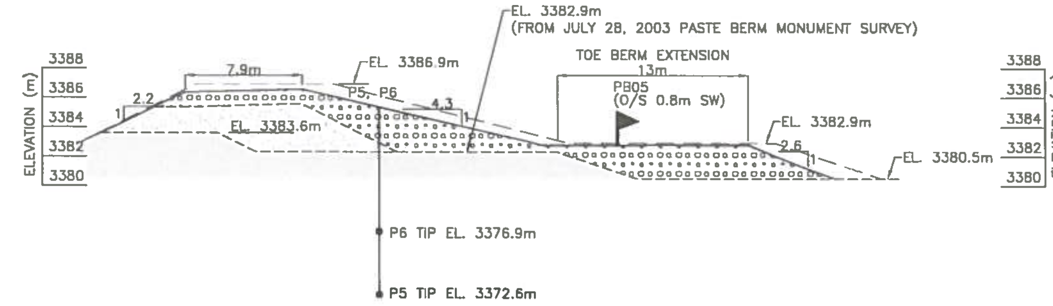


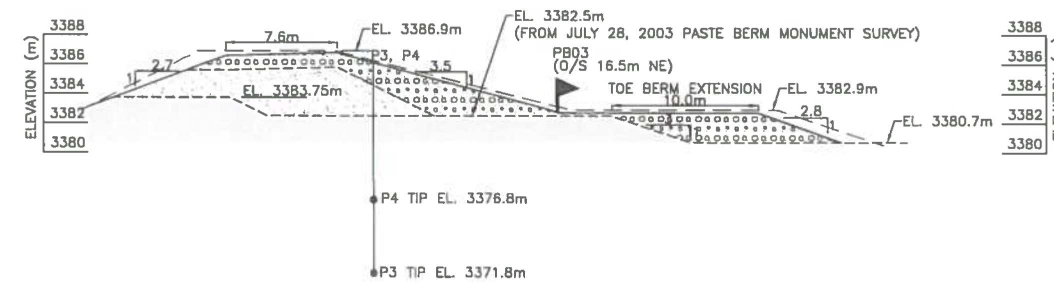
TRUE NORTH

MINE NORTH

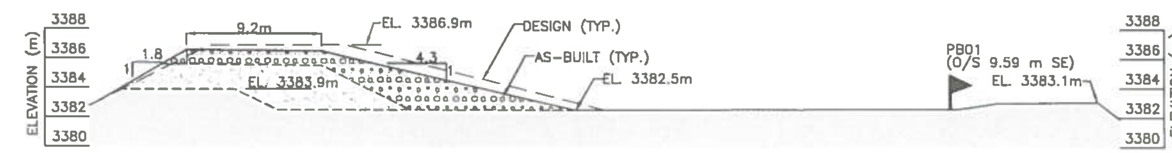
PLAN
SCALE A



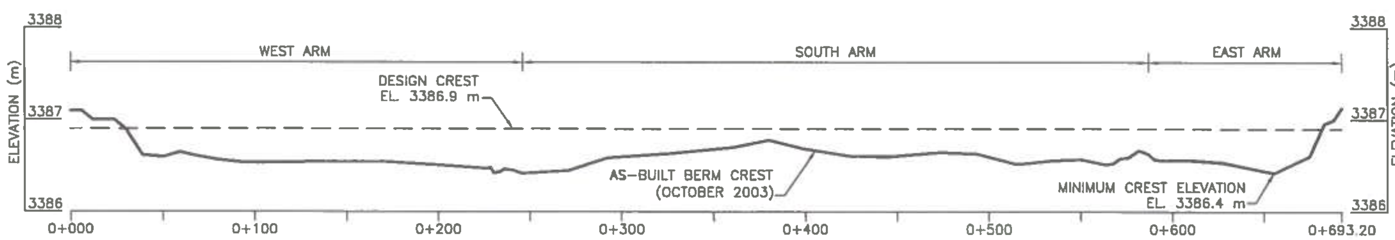
SECTION A
SCALE B



SECTION B
SCALE B



SECTION C
SCALE B



VERTICAL EXAGGERATION
VERTICAL SCALE = 50 X HORIZONTAL SCALE

CREST ELEVATION PROFILE
SCALE A

NOTES:

1. REF. TO QUARMBY SURVEY PROVIDED TO BOLIDEN; DATE RECEIVED OCT. 17, 2003.
2. CREST ELEVATION PROFILE ALONG MID–CREST.

LEGEND:

- P — EXISTING PIEZOMETER (PLAN)
- PB01 — SURVEY MONUMENTS (PLAN)
- P1 — PIEZOMETER
- PB05 — SURVEY MONUMENTS (SECTION)
- P3 — PIEZOMETER (SECTION)
- PAG FILL
- SAND FILL
- PASTE BERM STARTER EMBANKMENT

SCALE A: 0 — 100 m
SCALE B: 0 — 10 m

NO.	DATE	ISSUE / REVISION	DRAWN	CHK'D	DESIGN	APP'D
0	JULY 9, 2004	CONSTRUCTION RECORD – ISSUED WITH FINAL REPORT	AW	RWC	RF	RWC

DRAWING NO. | REFERENCE DRAWING

CLIENT: BOLIDEN WESTMIN (CANADA) LTD.

KLOHN CRIPPEN

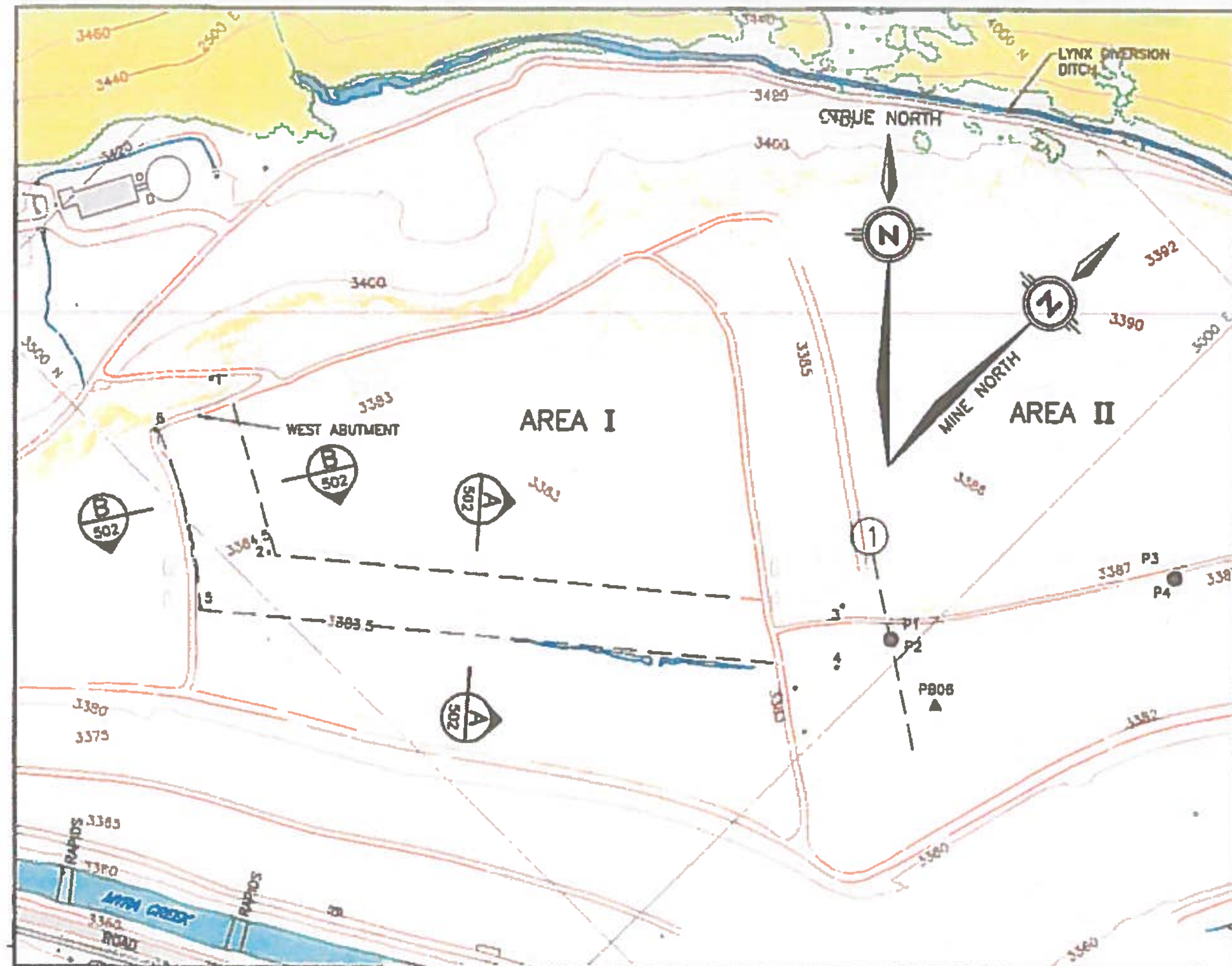
PROJECT: MYRA FALLS OPERATION
2003 PASTE BERM CONSTRUCTION

TITLE: PLAN AND SECTIONS – 2003 PASTE BERM RAISE AS–BUILT

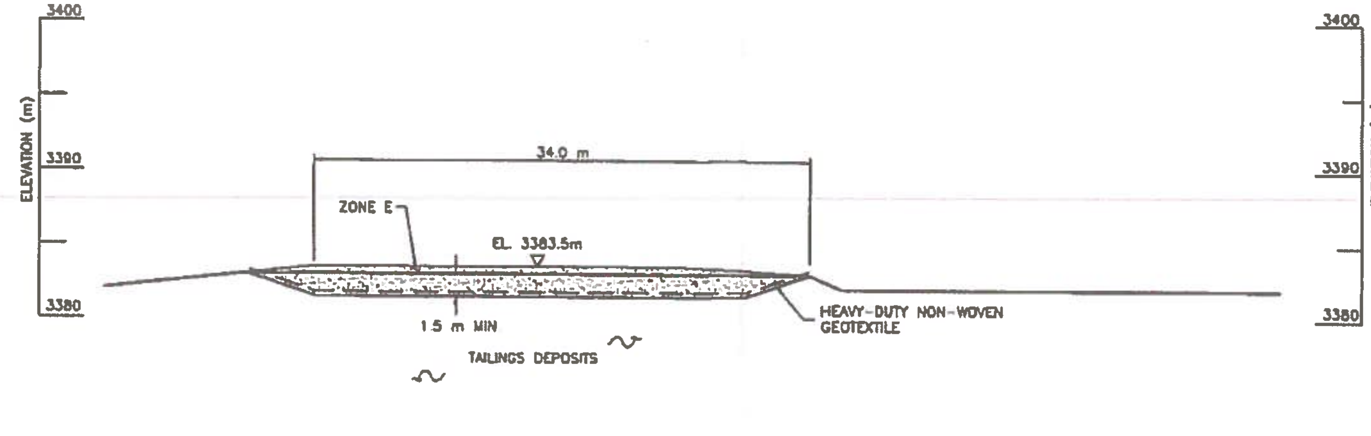
SCALE	PROJECT No.	DWG. No.	REV.
AS SHOWN	M07632 A46	D–46001	0

CANCEL PRINTS BEARING PREVIOUS REVISION

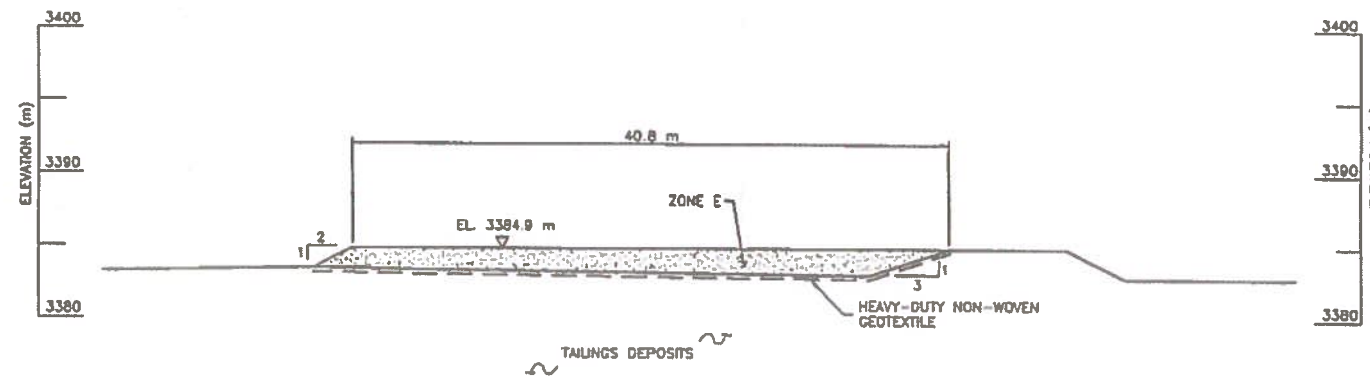
Time: 14:14:7 Date: 7/7/2004 Scale: 1=1(PS) Drawing File: M:\M07632\A46 -2003 Construction Monitoring\400 Design\410 Drawings\2003PasteBermReport\D-46001-0.dwg (awong)



PLAN
SCALE A



SECTION A 502
SCALE B



SECTION B 502
SCALE B

LEGEND:

① — — INSTRUMENTATION LINE

▲PB# SURVEY MONUMENT INSTALLED AUGUST 31, 2004

GENERAL NOTES:

1. FOR GENERAL NOTES DEE DRAWING D-49401
2. AS-BUILT SECTION (ALL ELEVATIONS, DIMENSIONS AND SLOPES SHOWN ARE APPROXIMATE).
3. WEST ABUTMENT CONTACT STRIPPED OF ALL ORGANIC AND LOOSE MATERIALS DOWN TO INTACT NATIVE MATERIALS. 4 m LONG SECTION OF ZONE F FILL PLACED BETWEEN EMBANKMENT FILL AND ABUTMENT.

CONSTRUCTION RECORD

SCALE A: 0 — 100 m
SCALE B: 0 — 10 m

$\frac{.24}{100} = \frac{x}{10\,\ell}$

$x = \frac{10}{100}$ $\frac{.24\,\ell}{10} =$ $.024\,\ell = 24\,m\ell$

DRAWING NO.	REFERENCE DRAWING	NO.	DATE	ISSUE / REVISION	DRAWN	CHK'D	DESIGN	APP'D
		2	MARCH 17, 2005	CONSTRUCTION RECORD	PL			
		1	MAY 28, 2004	ISSUED FOR CONSTRUCTION				
		0	APR. 28, 2004	ISSUED FOR TENDER – CONTRACT No. C04002	AW	RWC	HDP	RWC

PROFESSIONAL ENGINEER — PROVINCE OF BRITISH COLUMBIA — R. W. CHAMBERS

AS A MUTUAL PROTECTION TO OUR CLIENT, THE PUBLIC AND OURSELVES, ALL REPORTS AND DRAWINGS ARE SUBMITTED FOR THE CONFIDENTIAL INFORMATION OF OUR CLIENT FOR A SPECIFIC PROJECT AND AUTHORIZATION FOR USE AND/OR PUBLICATION OF DATA, STATEMENTS, CONCLUSIONS OR ABSTRACTS FROM OR REGARDING OUR REPORTS AND DRAWINGS IS RESERVED PENDING OUR WRITTEN APPROVAL.

CLIENT: NVI MINING LTD.

KLOHN CRIPPEN

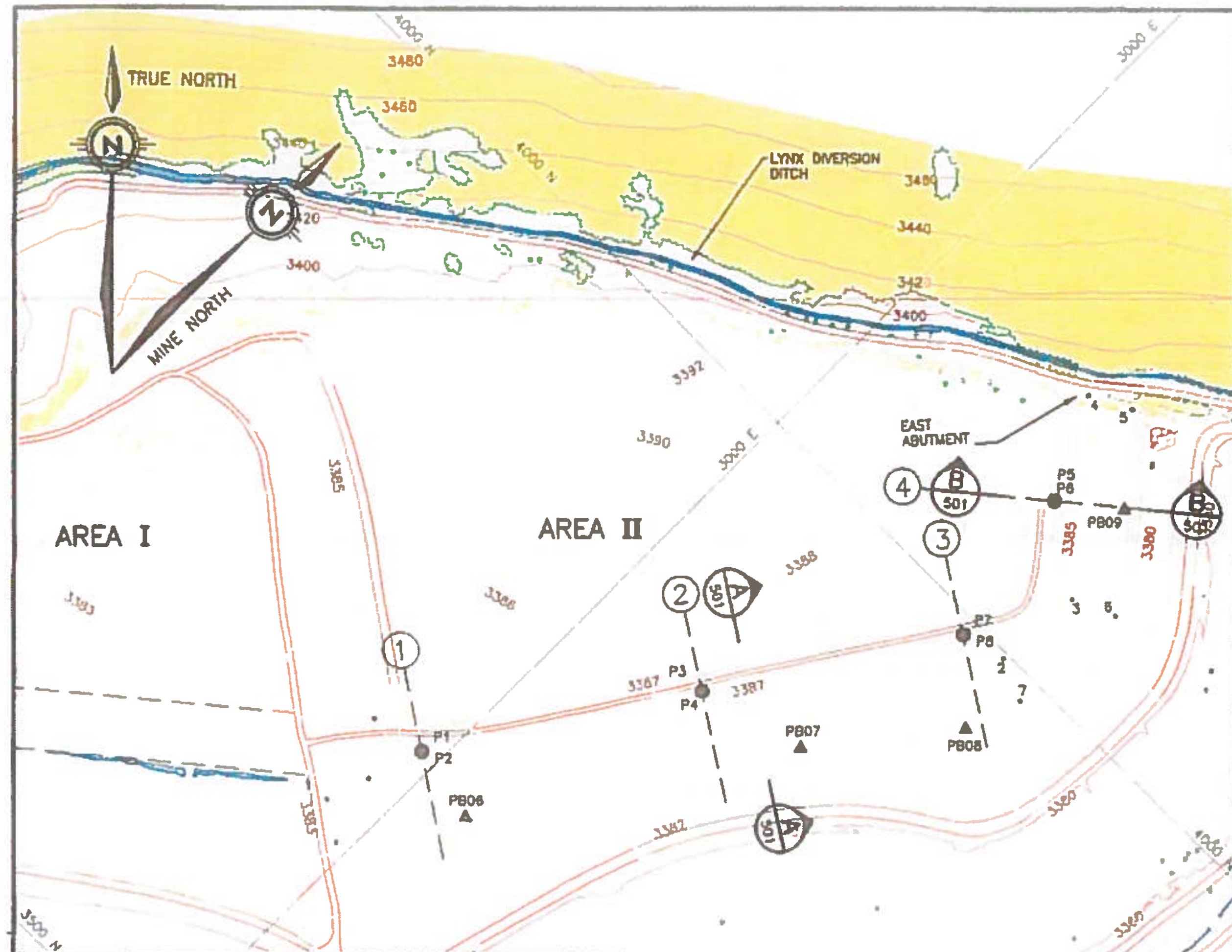
PROJECT: MYRA FALLS OPERATIONS 2004 CONSTRUCTION

TITLE: TAILINGS DISPOSAL FACILITY AREA I INTERNAL PASTE BERM PLAN AND SECTION

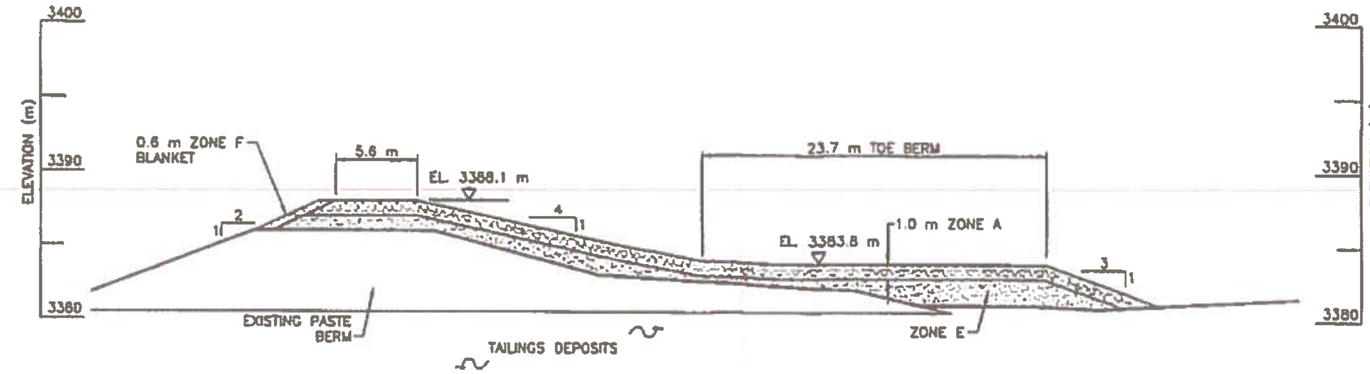
SCALE	PROJECT No.	DWG. No.	REV.
AS SHOWN	M07632 A50	D-49502	2

CANCEL PRINTS BEARING PREVIOUS REVISION

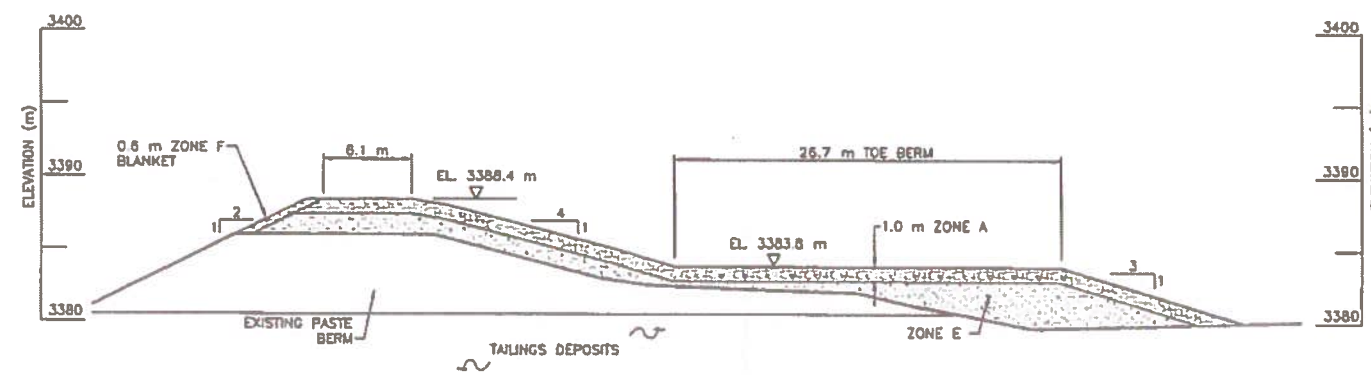
0.5 measured in 2002



PLAN
SCALE A



SECTION A 501
SCALE B



SECTION B 501
SCALE B

LEGEND:

- P1 PIEZOMETER
- ① — INSTRUMENTATION LINE
- ▲PB# SURVEY MONUMENT INSTALLED AUGUST 31, 2004

GENERAL NOTES:

1. FOR GENERAL NOTES DEE DRAWING D-49401.
2. AS-BUILT SECTION (ALL ELEVATIONS, DIMENSIONS AND SLOPES SHOWN ARE APPROXIMATE).
3. EAST ABUTMENT CONTACT STRIPPED OF ALL ORGANIC AND LOOSE MATERIALS DOWN TO INTACT NATIVE MATERIALS. 4 m LONG SECTION OF ZONE F FILL PLACED BETWEEN EMBANKMENT FILL AND ABUTMENT.

CONSTRUCTION RECORD

SCALE A: 0 — 100 m
SCALE B: 0 — 10 m

NO.	DATE	ISSUE / REVISION	DRAWN	CHK'D	DESIGN	APP'D
3	MARCH 17, 2005	CONSTRUCTION RECORD	PL			
2	JAN. 31, 2005	INTERIM RAISE FOR EAST ARM				
1	MAY 28, 2004	ISSUED FOR CONSTRUCTION				
0	APR. 28, 2004	ISSUED FOR TENDER – CONTRACT No. C04002	AW	RWC	HDP	RWC

DRAWING NO. | REFERENCE DRAWING

PROFESSIONAL ENGINEER — PROVINCE OF BRITISH COLUMBIA — R. W. CHAMBERS

AS A MUTUAL PROTECTION TO OUR CLIENT, THE PUBLIC AND OURSELVES, ALL REPORTS AND DRAWINGS ARE SUBMITTED FOR THE CONFIDENTIAL INFORMATION OF OUR CLIENT FOR A SPECIFIC PROJECT AND AUTHORIZATION FOR USE AND/OR PUBLICATION OF DATA, STATEMENTS, CONCLUSIONS OR ABSTRACTS FROM OR REGARDING OUR REPORTS AND DRAWINGS IS RESERVED PENDING OUR WRITTEN APPROVAL.

CLIENT: NVI MINING LTD.

KLOHN CRIPPEN

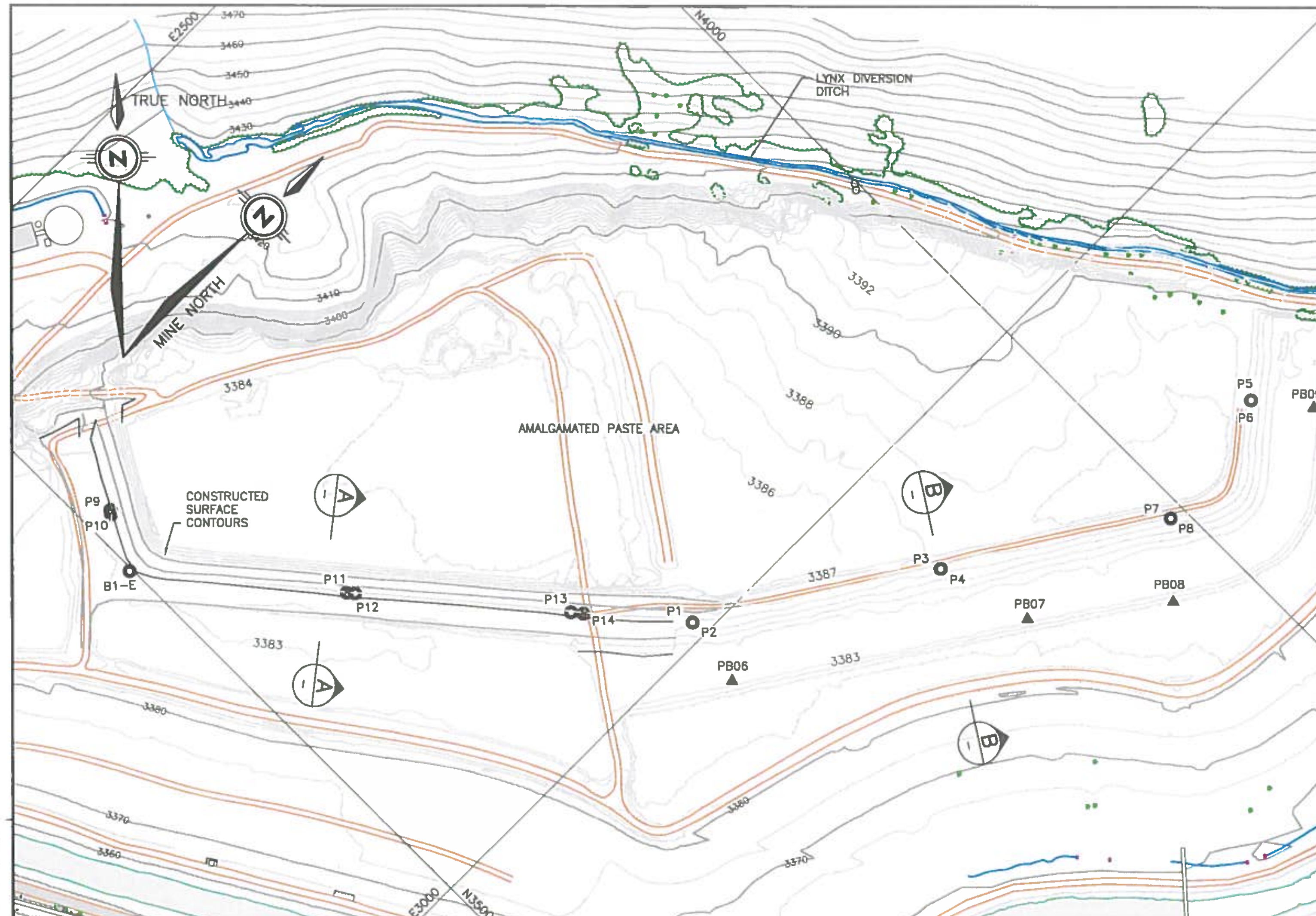
PROJECT: MYRA FALLS OPERATIONS 2004 CONSTRUCTION

TITLE: TAILINGS DISPOSAL FACILITY AREA II INTERNAL PASTE BERM PLAN AND SECTION

SCALE	PROJECT No.	DWG. No.	REV.
AS SHOWN	M07632 A50	D-49501	3

CANCEL PRINTS BEARING PREVIOUS REVISION

Bold $ 130.96 26 AUG.



PLAN
SCALE A

CONSTRUCTION SPECIFICATIONS NOTES:

ABUTMENT PREPARATION

A. WEST ABUTMENT
STRIP 0.6 m OF THE WASTE ROCK FILL SURFACE TO REMOVE LOOSE ROCKS AND EXPOSE WELL-GRADED WASTE ROCK. IF REQUIRED, MANUALLY FILL AND COMPACT ANY OPEN WORK ROCK FILL OR OTHER VOIDS PRIOR TO COVERING THE AREA WITH MACHINE PLACED AND COMPACTED FILL.

B. SAND AND GRAVEL CONTACT AT ABUTMENTS
A 4 m WIDE SECTION OF FULL-FACE SAND AND GRAVEL FILL AT EACH ABUTMENT CONTACT SHALL MEET THE GRADATION SPECIFICATIONS IN TABLE 2. PLACE THE SAND AND GRAVEL IN MAXIMUM 0.3 m LOOSE LIFTS AND COMPACT USING FOUR OVERLAPPING PASSES OF THE LARGEST DOZER CAPABLE OF BEING SUPPORTED BY THE UNDERLYING TAILINGS OR PREVIOUSLY PLACED FILL.

EMBANKMENT FILL

C. PAG WASTE ROCK AND CLEAN FILL (INITIAL LIFT)
USE WELL-GRADED ROCK FROM THE DESIGNATED SOURCES. PRIOR TO THE PLACEMENT OF ANY MATERIAL, COVER THE TAILINGS SURFACES WITH OWNER-SUPPLIED HEAVY-DUTY NON-WOVEN GEOTEXTILE. BASED ON THE 2001 CONSTRUCTION WORK IN AREA II, IT IS EXPECTED THAT MOST AREAS WILL REQUIRE 2 TO 4 LAYERS OF GEOTEXTILE (AS DIRECTED BY THE OWNER) AND AN INITIAL LIFT THICKNESS OF 1.1 m TO 1.4 m (TO ACHIEVE 1.0m NET). THE TAILINGS ARE VERY SOFT. THE GENERATION OF TAILINGS WAVES AHEAD OF THE WORK FRONT SHALL BE MINIMIZED BY LIMITING THE SIZE OF THE SPREADING EQUIPMENT (D6 OR D7 DOZERS MAY BE THE MAXIMUM SIZE PERMITTED), USING EXCAVATORS TO SIDECAST MATERIAL OVER THE GEOTEXTILE BEFORE FINISH GRADING WITH DOZERS, AND MINIMIZING THE AMOUNT OF MATERIAL STOCKPILED NEAR THE LEADING EDGE AHEAD OF THE SPREADING EQUIPMENT.

D. PAG WASTE ROCK AND CLEAN FILL (SUBSEQUENT LIFTS)
USE MATERIALS FROM DESIGNATED SOURCES. PLACE THE MATERIAL IN MAXIMUM 0.60 m LOOSE LIFTS AND COMPACT USING FOUR OVERLAPPING PASSES OF A D8 TRACTOR-DOZER OR LARGER. EACH LIFT SHALL BE PLACED OVER THE ENTIRE FOOTPRINT BEFORE STARTING ON THE NEXT LIFT, AND EACH LIFT SHALL BE ADVANCED OVER THE FULL WIDTH OF THE BERM.

E. SAND FILL (SUBSEQUENT LIFTS)
USE SAND OBTAINED FROM THE SAND STORAGE AREA AT THE WEST END OF THE TAILINGS DISPOSAL FACILITY. PRIOR TO THE PLACEMENT OF NEW SAND FILL IN ANY AREA, SCARIFY AND RECOMPACT THE EXISTING SURFACE OF THE PAG WASTE ROCK BERM AND REMOVE LOOSE STONES OR BOULDERS. PLACE ALL SAND FILL IN MAXIMUM 0.3 m LOOSE LIFTS AND COMPACT BY A MINIMUM OF FOUR PASSES BY A 10-TONNE VIBRATORY COMPACTOR. OVERWET MATERIAL SHALL NOT BE USED. CONDUCT FILL PLACEMENT AND OTHER ACTIVITIES IN THE SUBSIDENCE AREA OF THE SOUTH ARM IN ACCORDANCE WITH INSTRUCTIONS FROM THE OWNER WHO WILL MONITOR PERFORMANCE OF THE BERM IN THAT AREA DURING CONSTRUCTION OPERATIONS.

F. SAND FILL, IF AVAILABLE AND AT THE OWNER'S DISCRETION, CAN BE USED IN LIEU OF PAG WASTE ROCK (SUBSEQUENT LIFTS). SUCH SAND FILL SHALL BE PLACED IN ACCORDANCE WITH ITEM E.

G. THE CONTRACTOR SHALL BE PREPARED TO STOP WORK ALTOGETHER OR RE-ALLOCATE ITS EQUIPMENT FLEET TO OTHER WORK AREAS OF THE BERM IF DIRECTED BY THE OWNER.

H. UPSTREAM SAND AND GRAVEL BLANKET
THE SAND AND GRAVEL BLANKET SHALL MEET THE GRADATION SPECIFICATIONS IN TABLE 2. PLACE THE MATERIAL ON THE UPSTREAM SIDE OF THE EMBANKMENT RAISES BY END-DUMPING WITH NO COMPACTION.

I. QUALITY CONTROL
PERFORM ONE GRADATION TEST FOR EACH 10,000 m^3 OF SAND OR PAG MATERIAL PLACED, ONE FOR EACH 500 m^3 OF THE SAND AND GRAVEL BLANKET, AND ONE FOR EACH 500 m^3 OF THE SAND AND GRAVEL ABUTMENT CONTACT (BY OWNER) AT EQUALLY SPACED INTERVALS AS DIRECTED BY THE OWNER.

J. WEATHER RESTRICTIONS
FILL PLACEMENT MAY BE SUSPENDED BY THE OWNER DURING ADVERSE WEATHER CONDITIONS. PRIOR TO SUSPENSION, FILL SURFACE SHALL BE SHAPED TO DRAIN EXCESS WATER. PRIOR TO RESUMPTION OF FILL PLACEMENT, THE FILL SURFACE SHALL BE SCARIFIED AND RECOMPACTED AS DIRECTED BY THE OWNER.

K. INSTRUMENTATION
PROTECT ALL EXISTING PIEZOMETERS AND SURVEY MONUMENTS AND ASSIST THE OWNER AS REQUIRED FOR EXTENSION OF PIEZOMETERS DURING THE COURSE OF THE WORK.

TABLE 2 – GRADATION SPECIFICATION FOR SAND AND GRAVEL FOR ABUTMENT CONTACT AND FOR UPSTREAM BLANKET

PARTICLE SIZE/ SIEVE SIZE	PERCENT PASSING BY WEIGHT
75 mm	90 – 100
38 mm	75 – 100
19 mm	60 – 100
No. 4	40 – 80
No. 20	20 – 55
No. 40	15 – 45
No. 100	10 – 30
No. 200	5 – 20

TABLE 3 – PIEZOMETER DETAILS

PIEZOMETER	TIP ELEVATION
P9	3378.9m
P10	3374.4m
P11	3380.1m
P12	3374.0m
P13	3379.7m
P14	3374.2m

TABLE 4 – SURVEY POINT LOCATIONS

SURVEY POINT	NORTH (m)	EAST (m)	ELEVATION
PB06	3712.919	3027.002	3384.527
PB07	3874.281	3133.501	3384.918
PB08	3961.929	3190.043	3384.677

LEGEND:

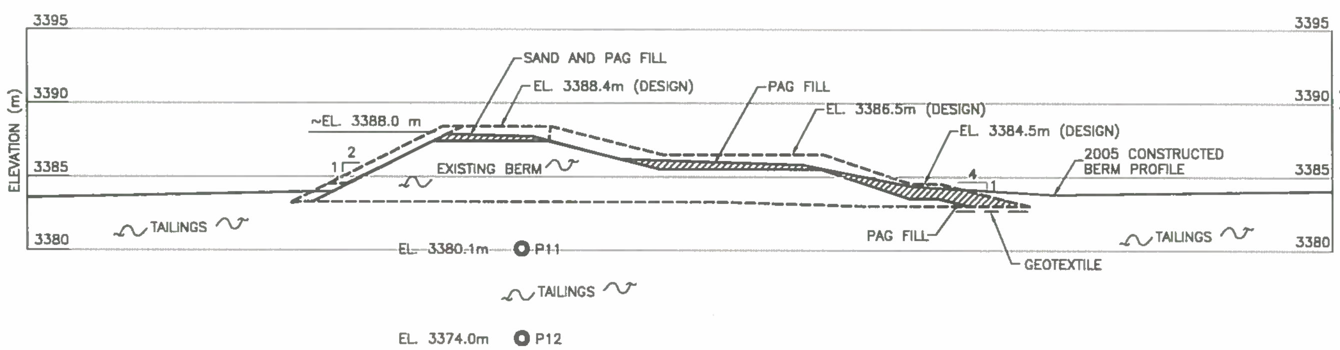
FILL PLACED DURING 2005

PAG POTENTIAL ACID GENERATING

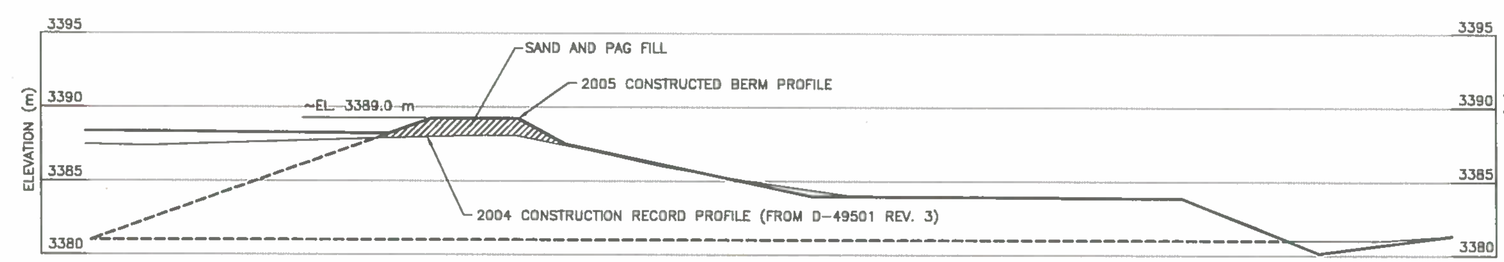
P9 PIEZOMETER

NOTES:

1. ALL ELEVATIONS AND DIMENSIONS ARE IN METRES.
2. DETAILS OF THE PASTE BERM DESIGN CAN BE FOUND IN THE JULY 2001 KLOHN CRIPPEN REPORT TITLED "PASTE TAILINGS IN EXISTING FACILITY, DETAIL ENGINEERING, FINAL REPORT."
3. PASTE BERM CONSTRUCTION WAS DIRECTED BY THE OWNER.
4. 2005 AS-BUILT PROFILES ARE BASED ON SURVEY PROVIDED BY MFO, DATED APRIL 2006.



SECTION A TYPICAL AREA I BERM SECTION
SCALE B



SECTION B TYPICAL AREA II BERM SECTION
SCALE B

SCALE A: 0 100 m

SCALE B: 0 10 m

CONSTRUCTION RECORD

NO.	DATE	ISSUE / REVISION	DRAWN	CHK'D	DESIGN	APP'D
3	DEC. 22, 2006	2005 CONSTRUCTION RECORD				
2	JUNE 2, 2005	REVISED BERM SECTION				
1	MARCH 1, 2005	APPROVED FOR CONSTRUCTION				

DRAWING NO. | REFERENCE DRAWING

CLIENT: NVI MINING LTD.

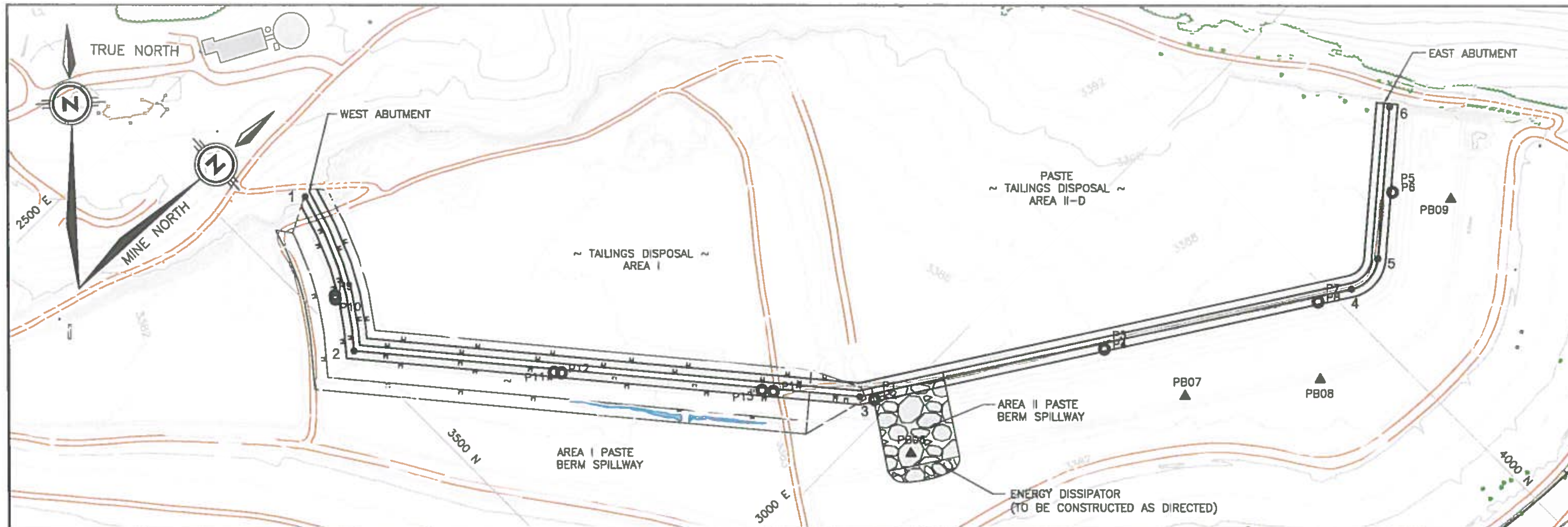
KLOHN CRIPPEN

PROJECT: MYRA FALLS OPERATIONS
2005 CONSTRUCTION

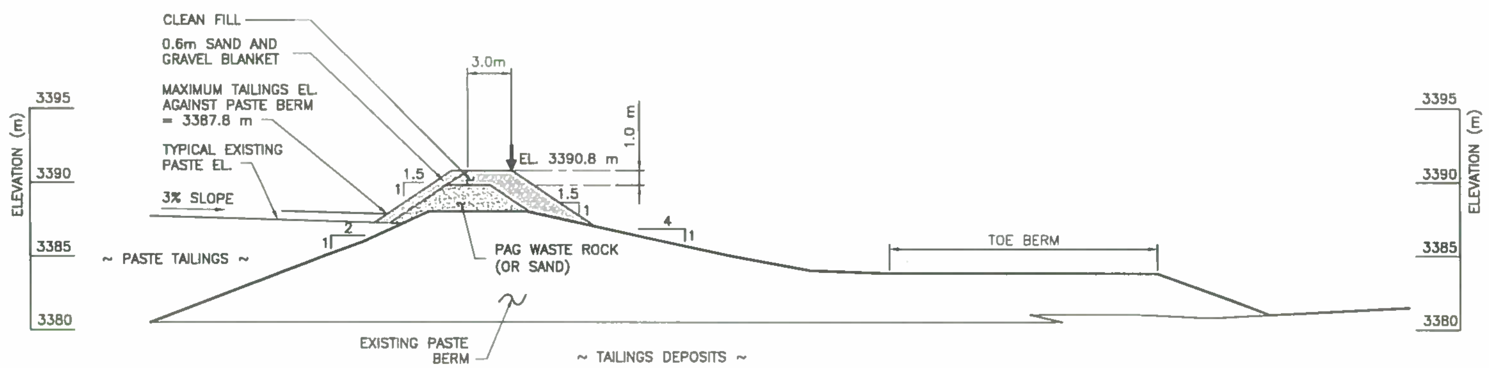
TITLE: TAILINGS DISPOSAL FACILITY
AREA I PASTE BERM
PLAN, SECTION AND SPECIFICATIONS

SCALE	PROJECT No.	DWG. No.	REV.
AS SHOWN	M07632 A51	D-51502	3

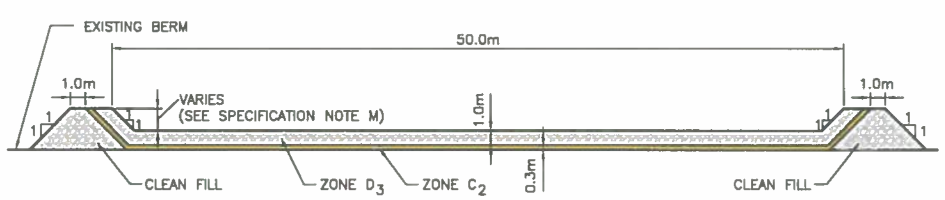
CANCEL PRINTS BEARING PREVIOUS REVISION



PLAN
SCALE A



TYPICAL RAISE SECTION
SCALE: B



TYPICAL SPILLWAY SECTION
SCALE: B

CONSTRUCTION SPECIFICATIONS NOTES:

ABUTMENT PREPARATION

A. WEST ABUTMENT
STRIP 0.6 m OF THE WASTE ROCK FILL SURFACE TO REMOVE LOOSE ROCKS AND EXPOSE WELL–GRADED WASTE ROCK. IF REQUIRED, MANUALLY FILL AND COMPACT ANY OPEN WORK ROCK FILL OR OTHER VOIDS PRIOR TO COVERING THE AREA WITH MACHINE PLACED AND COMPACTED FILL.

B. EAST ABUTMENT
STRIP GROUND SURFACE OF ALL ORGANICS AND LOOSE MATERIALS DOWN TO THE INTACT NATIVE MATERIALS.

C. SAND AND GRAVEL CONTACT AT ABUTMENTS
A 4 m LONG SECTION OF FULL–FACE SAND AND GRAVEL FILL AT EACH ABUTMENT CONTACT SHALL MEET THE GRADATION SPECIFICATIONS IN TABLE 2. PLACE THE SAND AND GRAVEL IN MAXIMUM 0.3 m LOOSE LIFTS AND COMPACT USING FOUR OVERLAPPING PASSES OF THE LARGEST DOZER CAPABLE OF BEING SUPPORTED BY THE UNDERLYING TAILINGS OR PREVIOUSLY PLACED FILL.

EMBANKMENT FILL

D. PAG WASTE ROCK
USE PAG WASTE ROCK FROM DESIGNATED SOURCES. PLACE THE MATERIAL IN MAXIMUM 0.60 m LOOSE LIFTS AND COMPACT USING FOUR OVERLAPPING PASSES OF A D8 TRACTOR–DOZER OR LARGER. EACH LIFT SHALL BE PLACED OVER THE ENTIRE FOOTPRINT BEFORE STARTING ON THE NEXT LIFT, AND EACH LIFT SHALL BE ADVANCED OVER THE FULL WIDTH OF THE BERM.

E. SAND FILL
USE SAND OBTAINED FROM THE SAND STORAGE AREA AT THE WEST END OF THE TAILINGS DISPOSAL FACILITY. PRIOR TO THE PLACEMENT OF NEW SAND FILL IN ANY AREA, SCARIFY AND RECOMPACT THE EXISTING SURFACE OF THE PAG WASTE ROCK BERM AND REMOVE LOOSE STONES OR BOULDERS. PLACE ALL SAND FILL IN MAXIMUM 0.3 m LOOSE LIFTS AND COMPACT BY A MINIMUM OF FOUR PASSES BY A 10–TONNE VIBRATORY COMPACTOR. OVERWET MATERIAL SHALL NOT BE USED. CONDUCT FILL PLACEMENT AND OTHER ACTIVITIES IN THE SUBSIDENCE AREA OF THE SOUTH ARM IN ACCORDANCE WITH INSTRUCTIONS FROM THE OWNER WHO WILL MONITOR PERFORMANCE OF THE BERM IN THAT AREA DURING CONSTRUCTION OPERATIONS.

F. SAND FILL, IF AVAILABLE AND AT THE OWNER'S DISCRETION, CAN BE USED IN LIEU OF PAG WASTE ROCK. SUCH SAND FILL SHALL BE PLACED IN ACCORDANCE WITH ITEM E.

G. THE CONTRACTOR SHALL BE PREPARED TO STOP WORK ALTOGETHER OR RE–ALLOCATE ITS EQUIPMENT FLEET TO OTHER WORK AREAS OF THE BERM IF DIRECTED BY THE OWNER.

H. UPSTREAM SAND AND GRAVEL BLANKET
THE SAND AND GRAVEL BLANKET SHALL MEET THE GRADATION SPECIFICATIONS IN TABLE 2. PLACE THE MATERIAL ON THE UPSTREAM SIDE OF THE EMBANKMENT RAISES BY END–DUMPING WITH NO COMPACTION.

I. QUALITY CONTROL
PERFORM ONE GRADATION TEST FOR EACH 10,000 m^3 OF SAND OR PAG MATERIAL PLACED, ONE FOR EACH 500 m^3 OF THE SAND AND GRAVEL BLANKET, AND ONE FOR EACH 500 m^3 OF THE SAND AND GRAVEL ABUTMENT CONTACT (BY OWNER) AT EQUALLY SPACED INTERVALS AS DIRECTED BY THE OWNER.

J. WEATHER RESTRICTIONS
FILL PLACEMENT MAY BE SUSPENDED BY THE OWNER DURING ADVERSE WEATHER CONDITIONS. PRIOR TO SUSPENSION, FILL SURFACE SHALL BE SHAPED TO DRAIN EXCESS WATER. PRIOR TO RESUMPTION OF FILL PLACEMENT, THE FILL SURFACE SHALL BE SCARIFIED AND RECOMPACTED AS DIRECTED BY THE OWNER.

K. INSTRUMENTATION
PROTECT ALL EXISTING PIEZOMETERS AND SURVEY MONUMENTS AND ASSIST THE OWNER AS REQUIRED FOR INSTALLATION AND EXTENSION OF PIEZOMETERS DURING THE COURSE OF THE WORK.

SPILLWAY

L. SPILLWAY INVERT TO BE SET AT EL. 3389.9m

M. HEIGHT OF CONTAINMENT BERMS ABOVE RIPRAP LEVEL EQUAL TO 1 m ALONG BERM SLOPES AND 2 m ALONG TOE BERM.

TABLE 1 - WORK POINTS

	NORTHING (m)	EASTING (m)
1	3550.8	2617.1
2	3499.7	2714.6
3	3722.3	2981.0
4	4012.5	3166.9
5	4039.8	3165.0
6	4118.9	3097.7

TABLE 2 - GRADATION SPECIFICATION FOR SAND AND GRAVEL FOR ABUTMENT CONTACT AND FOR UPSTREAM BLANKET

PARTICLE SIZE/ SIEVE SIZE	PERCENT PASSING BY WEIGHT
75 mm	90 – 100
38 mm	75 – 100
19 mm	60 – 100
No. 4	40 – 80
No. 20	20 – 55
No. 40	15 – 45
No. 100	10 – 30
No. 200	5 – 20

GENERAL NOTES:

1. ALL ELEVATIONS AND DIMENSIONS ARE IN METRES. COORDINATES AND ELEVATIONS ARE REFERENCED TO MINE DATUM. (AMSL + 3048m)
2. DETAILS OF THE PASTE BERM DESIGN CAN BE FOUND IN THE JULY 2001 KLOHN CRIPPEN REPORT TITLED "PASTE TAILINGS IN EXISTING FACILITY, DETAIL ENGINEERING, FINAL REPORT."
3. FOUR EXISTING SURVEY MONUMENTS INSTALLED ALONG AREA II TOE BERM, AUGUST 31, 2004.

LEGEND:

- ^{1} WORK POINTS
- SCOPE OF WORK COVERED BY THIS DRAWING
- WORK POINT LOCATIONS
- SURVEY MONUMENT LOCATION (NOTE 3)
- PAG — POTENTIAL ACID GENERATING
- P10 — EXISTING PIEZOMETER
- PB07 — PASTE BERM SURVEY MONUMENT

SCALE A: 0 — 100 m
SCALE B: 0 — 10 m

APPROVED FOR CONSTRUCTION

CLIENT: NVI MINING LTD.

KLOHN CRIPPEN

PROJECT: MYRA FALLS OPERATIONS 2005 CONSTRUCTION

TITLE: TAILINGS DISPOSAL FACILITY AREAS I & II FLOOD ROUTING PLAN, SECTION AND SPECIFICATIONS

SCALE	PROJECT No.	DWG. No.	REV.
AS SHOWN	M07632 A51	D-51501	0

CANCEL PRINTS BEARING PREVIOUS REVISION

NO.	DATE	ISSUE / REVISION	DRAWN	CHK'D	DESIGN	APP'D
0	JUNE 2, 2005	APPROVED FOR CONSTRUCTION				

DRAWING NO.	REFERENCE DRAWING

Time: 15:29:28
Date: 6/2/2005
Scale: 1=10(PS)
Drawing File: M:\M07632\A51 - 2005 Construction Services\400 Design\410 Drawings\09 - 2005 Construction\D-51501_rev0.dwg (LHutton)
Xrefs: BM-MyraFalls-Mar05

APPENDIX B

Acceleration, Velocity and Displacement Time Histories

APPENDIX B
Acceleration, Velocity and Displacement Time Histories

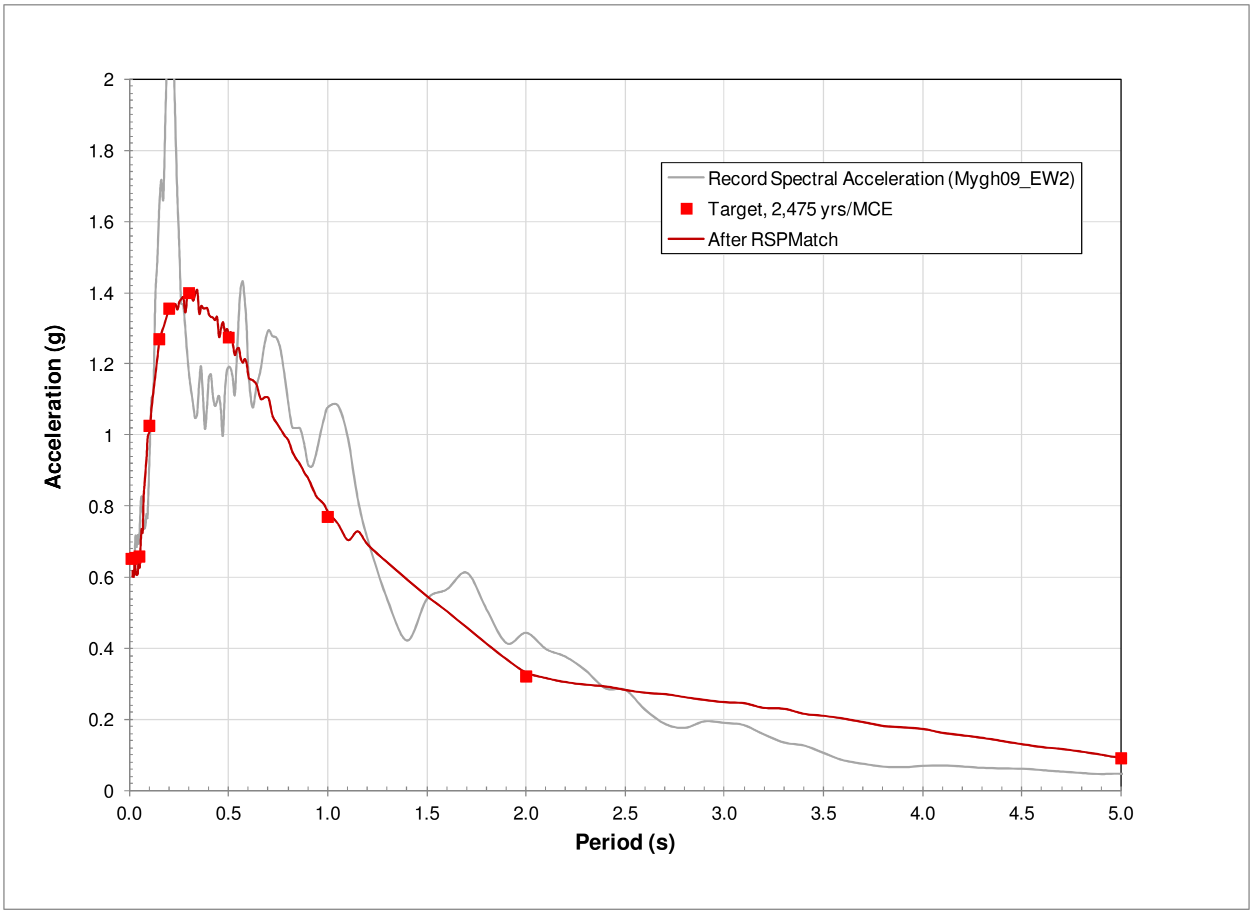


Figure B1: Record 1 MYGH09 EW2 Spectrum before and after Spectral Matching Compared to Target Spectrum for Passive – Closure Case

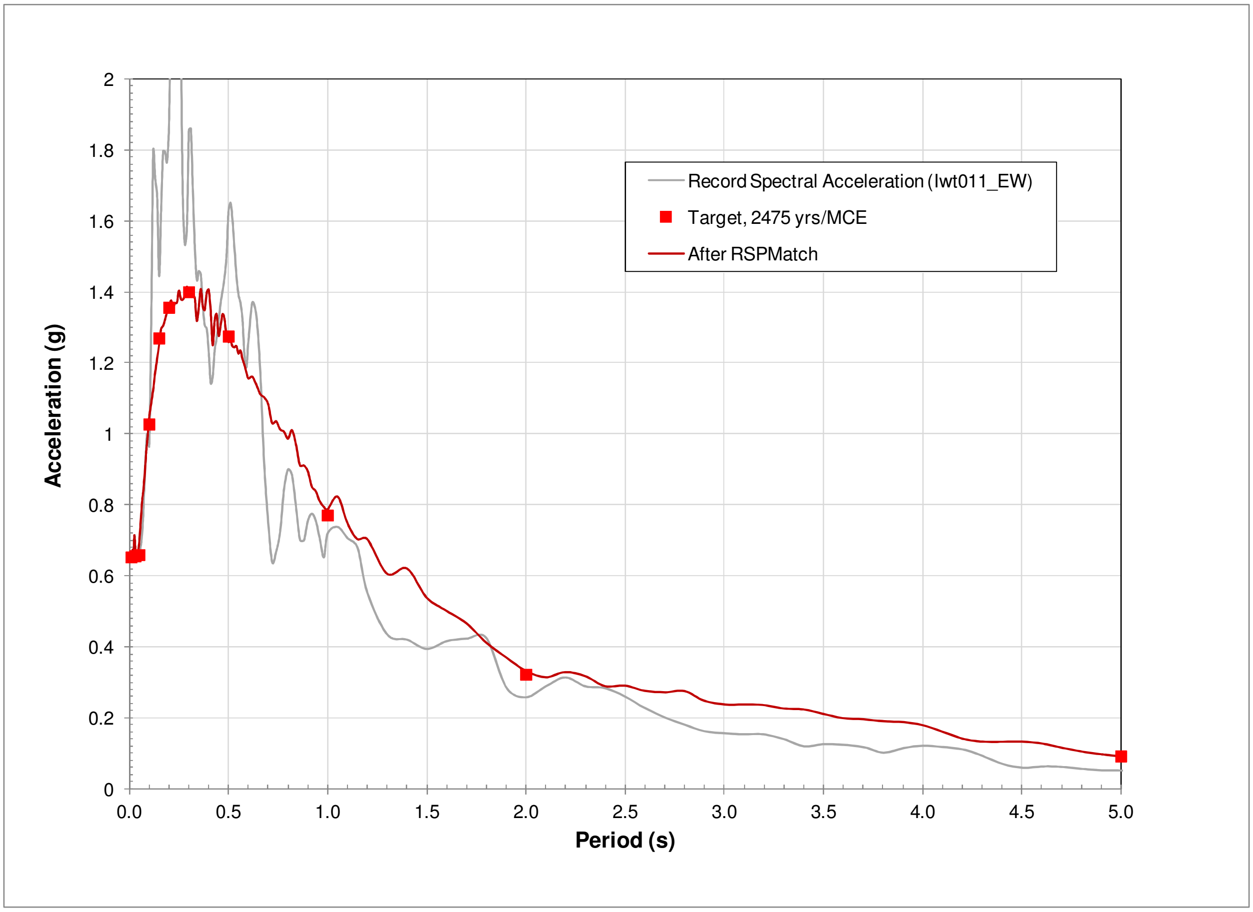


Figure B2: Record 2 IWT011 EW Spectrum before and after Spectral Matching Compared to Target Spectrum for Passive – Closure Case

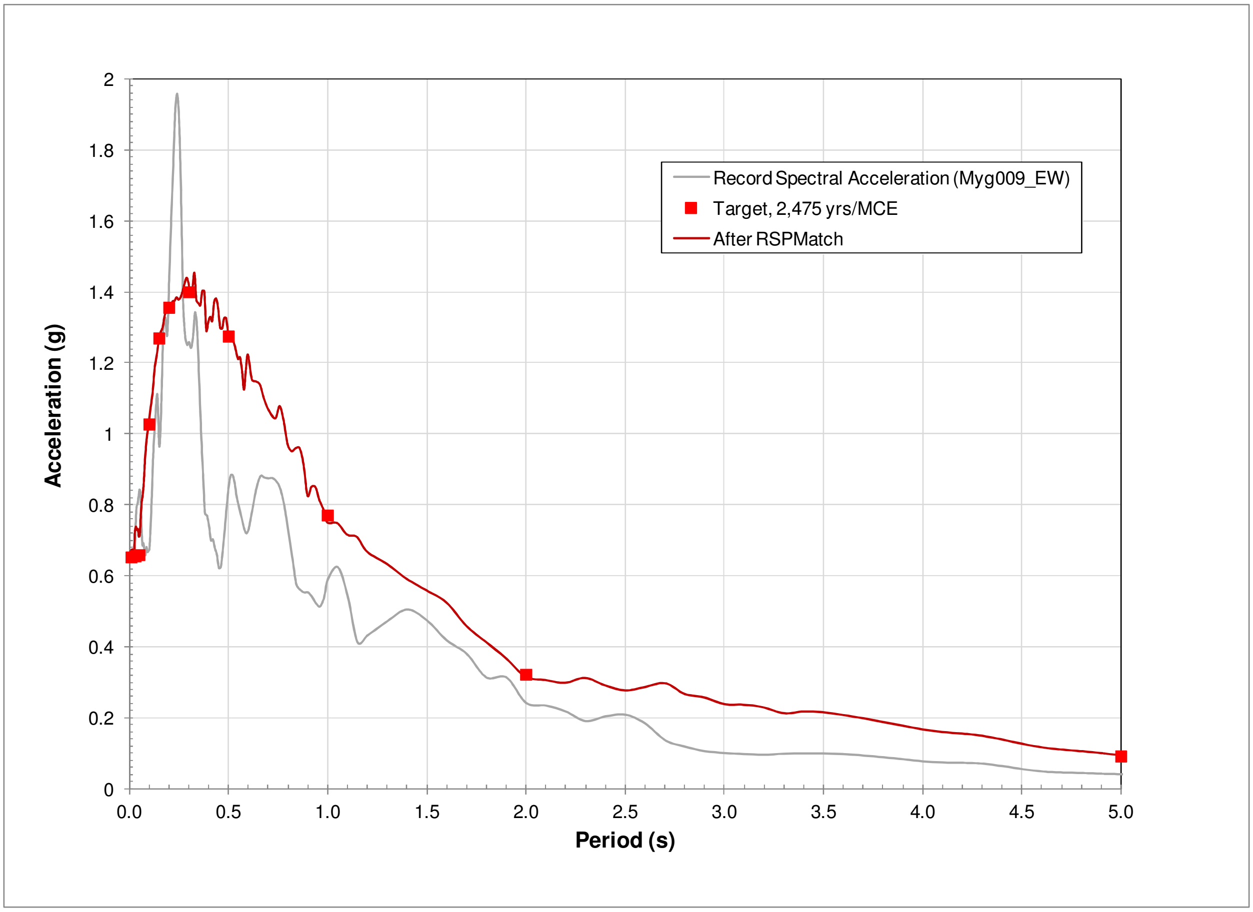


Figure B3: Record 3 MYG009 EW Spectrum before and after Spectral Matching Compared to Target Spectrum for Passive – Closure Case

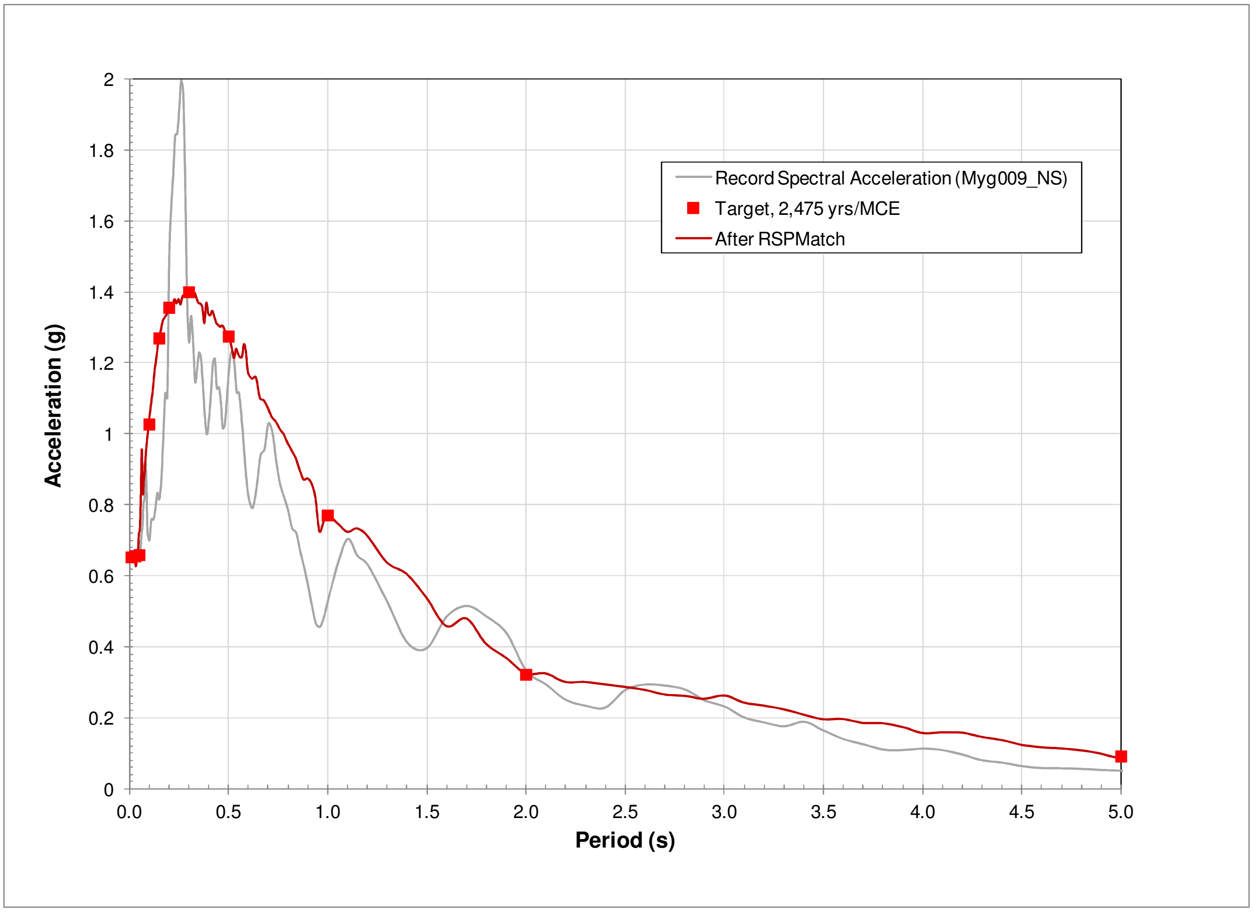


Figure B4: Record 4 MYG009 NS Spectrum before and after Spectral Matching Compared to Target Spectrum for Passive – Closure Case

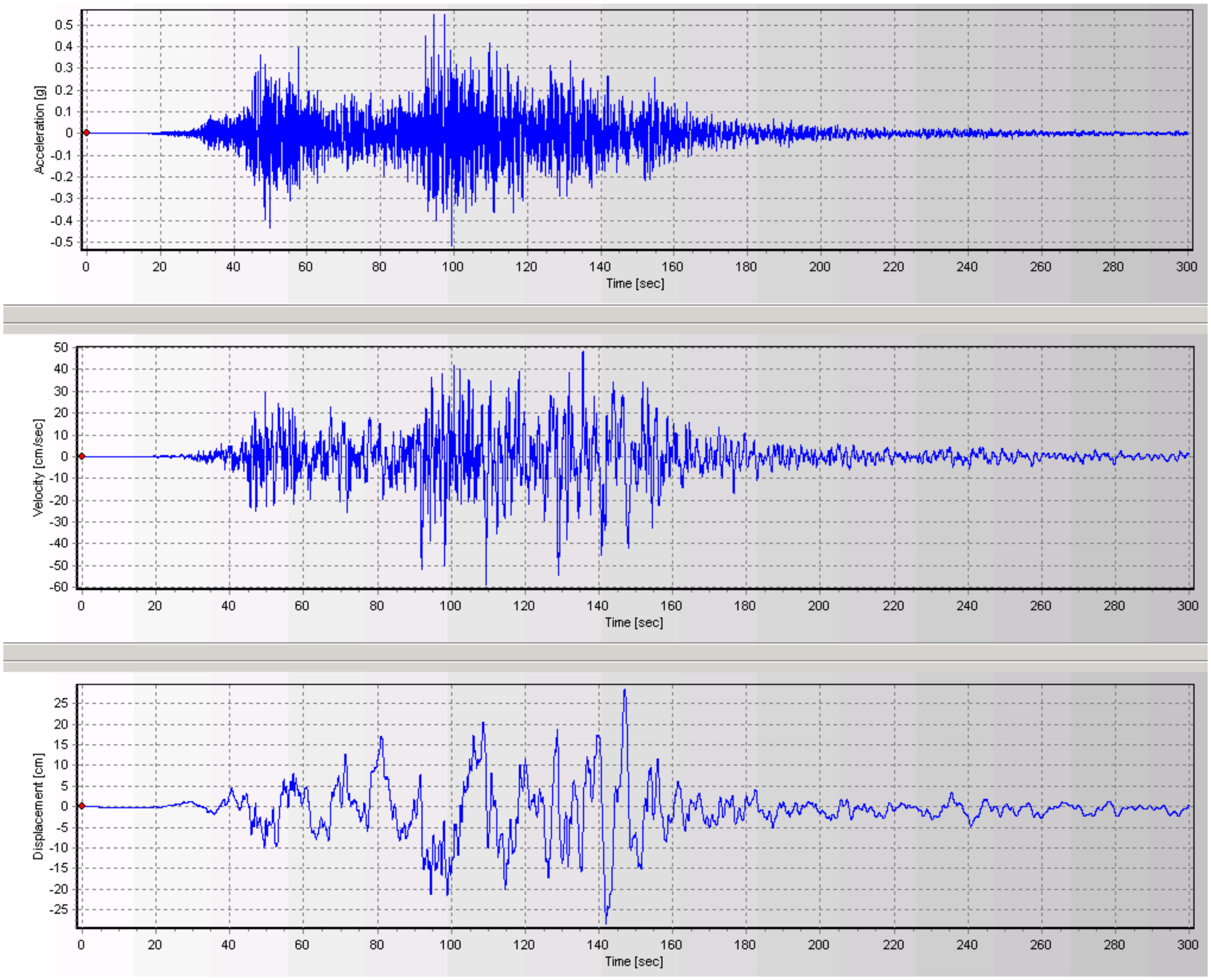


Figure B5: Record 1 MYGH09 EW2 Horizontal Acceleration, Velocity and Displacement Time Histories after Spectral Matching

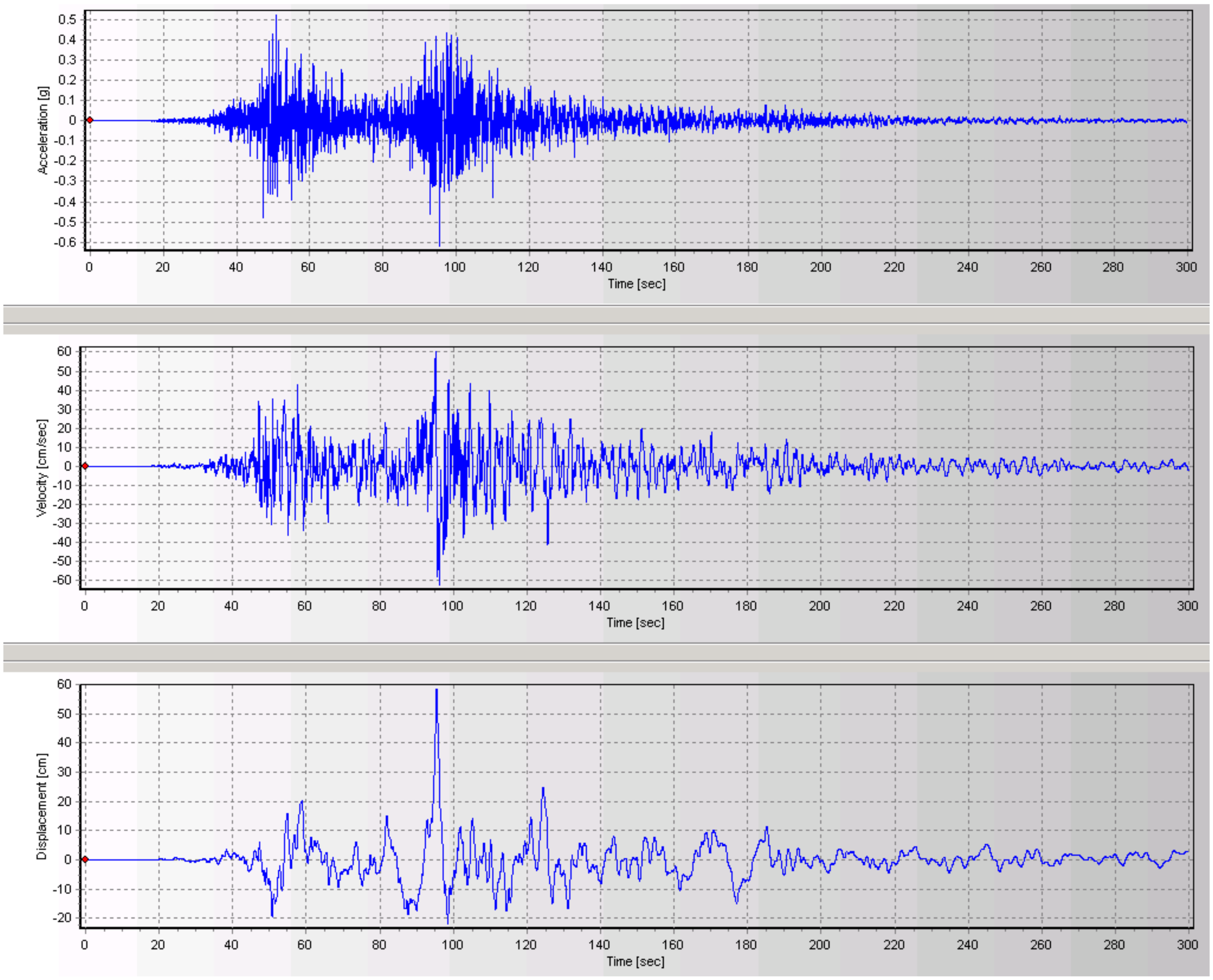


Figure B6: Record 2 IWT011 EW Horizontal Acceleration, Velocity and Displacement Time Histories after Spectral Matching

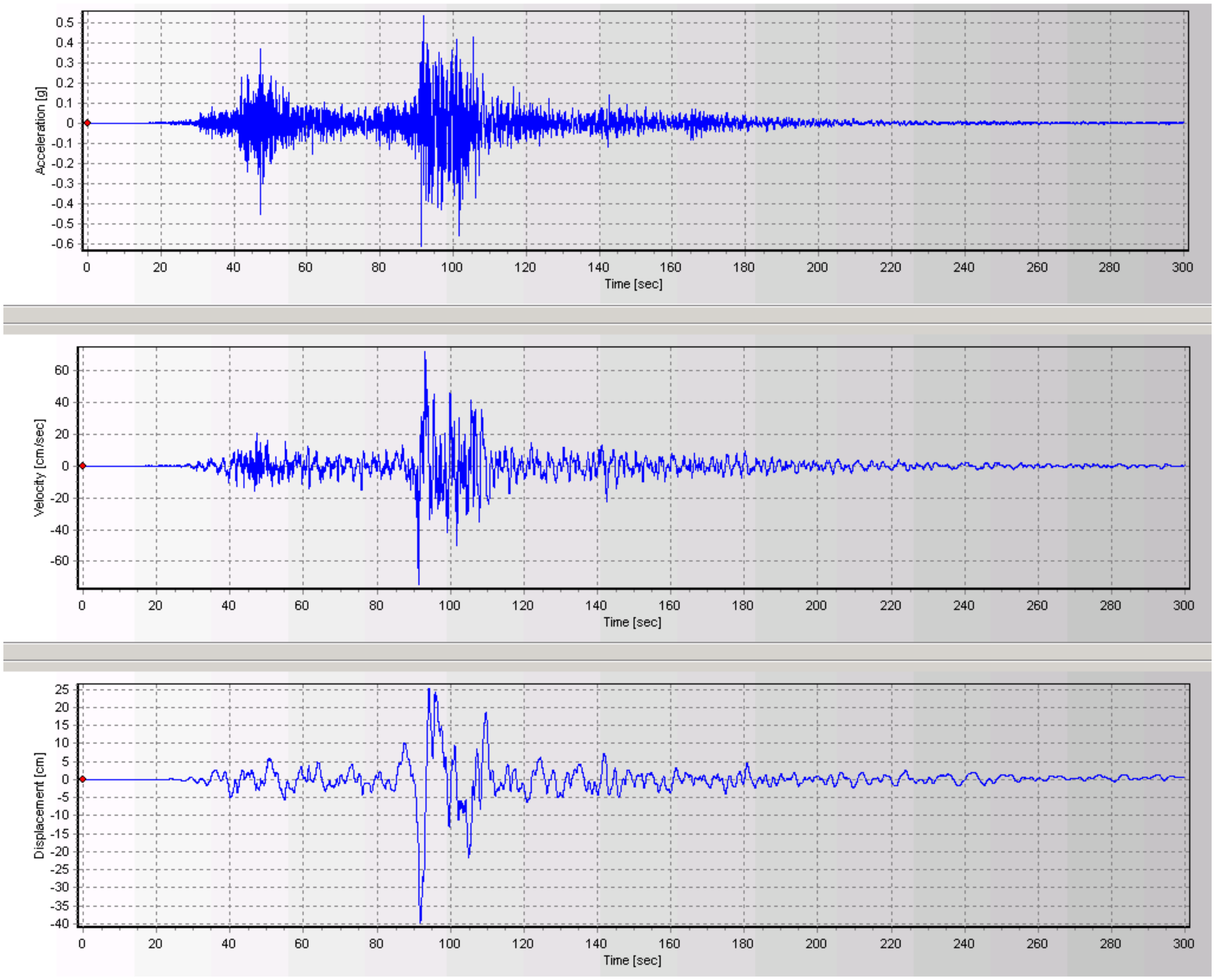


Figure B7: Record 3 MYG009 EW Horizontal Acceleration, Velocity and Displacement Time Histories after Spectral Matching

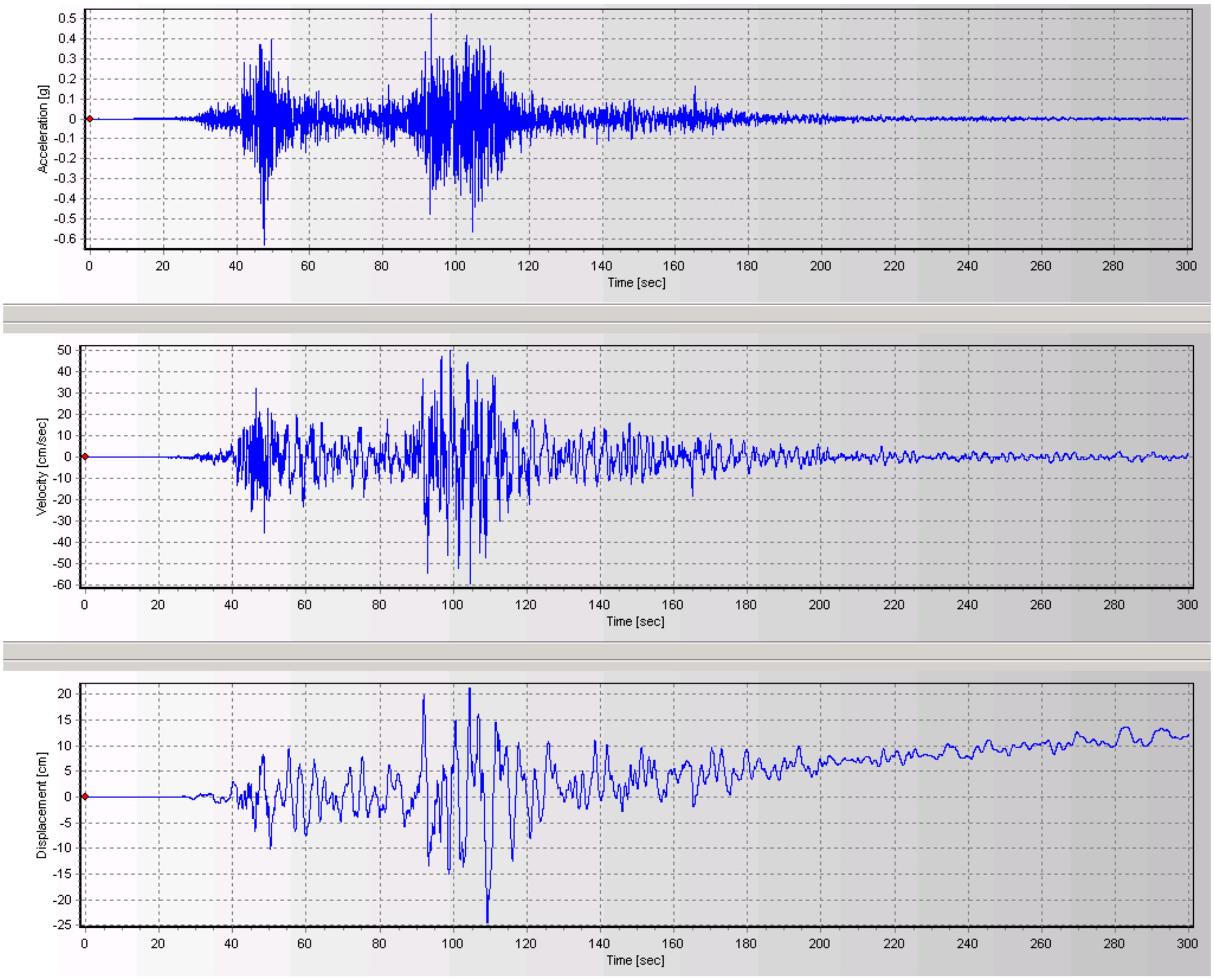


Figure B8: Record 4 MYG009 NS Horizontal Acceleration, Velocity and Displacement Time Histories after Spectral Matching

APPENDIX C

Nyrstar Myra Falls Old TDF Foundation Materials Properties[1]

[1] Amec Foster Wheeler, 2016. "Myra Falls Old Tailings Disposal Facility Glaciolacustrine Evaluation Report - Appendix A", GLU Evaluation Report, March 2016.

Memo

To:	File		
From:	Dixie Ann Simon (AmecFW)		
Tel:	1 (250) 847-8783	**File No:**	NX14001F.4
Date:	31 March 2016	**Cc:**	Ed McRoberts (AmecFW)

Subject: Old TDF Foundation Soils Geotechnical Material Properties

The purpose of this memorandum is to summarize the geotechnical material properties used in the historical and current Old TDF stability analyses. The generalized stratigraphy of the foundation soils is described briefly and a corresponding table of the material properties is presented. The same unit designations used in previous descriptions of the stratigraphy have been adopted for consistency. Background information and detailed descriptions of the field programs, the laboratory data, the stratigraphy and the process used to select the material properties are provided in the attachments to this memorandum and in the glaciolacustrine data report[1]. Geotechnical material properties for the tailings are provided under separate cover.

Brief descriptions of the foundation soils are as follows:

Unit 2b: Colluvial silty sand and gravel.

Unit 2a: Glacial fluvial soils consisting of sand and gravel with cobbles and boulders.

Unit 3: Silty sand considered to be a 'transition' zone between the glaciolacustrine soils and the overlying glaciofluvial soils.

Unit 4: Highly laminated fine sand and nonplastic to low plasticity silt and clay of lacustrine origin.

The geotechnical material properties for the Old TDF foundation soils selected for use in the assessment of the impact of the presence of the glaciolacustrine soils on the stability of the Old TDF are provided in Table 1.

Table 1: Geotechnical Strength Parameters for Limit Equilibrium Slope Stability Analyses

Soil Unit	Soil Description	Static Parameters		Post-seismic Parameters	
		γ (kN/m^3)	ϕ, S_u	γ (kN/m^3)	ϕ, S_r
2a	Dense Sand and Gravel with Loose Layers	24	34 ° to 36°	24	34°
2b	Medium Dense Silty Sand and Gravel - Colluvium	22	34°	22	0.28 σ'_{v0}
3	Unit 3 (Silty Sand)	22	32°	22	0.35 σ'_{v0}
4	Unit 4 (Glaciolacustrine)	20	0.22 σ'_{v0}	20	0.13 σ'_{v0}
	Bedrock	--	--	--	--

In the data presentations and plots, depth or depth below original ground surface has generally been used instead of elevation because of the general slope of the Myra Creek topography down the thalweg. In some design applications, such as consideration of the contact between the granular foundations soils (Units 2a, 2b, and 3) and the glaciolacustrine soils (Unit 4), elevation is considered more appropriate because of the nature of deposition of the soils in the glacial lake rather than along and down a high energy stream.

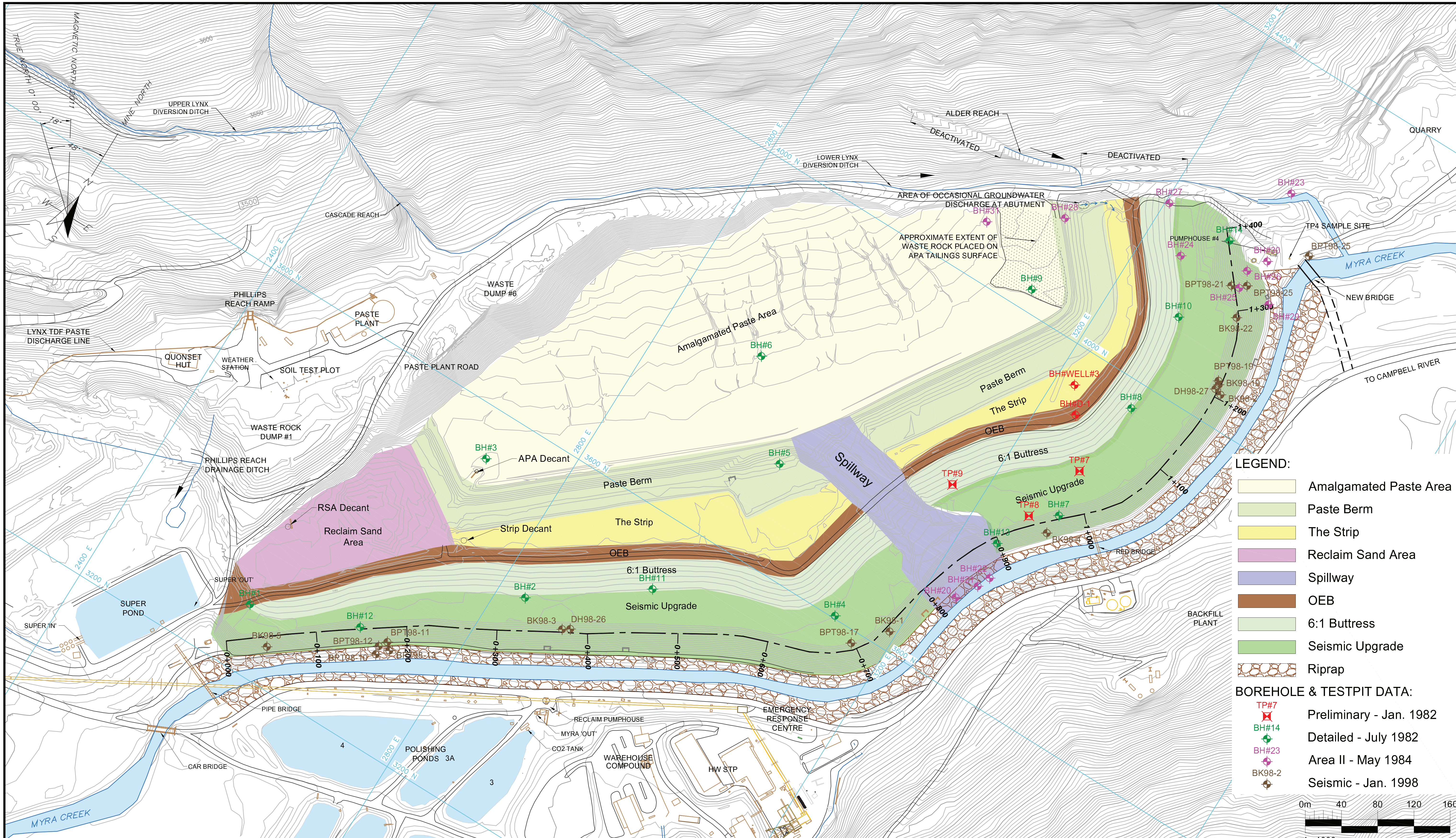
In some of the historical reports made available to Amec Foster Wheeler, Unit 4 was not included in stratigraphic summaries. This appears to be the result of assumption that the static and post-seismic strength of the glaciolacustrine soils was higher than the overlying granular materials. This assumption is not supported by the geological history of the site or by the re-evaluation of the historical field and laboratory data.

The ongoing analysis could result in the conclusion that the granular foundation materials, Units 2a and 3, are dilatant when sheared instead of contractant. If this is the case, the post-seismic parameters provided in Table 1 will be revised. Use of these parameters in the assessment of the impact of the glaciolacustrine soils on the stability of the Old TDF is considered prudent.

A. SUBSURFACE EXPLORATION PROGRAMS

Numerous subsurface exploration programs have been conducted at the site of the Old TDF. The subsurface exploration programs included borehole drilling, sampling and laboratory testing. Programs were completed by Knight Piesold Limited, Klohn Crippen Consultants Limited (KCL) and Amec Foster Wheeler. Each of the subsurface exploration programs are described briefly in sections 1.1 through 1.3. The borehole locations with the exception of the 2003 KCL boreholes are shown in Figure A-1. Most of the boreholes are relatively shallow with only a few extending through the glaciolacustrine materials and into the underlying till and/or bedrock.F

[1] Nyrstar Myra Falls Old Tailings Disposal Facility Glaciolacustrine Investigation Data Report, Amec Foster Wheeler, NX14001F, February 2016



NOTES:
1. THIS DRAWING SHOULD BE READ IN CONJUNCTION WITH THE AMEC ENVIRONMENT & INFRASTRUCTURE REPORT No. NX14001-2014-11 "2014-Q3 DAM SAFETY INSPECTION REPORT" DATED NOVEMBER 2014.
2. SURVEY SURFACE TOPOGRAPHY SUPPLIED BY CLIENT
3. THIS SURVEY IS REFERENCED TO MINE DATUM (MASL + 3048 m).

REV	D	M	Y	ISSUE / REVISION DESCRIPTION	ENG.	APPR.
G						
F						
E						
D						
C						
B						
A						

Client: nyrstar

AMEC Environment & Infrastructure
4385 Boban Drive, Nanaimo, B.C. V9T 5V9
Tel. 250 758-1887 Fax 250 758-1899

amec

DATUM: MINE
PROJECTION: MINE
DRAWN BY: J. Ford
REVIEWED BY: D.A.SIMON
ORIGINAL SCALE: 1:4000

PROJECT: MYRA FALLS TAILINGS DISPOSAL FACILITIES GLACIOLACUSTRINE EVALUATION

TITLE: OLD TDF BOEHOLE LOCATION PLAN

PROJECT NO.: NX14001F.4
REVISION NO.:
DATE: MARCH 2016
DRAWING NO.: A-1
SHEET NO.: 1 of 1

A.1 Knight Piesold Limited

A preliminary subsurface exploration program was conducted in 1981 by Knight Piesold Limited (KPL). The subsurface exploration consisted of five shallow test pits, ten shallow hydrogeological boreholes with piezometers, two deep hydrogeological boreholes advanced using a cable tool system and three pumping test wells. The preliminary subsurface exploration program report was not available for review. Amec Foster Wheeler understands that a copy of the report could not be found in NMF's files.

A detailed subsurface exploration program was conducted in 1982 also by KPL. The report also was not available for review; however, copies of the borehole logs and laboratory testing were included in a subsequent report issued by KPL. The detailed subsurface exploration program consisted of 14 boreholes advanced using the mud rotary method of drilling. Penetration testing was conducted at 1.5 m to 3.0 m intervals. Thin-walled (Shelby) tube samples of the materials encountered in the boreholes also were obtained. Laboratory testing consisted of the following:

- 10 unit weight measurements
- 63 water content measurements
- 6 specific gravity measurements
- 7 index properties (Atterberg limits)
- 7 laboratory vane shear tests (4 with residual strength)
- 48 grain size analyses(7 with hydrometer)
- 2 consolidated undrained multi-stage Triaxial shear tests with pore pressure measurements on glaciolacustrine materials.

KPL conducted a third subsurface exploration program (12 boreholes) in 1984 to further explore areas where bedrock was encountered at relatively shallow depths in the detailed subsurface exploration program. The report was not available for review.

A final three-phase subsurface exploration program was completed in 1995 to further explore the nature and extent of sandy tailings present near the existing pipe bridge. The first phase consisted of cone penetration testing (CPT) at seven locations, seismic cone penetration testing (SCPT) at two locations and two boreholes advanced using hollow stem augers. Standard Penetration Tests (SPT) were conducted in the boreholes. Five thin-walled tube samples were obtained of the tailings encountered in the boreholes. Seven piezometers of various types also were installed.

The second phase consisted of CPTs at 14 locations, SCPT at one location, two boreholes advanced using hollow stem augers and installation of 12 piezometers. Eight thin-walled tube samples were obtained from the tailings encountered in the boreholes. One thin-walled tube sample was obtained of the near surface materials. Laboratory testing on the tailings samples consisted of the following:

- 12 cyclic, constant volume, simple shear tests
- 10 bulk density measurements
- 11 grain size distribution (three with hydrometer)
- 3 specific gravity measurements
- 12 water content measurements

A.2 Klohn Crippen Consultants Limited

Klohn Crippen Consultants Limited (KCL) conducted two concurrent subsurface exploration programs in 1998 in support of the design of the Old TDF seismic upgrade and preliminary design of the foundation densification program. The subsurface exploration programs consisted of closed-ended and open-ended Becker penetration testing (BPT) at 15 locations, closed-ended only BPT at 10 locations, and two boreholes advanced using the mud rotary method of drilling. Downhole shear wave velocity measurements were made in casings installed in two of the BPT boreholes. SPT was conducted in the mud rotary boreholes at 1.5 m to 3.0 m intervals in the mud rotary boreholes. Laboratory testing consisted of the following:

- 14 index properties (Atterberg limits)
- 76 grain size analyses (3 with hydrometer)
- 57 water content measurements

A second subsurface exploration program was conducted in 2003 in support of the design of the densification program. At that time, closed-ended and open-ended BPT was conducted at 15 locations and closed-ended BPT at 5 locations. Large diameter penetration testing was conducted at selected intervals in six of the open-ended BPT boreholes. Eighteen piezometers and/or monitoring wells were installed in 12 of the open-ended BPT boreholes. Laboratory testing consisted of the following:

- 27 water content measurements
- 10 index properties (Atterberg limits)
- 28 grain size analyses (6 with hydrometers)

A.3 Amec Foster Wheeler

Amec Foster Wheeler conducted a subsurface exploration program in 2015, targeting the glaciolacustrine (GL) materials. The program consisted of ten boreholes advanced using the sonic method of drilling, four boreholes advanced using the mud rotary method of drilling, CPT in the GL at four locations and SCPT in the GL at two locations. SPT was conducted at 1.5 m or 10 m intervals in the mud rotary boreholes. Downhole shear and compression wave velocities were measured in casings installed in three of the sonic boreholes. Grab samples were obtained from the sonic core. Thin-walled tube samples were obtained of the GL materials and also of the tailings. Field and laboratory testing consisted of the following:

- 10 vane shear tests
- 31 index properties (Atterberg limits) on GLU materials
- 52 water content measurements
- 9 grain size analyses (5 with hydrometer)
- 10 static, constant volume, direct simple shear tests
- 11 cyclic, constant volume, direct simple shear tests
- 6 post-cyclic static, constant volume, direct simple shear tests
- 6 specific gravity measurements

B. DESCRIPTIONS OF OLD TDF FOUNDATION STRATIGRAPHY

The stratigraphy beneath the Old TDF has been historically has been divided in to four units. The upper three units consist of glaciofluvial and colluvial deposits of primarily sand and gravel. The lowest unit consists of glaciolacustrine silt and clay. General descriptions are provided in the following subsections. Detailed descriptions can be found in the borehole logs provided in the Glaciolacustrine Investigation Data Report.

B.1 Granular Foundation Units

Unit 2b – Colluvium (Silty Sand and Gravel)

The colluvial soils consisted of gravel with some sand and trace silt and clay to gravelly sand with some to trace cobble, silt and clay. The colluvial soils were generally encountered within the same depth range as the glaciofluvial gravels and sands but in thinner and possibly discontinuous layers. Particles of colluvial material were found to be a little more angular as well, ranging from subangular to angular in particle shape. Color was generally brown to grey with none of the bluish color observed in the other gravels and sands. The density of the colluvial gravels and sands was also quite variable across site ranging from loose to very dense. Colluvial gravel and sand layers, when encountered, ranged in thickness from less than 1m to 7.5m.

Unit 2a – Glaciofluvial Sand and Gravel

Unit 2a consisted of sandy gravel to sand with some gravel and trace fines. Relatively thin zones of silt also were encountered in some of the boreholes; however, a continuous layer could not be identified. Boulders were also encountered based on drilling action. Particles were generally subrounded to subangular. Density was quite variable over this unit, generally ranging from compact to very dense; however, looser zones were also observed. The glaciofluvial gravel and sands were most commonly a bluish grey but grey and brown colors also were noted. Observed moisture conditions ranged between wet and saturated. Thickness of these gravels and sands was also quite variable, ranging from 17m near the east end of the TDF to 45m near the west end of the TDF.

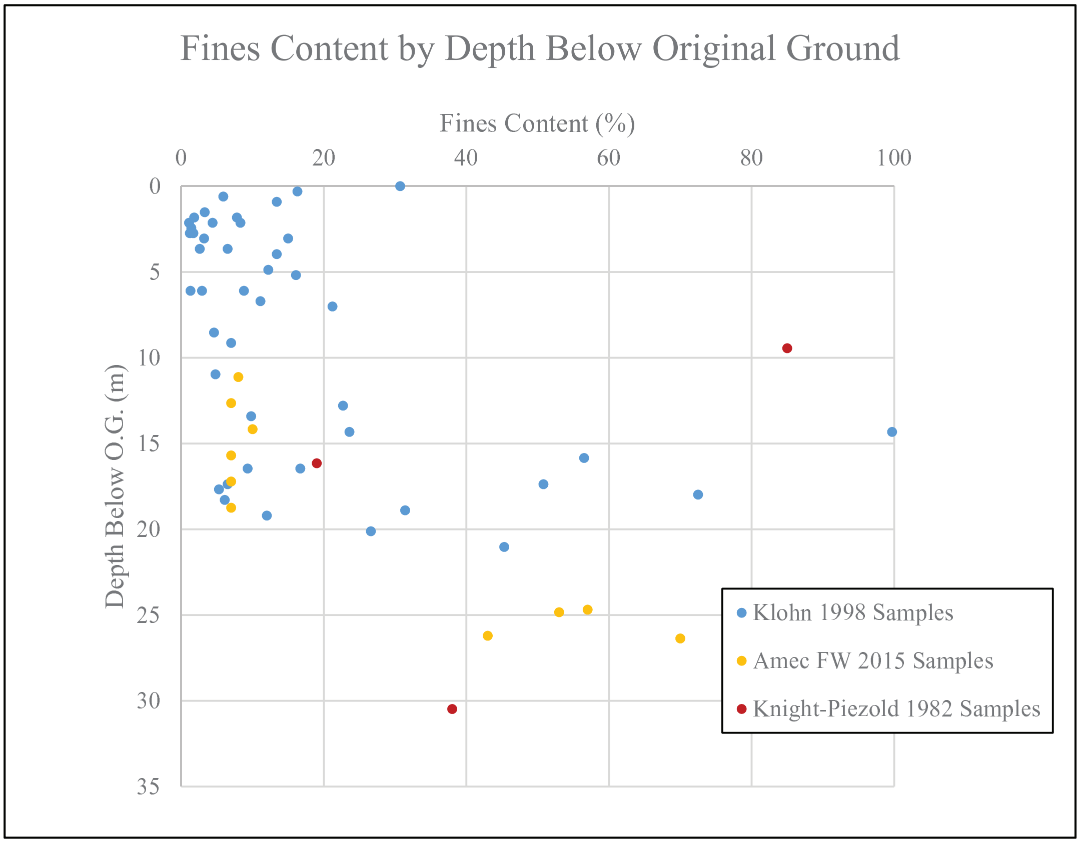
Unit 3 – Sand/Interbedded Sand and Silt

Unit 3 consisted of interbedded sand and silt. Fine to medium sand with some silt was interbedded with low plasticity silt with trace clay. The interbedded layers generally ranged in thickness between a few centimeters and about 30 cm.

The unit appears to be a transitional unit between the glaciofluvial sand and gravel and the underlying glaciolacustrine unit (Unit 4). The thickness of the transitional unit ranges between one meter and eight meters; however, unit descriptions contained in reports of previous subsurface exploration programs indicated thickness up to 10 m. This could be the result of the method of drilling. Amec Foster Wheeler’s boreholes were drilled using the sonic method of drilling which provides a continuous core. Open-ended BPT with SPT only provides representative samples at discrete intervals.

Testing completed on samples of Units 2b, 2a and 3 soils consists primarily of grain size analyses and index properties. The variation of percentage of particles smaller than 0.075 mm (fines content) with depth for the three granular foundation units is shown graphically in Figure B-1. The fines content for Unit 2b (colluvium) was generally greater than about 15%. For Unit 3, fines content was generally greater than 35%. The fines content of Unit 2a was generally between 5% and 10%; however, isolated zones of higher fines content soils were encountered within the unit.

Figure B-1: Variation in Fines Content with Depth below Original Ground



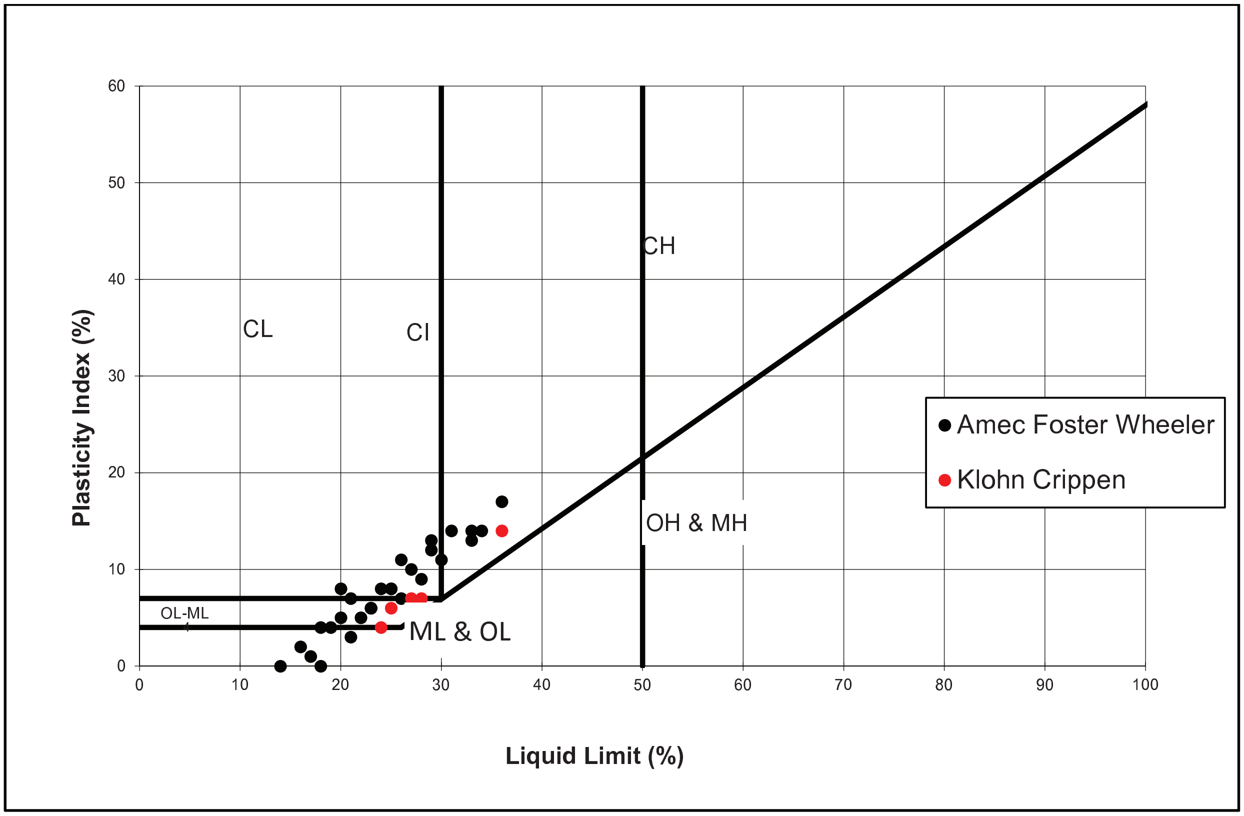
B.2 Fine Grained Foundation Unit

Unit 4 – Glaciolacustrine Silt and Clay

Unit 4 consisted of highly laminated, nonplastic to low plasticity, clayey silt or clay with trace to some silt. Laminations consisted of fine sand and were observed to be generally less than about two millimetres in thickness. Trace organics were observed throughout the glaciolacustrine unit. Water contents ranged between 3% and 36% but were generally in the range of 20% to 30% with an average of 23%.

Amec Foster Wheeler determined the index properties (Atterberg limits) of 31 samples of the glaciolacustrine soils. Liquid limits ranged between 9% and 36% with an average of 24%. Plastic limits ranged between 12% and 20% with an average of 17%. The corresponding plasticity indices ranged between 0% and 16% with an average of 8%. The glaciolacustrine soils generally classified as ML-CL, CL and CI when classified in accordance with the Modified Unified Soil Classification System as shown in Figure B-2.

Figure B-2: Modified Unified Soil Classification – Glaciolacustrine Soils



KCL determined the index properties of five samples of the glaciolacustrine soils. The results also are provided in Figure B-2. KPL determined the index properties of five samples of the glaciolacustrine soils; however, the results cannot be shown in the figure because only the plasticity index was provided in the historical reports available to Amec Foster Wheeler. The plasticity index ranged between four and 15.

Amec Foster Wheeler conducted grain size analyses on seven samples of the glaciolacustrine soils. KCL conducted grain size analyses on five samples of the glaciolacustrine soils; KPL on six samples. The percent of particles by weight smaller than 0.075mm ranged between 80% and 100%. The average was about 95%.

KPL conducted multi-stage, consolidated undrained Triaxial compression tests with pore pressure measurements on two samples of the glaciolacustrine soils. The undrained shear strength ratios, s_u/σ'_{v0} were 0.44 and 0.45. The calculated angles of internal friction, ϕ', were 33° and 37°.

Amec Foster Wheeler conducted static, cyclic and post cyclic direct simple shear tests under constant volume conditions on samples of the glaciolacustrine materials. Gamma ray scans of the tubes were uses to select samples for testing; however, the samples were found to be highly laminated. On extrusion it was found that some laminations were at significant angles from vertical making sample selection extremely difficult. The results are provided in Table B-1.

Table B-1: Summary of Index Properties

Borehole Number	Sample ID	Depth m	Atterberg Limits LL %	PL %	PI %	USC	Specific Gravity	Grain Size Distribution % Sand	% Silt	% Clay	Laboratory Shear Wave Velocity m/s	Downhole Shear Wave Velocity m/s
BH15-32	S3	60.1	29	15	14	CL	--	0	48	52	381 - 446	347
	S5	61.3	23	14	9	CL	2.81	0	38	62	439	309
	S7	65	42	21	21	CL	2.83	0	28	72	--	434

Table B-2: Summary of Static, Cyclic and Post-Cyclic Direct Simple Shear

Borehole Number	Sample ID	Depth m	Total Unit Weight γ kN/m^3	Effective Overburden Stress σ'_{v0} kPa	Undrained Shear Strength from Static DSS S_u kPa	Undrained Shear Strength Ratio S_u/σ'_{v0}	Cyclic Stress Ratio τ_{cyc}/σ'_{v0}	Number of Cycles at 0.1 Hz to Trigger Cyclic Mobility	Post Cyclic Undrained Shear Strength S_{ur} kPa	Post Cyclic Shear Strength Ratio S_{ur}/σ'_{v0}
0.220.13 BH15-32	S3	60.1	20	1100	--[1]	--[1]	0.12	>30[4]	225	0.21
							0.17	10	200	0.20
							0.21	3	130	0.13
	S5	61.3	21	1100	230	0.21	0.1	>30	--[2]	--[2]
							0.14	>30	--[2]	--[2]
							0.18	6	220	0.20
	S7	65.1	20	1150	280	0.24	--[3]	--[3]	--[3]	--[3]
				2300	530	0.23				

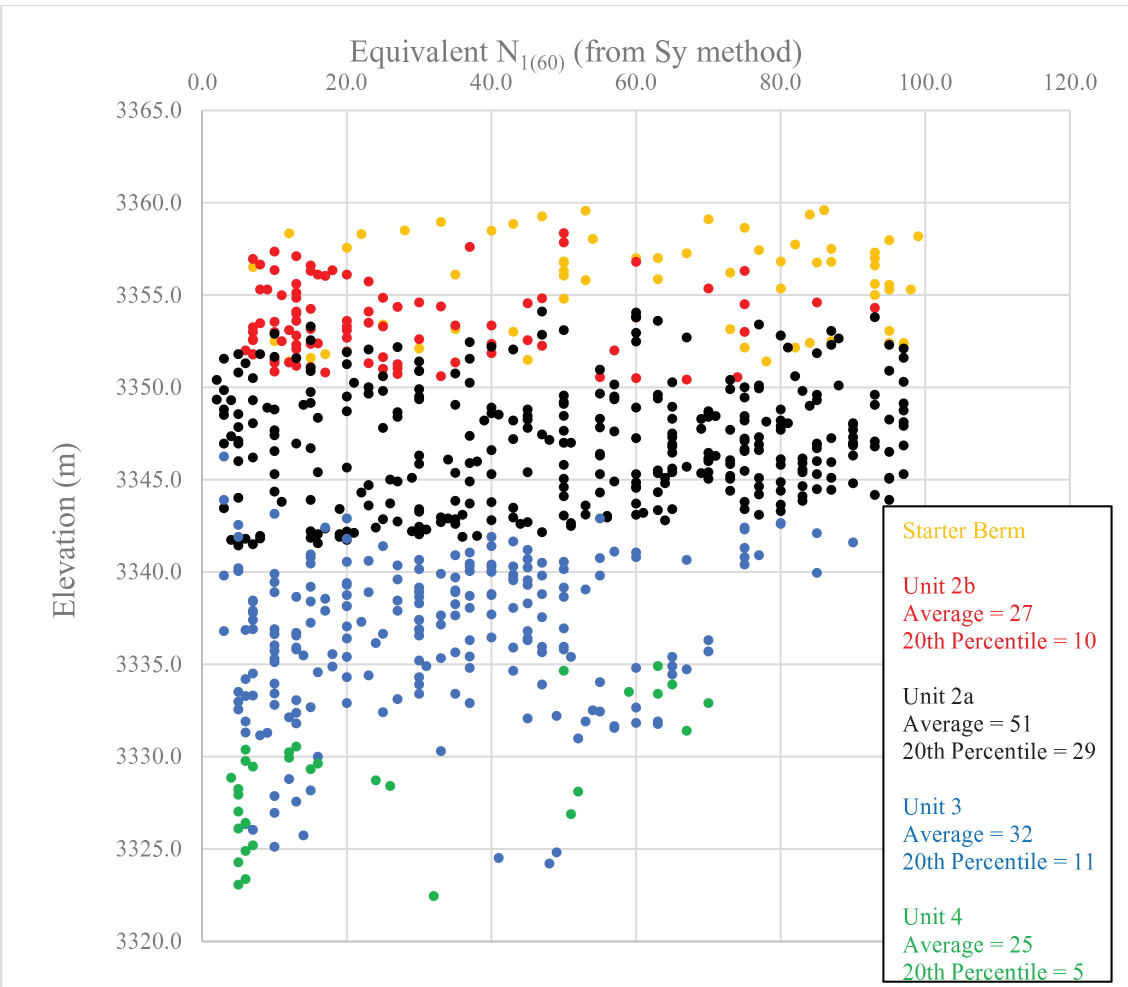
Notes: 1. Not tested. 2. Sample did not satisfy criteria for post-cyclic testing, i.e. did not experience transient cyclic mobility nor more than 3.5% strain Post cyclic undrained shear strength likely is at least equal to the static undrained shear strength. 3. Insufficient sample without laminations to continue testing S7. 4. Strain after 30 cycles was less than 3.5%.

C. BECKER PENETRATION TEST DATA

KCL completed eight deep closed-ended Becker penetration tests in 1998 and 20 additional in 2003. The data was used in the liquefaction analyses completed by KCL and in the design of the Old TDF seismic upgrade. The data was also used for evaluation and design of ground improvement.

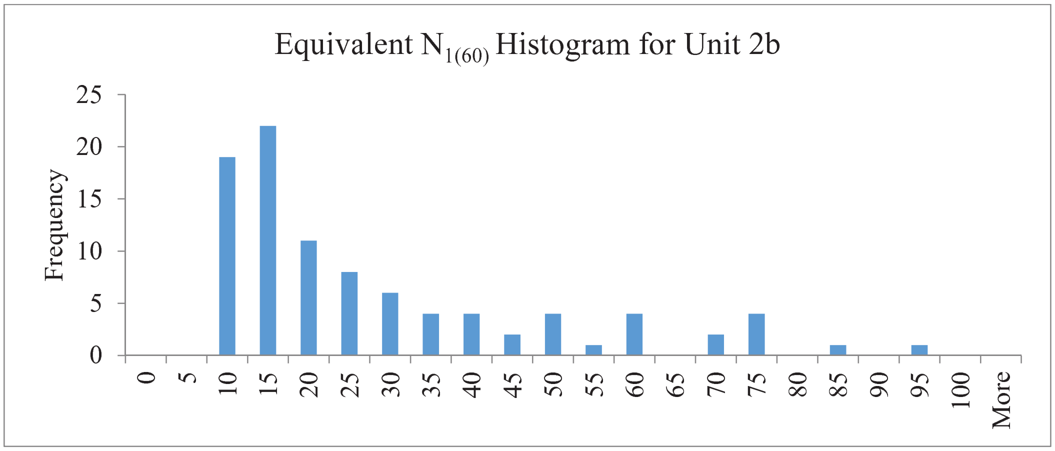
KCL calculated the equivalent $N_{1(60)}$ values using the Sy (1997) method. The corrected equivalent $N_{1(60)}$ values were presented in the available reports as plots. Amec Foster Wheeler digitized the available plots and obtained the $N_{1(60)}$ values. Plots were not available for all closed-ended Becker penetration test boreholes. The corresponding units are based on the KCL logs and stratigraphic profiles. The values are provided in Figure C-1.

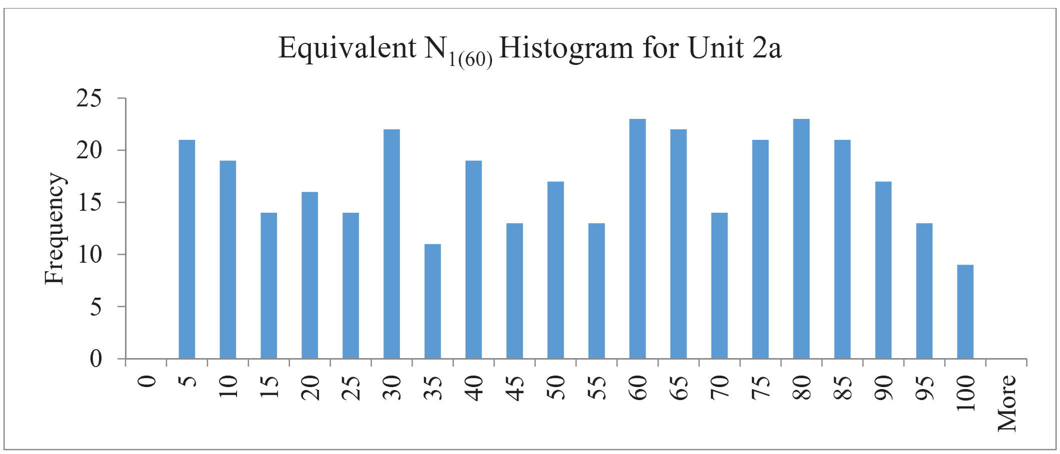
Figure C-1: Equivalent $N_{1(60)}$ Values Excluding Values Greater than or Equal to 100

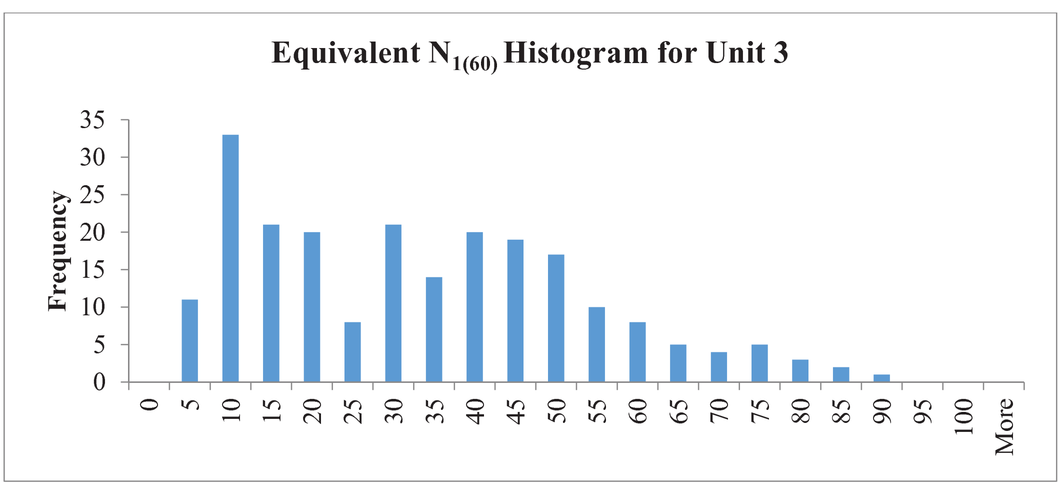


Histograms of the equivalent $N_{1(60)}$ values were developed for Units 2a, 2b and 3. The histograms show the uniform variability of Unit 2a when compared to Units 2b and 3. The histograms are shown in Figure C-1.

Figure C-2: Histograms of Equivalent $N_{1(60)}$ values for Units 2b, 2a and 3



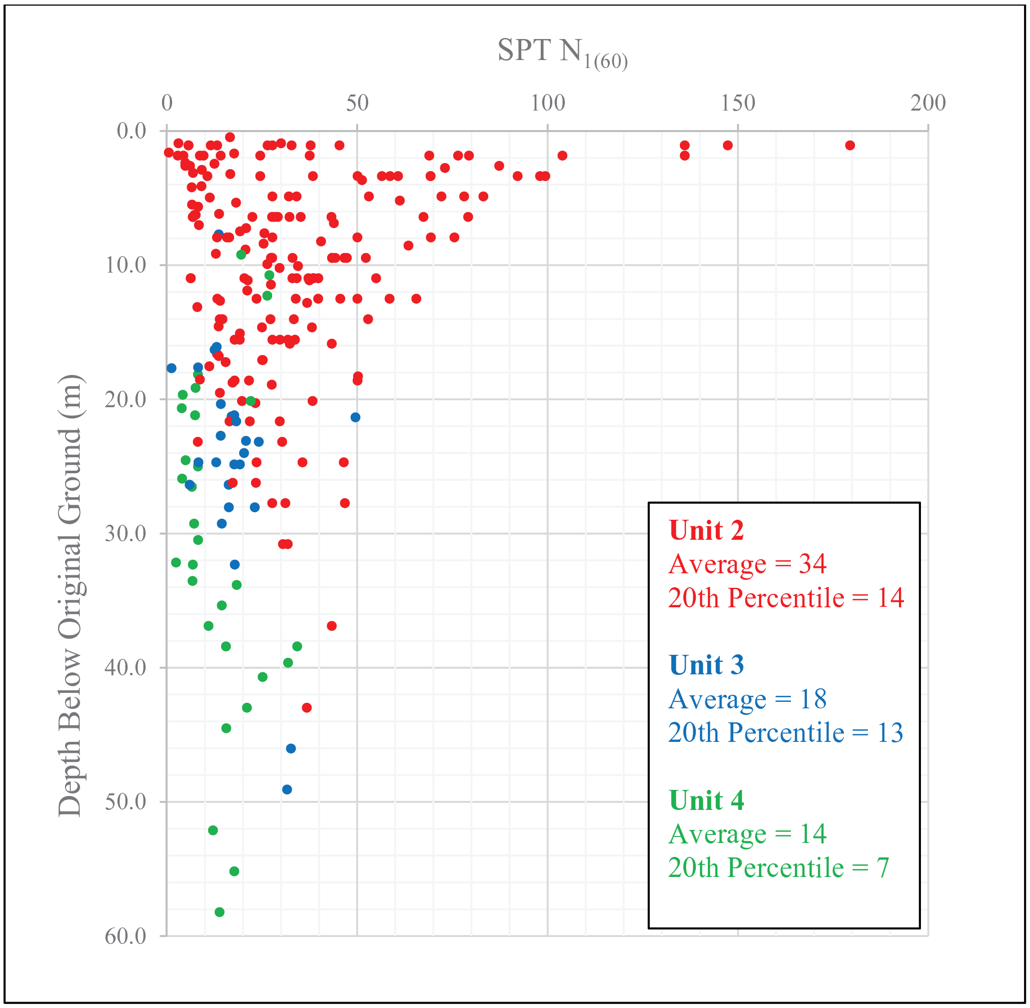




D. STANDARD PENETRATION TEST DATA

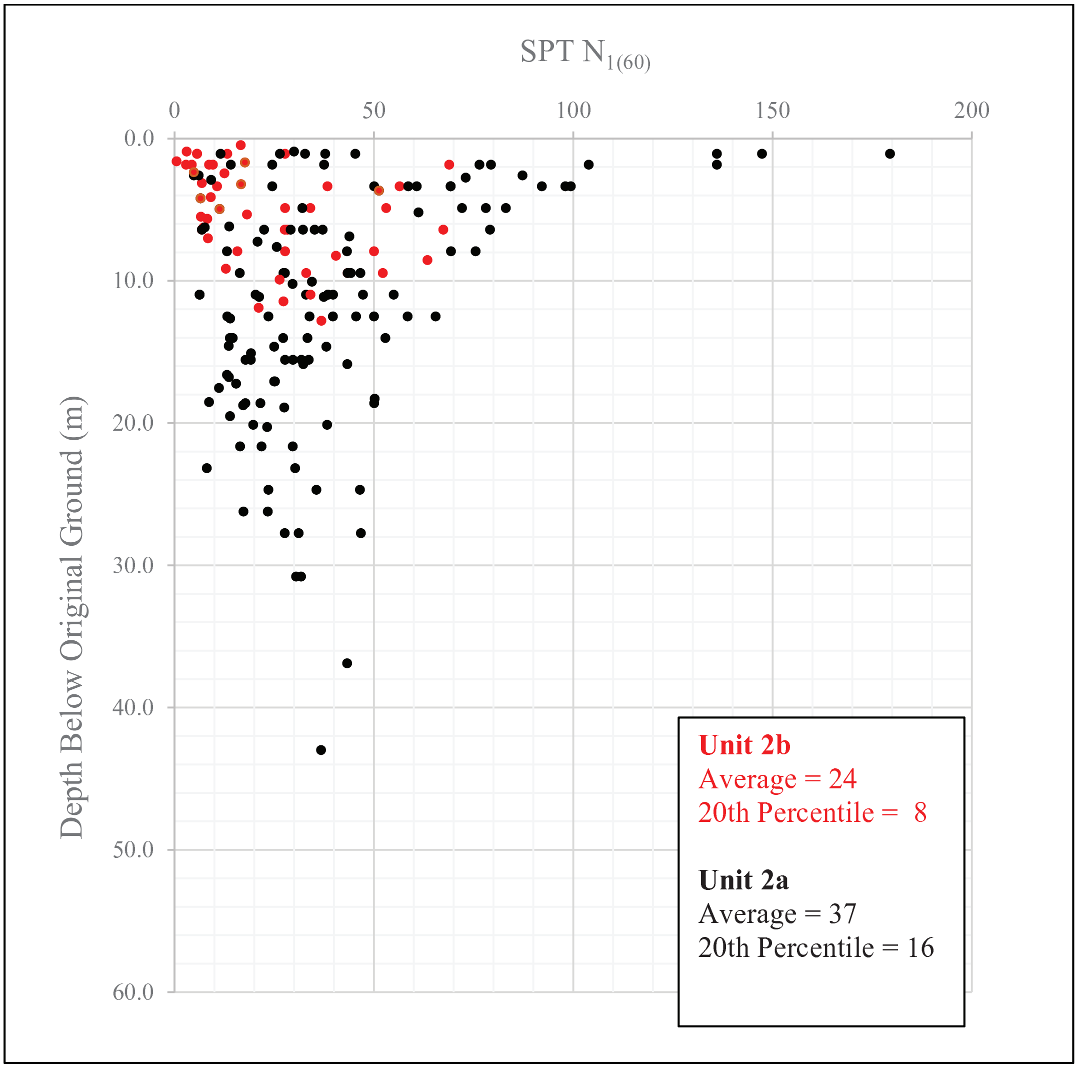
Standard penetration testing was conducted in the 14 mud rotary boreholes advanced by KPL, the two mud rotary boreholes advanced by KCL and the mud rotary boreholes advanced by Amec Foster Wheeler. The values were energy corrected (hammer energy, $E_m = 0.6$, borehole diameter factor, $C_B = 1.0$, sampling method factor, $C_s = 1.0$ and rod length correction, C_R based on the sampling depth). The values were also overburden stress normalized to one atmosphere. The energy corrected, overburden stress normalized $N_{1(60)}$ values as well as the average and 20th percentile for each unit are shown in Figure D-1. ***SPT N-values greater than 100 – about 25 percent of the total number of values - have been excluded.***

Figure D-1: SPT $N_{1(60)}$ vs Depth below Original Ground



Unit 2 has been further divided into two subunits – Unit 2a consisting of fluvial sand and gravel and Unit 2b consisting of loose silty sand and gravel (colluvium). Unit 2b is present generally between Stations 0+00 and 4+00 (western Old TDF); however, colluvium could be present in the unexplored areas of the Old TDF foundation. The SPT $N_{1(60)}$ values, averages and 20 percentile values for Units 2a and 2b are shown in Figure D-2.

Figure D-2: $N_{1(60)}$ vs Depth for Unit 2a and 2b



The average and 20th percentile values for BPT and SPT $N_{1(60)}$ values are provided in Table G-1 for comparison.

Table D-1: Average and 20th Percentile BPT and SPT $N_{1(60)}$ Values

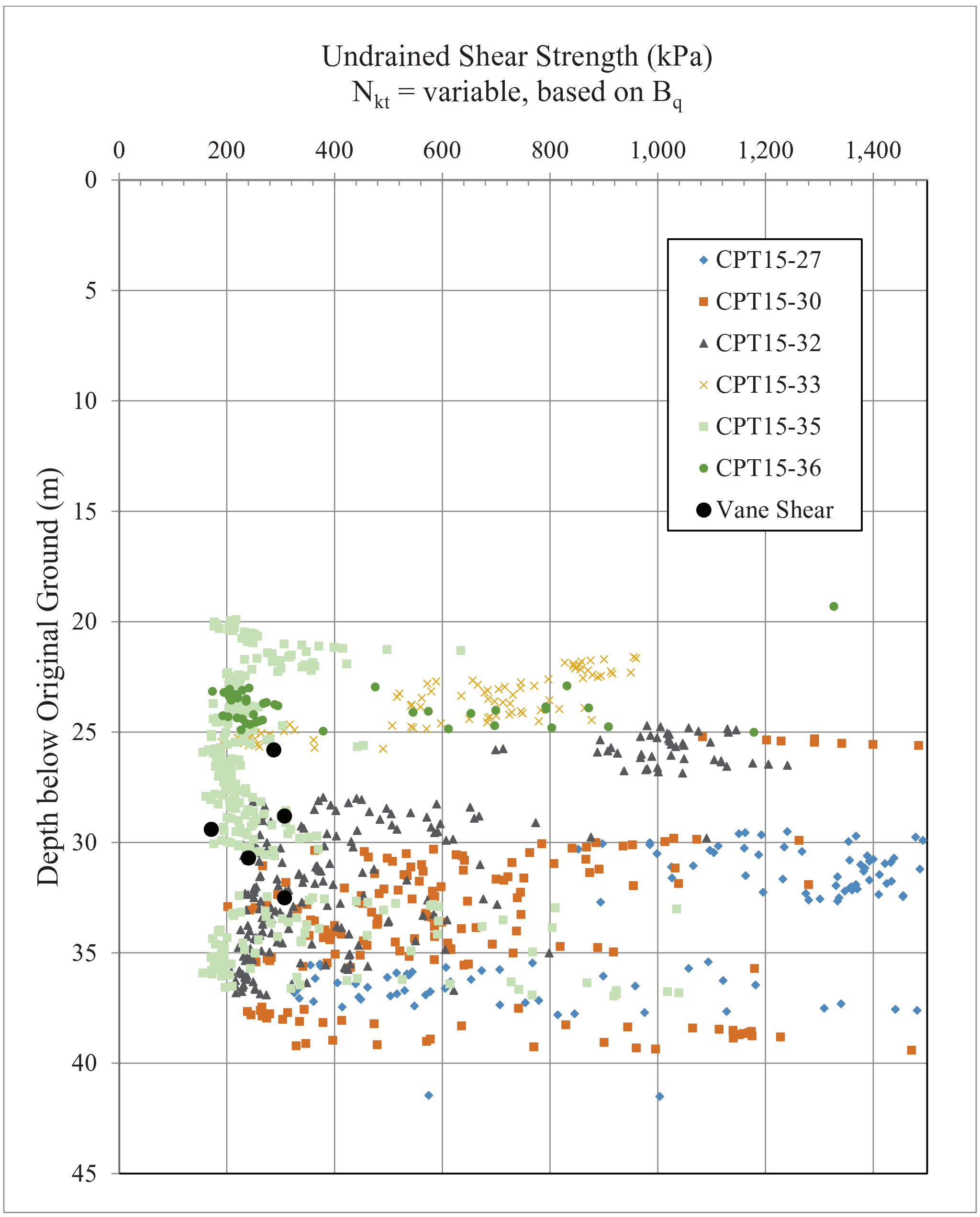
Unit	Average		20th Percentile	
	BPT	SPT	BPT	SPT
2b	27	24	10	8
2a	51	37	29	16
3	32	18	11	13
4	25	14	5	7

E. IN-SITU TESTING – CONE PENETRATION TESTING AND VANE SHEAR TESTS

Cone penetration testing (CPT) was conducted in the glaciolacustrine (Unit 4) soils to further assess the undrained shear strength as well as evaluate the potential for cyclic mobility. In-situ vane shear testing was conducted to 'calibrate' the undrained shear strength from CPT. All of the CPT and vane shear data is provided in the data report[1].

The calculated undrained shear strength from the CPT and results of the vane shear tests are provided in Figure E-1.

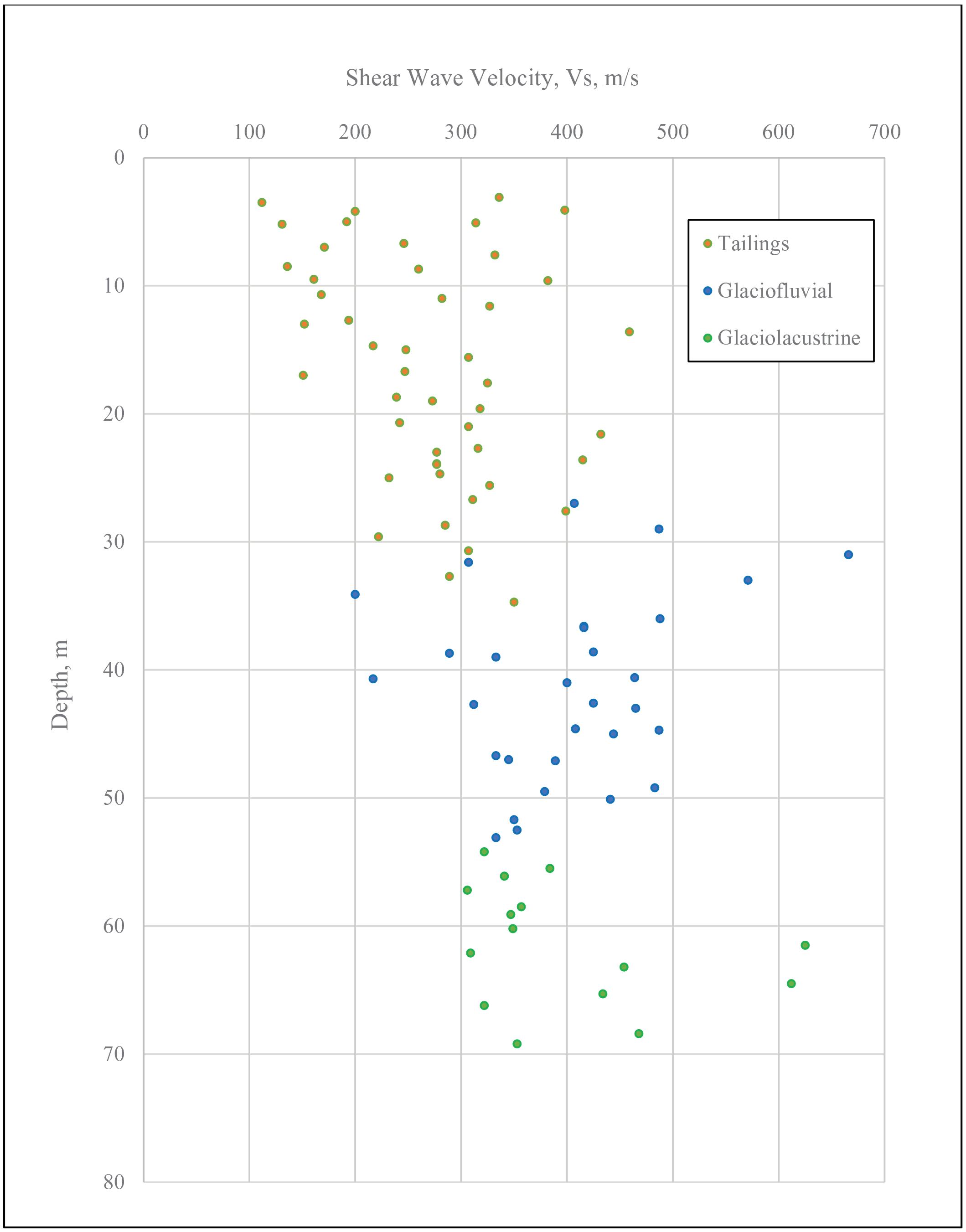
Figure E-1: Undrained Shear Strength from CPT and In-situ Vane Shear Testing



F. DOWNHOLE SHEAR WAVE VELOCITY MEASUREMENTS

Downhole shear wave velocity measurements were made in casings installed in three of the boreholes. The measurement interval varied between one meter and four meters. The values are shown in Figure F-1.

Figure F-1: Downhole Shear Wave Velocity Measurements



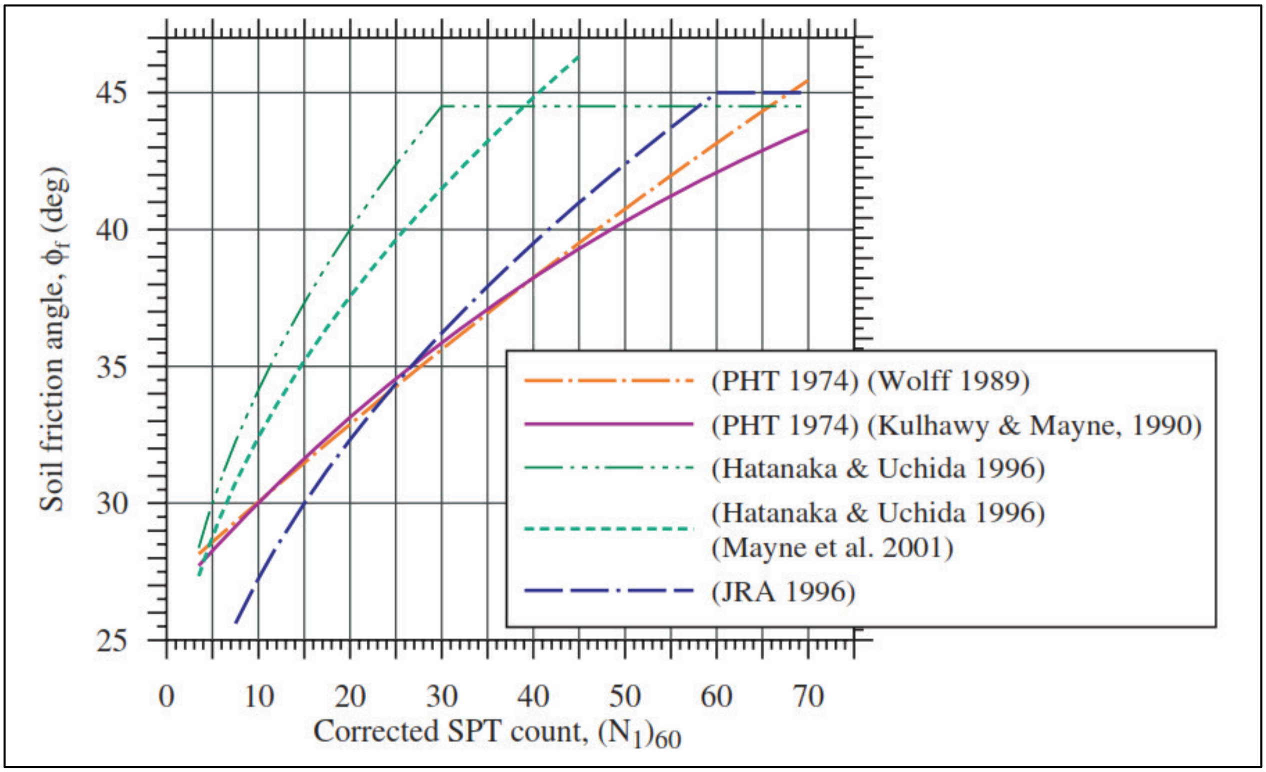
G. STATIC AND POST-SEISMIC MATERIAL PROPERTIES FOR FOUNDATION SOILS

The static and post-seismic materials properties were estimated from the field and laboratory data

G.1 Assessment of Static Strength Parameters for Granular Foundation Soils (Units 2b, 2a and 3)

The static strength parameters were estimated using common correlations[ii] between average $N_{1(60)}$ and angle of internal friction as shown in Figure G-1. The correlation proposed by Peck Hansen and Thornburn (PHT 1974) was selected. The values proposed for use in slope stability analyses are provided in Table 1.

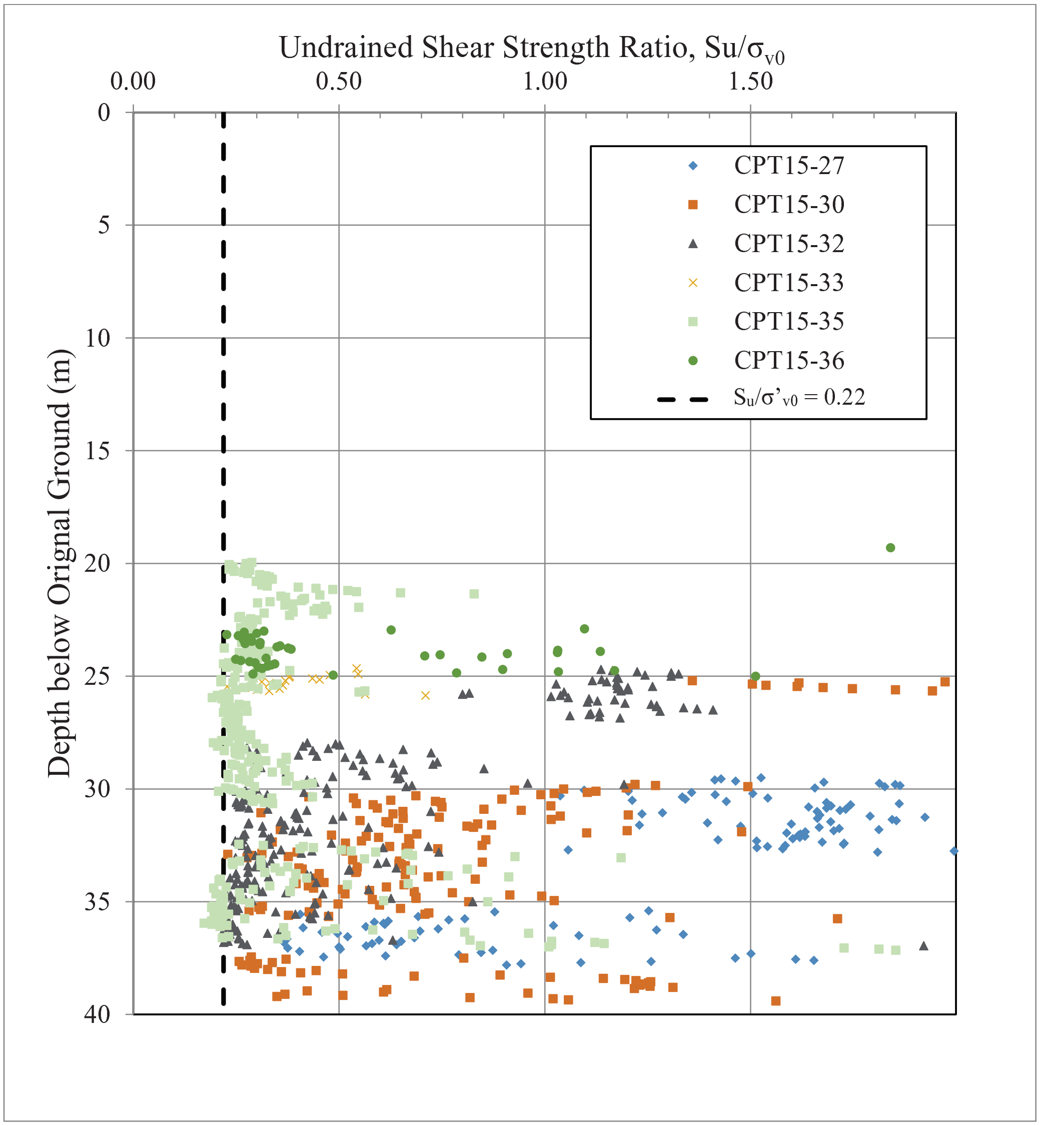
Figure G-1: Correlations between Granular Soil Friction Angle and $N_{1(60)}$



G.2 Assessment of Static Strength Parameters for Glaciolacustrine Soils

The results of the static DSS tests indicate that the glaciolacustrine soils will behave in a contractant manner during shear; therefore, the static stability of the Old TDF is best assessed using undrained strength parameters for the glaciolacustrine soils, specifically an undrained shear strength ratio, s_u/σ'_{v0}. The undrained shear strength ratio for the glaciolacustrine soils was estimated from CPT data. The results of the analyses are shown in Figure G-9. Based on analysis of the CPT data, an undrained shear strength of 0.22 was selected for use in stability analyses. This value is consistent with the magnitude of the undrained shear strength ratio estimated from DSS test data and the vane shear data.

Figure G-2: Undrained Shear Strength Ratio, s_u/σ'_{v0} from CPT vs Depth below Original Ground

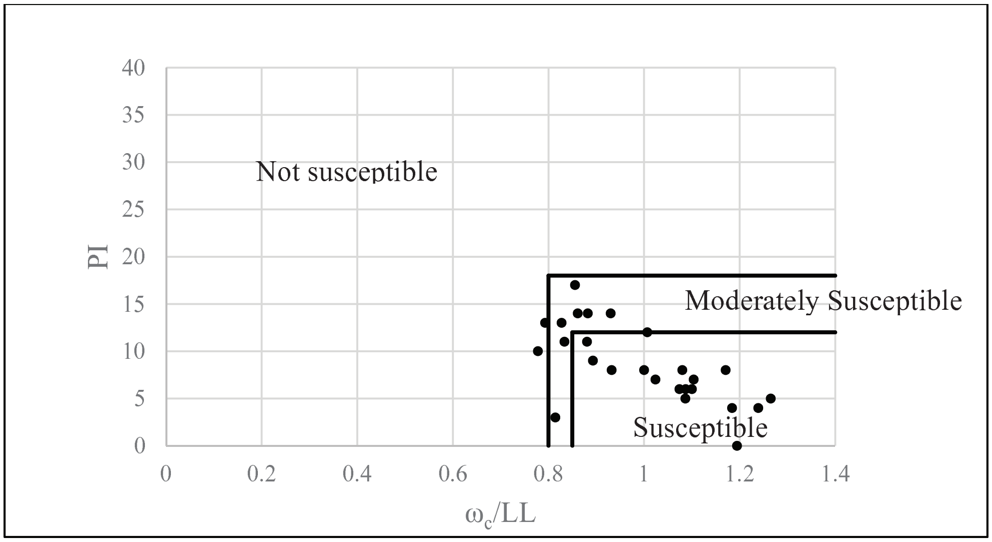


G.3 Assessment of Post-seismic Residual Shear Strength Parameters from Standard Penetration Testing and Laboratory Testing

The post-seismic residual shear strength parameters for Units 2b, 2a and 3 were estimated using $N_{1(60)}$ values and several correlations and relationships available in the literature. The correlations rely heavily on back analysis of a small number of case histories of failures. Failures are relatively rare occurrences and the pre-failure conditions are generally not well documented. There is uncertainty associated with the available correlations and relationships; therefore, a level of conservatism should be incorporated in the assessment of post-seismic residual shear strength parameters.

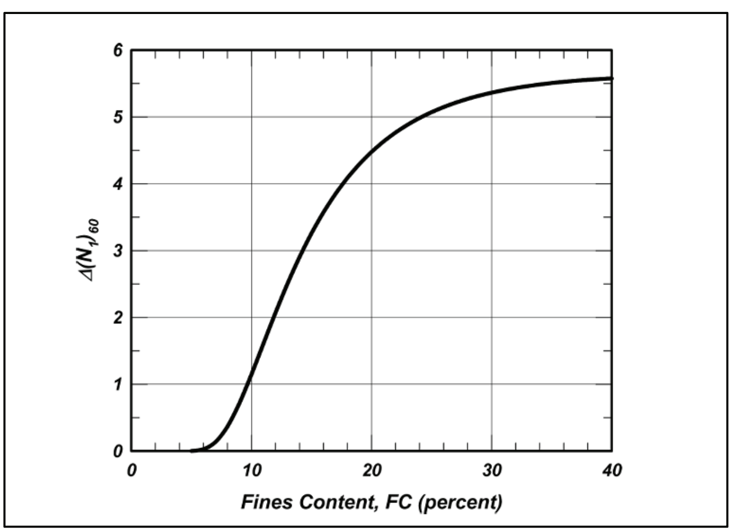
The correlations are generally for granular soils such as sands and silty sands with limited data for gravels. The index properties of the glaciolacustrine soils (Unit 4) were plotted against the liquefaction criteria proposed by Bray and Sancio[iii] as shown in Figure G-3 . The criteria suggests that the glaciolacustrine soils are susceptible to liquefaction. As such, the post-seismic strength of Unit 4 can be evaluated in a similar manner as the granular materials of Units 2b, 2a and 3.

Figure G-3: Liquefaction Susceptiblity of the Glaciolacustrine Materials



Energy corrected, stress normalized $N_{1(60)}$ values discussed in Section D were further corrected to account for the effect of fines content in evaluation of post-seismic residual strength. Figure G-4 which was excerpted from Idriss & Boulanger[iv]. Based on fines content, the average $N_{1(60)}$ values for Units 2b and 3 were increased by 3; the average $N_{1(60)}$ for Unit 4 by 5.

Figure G-4: Variation of $\Delta N_{1(60)}$ with fines Content Used in Evaluation of Post-Seismic Residual Strength



A summary of the average and 20th percentile $N_{1(60)}$ and $N_{1(60\text{-}CS}$ values are provided in Table G-1.

Table G-1: Average and 20th Percentile $N_{1(60)}$ and $N_{1(60)\text{-}CS}$ Values

Unit	$N_{1(60)}$				$N_{1(60)\text{-}CS}$			
	Average		20th Percentile		Average		20th Percentile	
	SPT	BPT	SPT	BPT	SPT	BPT	SPT	BPT
2b	24	27	7	10	27	30	11	13
2a	37	51	16	29	37	51	16	29
3	18	32	13	11	23	37	18	16
4	14	25	7	5	19	30	12	10

Evaluation of the post-seismic residual shear strength ratios for Units 2, 3 and 4 were completed as follows:

- For average N1(60) values (not adjusted for fines content) that were less than or equal to 15, the liquefied shear strength ratio was estimated using the relationship proposed by Olsen & Starkv.
- For average N1(60) values greater than 15 but less than 25, a linear variation of liquefied shear strength ratio was used.
- For average N1(60) values greater than 25, the drained strength was used.

Checks were made using other methods of evaluation as discussed in this section.

Based on average $N_{1(60)}$ values provided in Table G-1, Olsen & Stark only applies to Units 2b, 3 and 4. The average $N_{1(60)}$ value for Unit 2a is greater than 25 and the drained shear strength parameters were used in the analyses. The average $N_{1(60)}$ values for Units 2b, 3 and 4 are 24,18 and 14, respectively, and the liquefied shear strength ratio should be used in the analyses.

The residual shear strength ratio can also be assessed using average $N_{1(60)}$ values that are not corrected for fines content and the correlation proposed by Olsen & Stark.

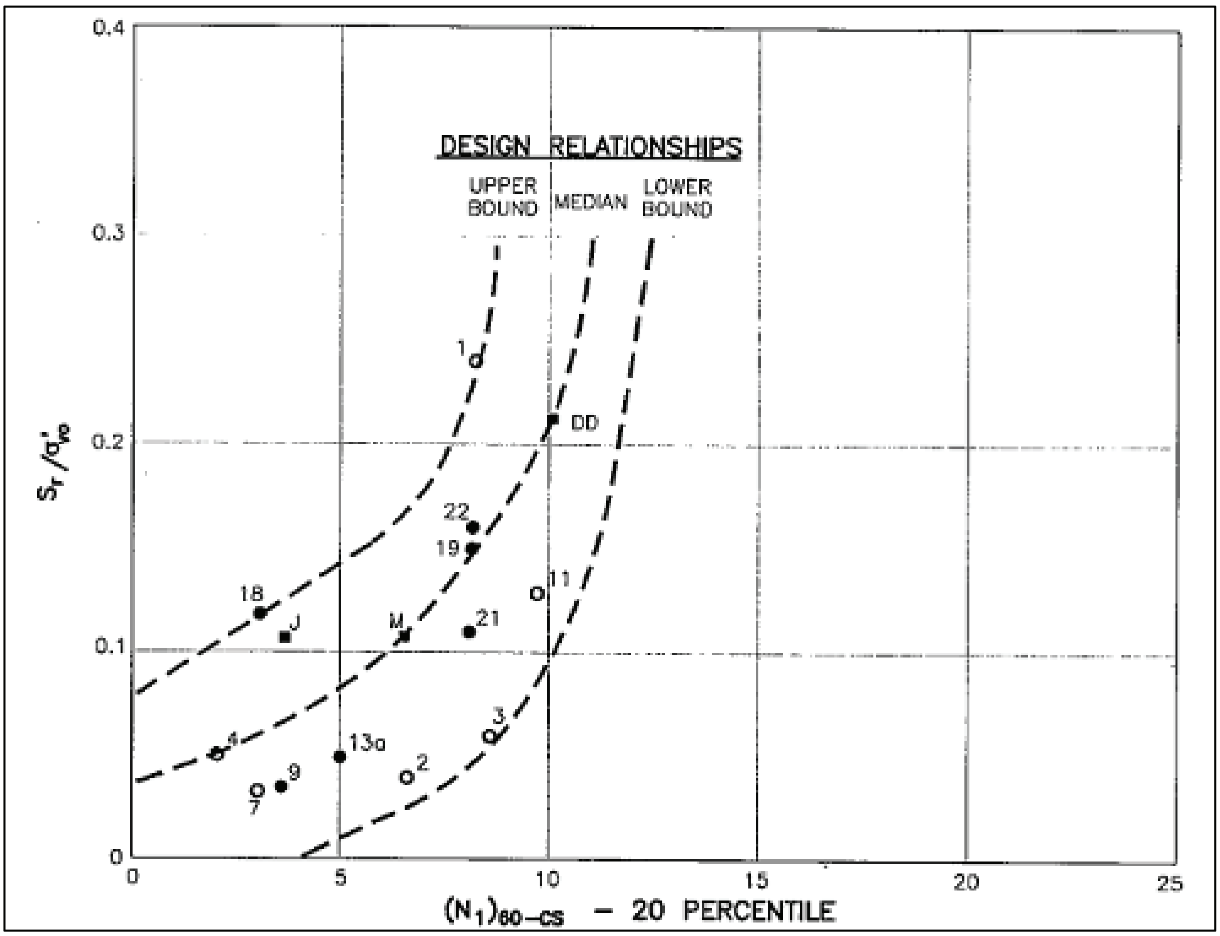
$$S_u(LIQ)/\sigma'_{v0} = 0.03 + 0.0075N_{1(60)}$$

Use of this relationship is limited to for values less than or equal to 12 but was linearly extrapolated for values up to 25.

Based relationship, the residual shear strength ratios for Unit 2b and Unit 3 are 0.21 and 0.17, respectively. For Unit 4, the residual shear strength ratio is 0.13. This value is consistent with the value obtained from the results of lower of the three post-cyclic, monotonic direct simple shear tests.

The design relationships proposed by Robertson[vi] was used to further assess residual shear strength ratios for use in post-seismic slope stability analyses. This relationship applies to soils with 20th percentile N1(60)-CS values that are less than about 12 based on Figure G-5. Only Units 2b and 4 have 20th percentile N1(60)-CS values less than 12. For a 20th percentile N1(60)-CS value of 10, the corresponding residual shear strength ratio is about 0.21; for a value of 12, the corresponding residual shear strength ratio is 0.35.

Figure G-5: Design Relationships - Robertson



G.4 Assessment of Post-seismic Residual Shear Strength Ratios from Cone Penetration Testing Data

The residual shear strength ratio also can be assessed using tip resistance, q_{c1}, calculated from cone penetration data and the correlation proposed by Olsen & Stark.

$$S_u(LIQ)/\sigma'_{v0} = 0.03 + 0.0143q_{c1}$$

Cone penetration testing was conducted in the glaciolacustrine soils as discussed in Attachment E. This correlation is appropriate for q_{c1} less than or equal to 6.5MPa.

The normalized tip resistance is provided in Figure G-6. The residual shear strength ratios with depth are provided in Figure G-7. The residual shear strength ratios calculated using the correlation proposed by Olsen & Stark are unrealistically low and inconsistent with that estimated using other methods. Olsen & Stark suggest that the q_{c1} values should not be corrected for fines content; however, it is common practice to do so. The residual shear strength ratios calculated using fines corrected are shown in Figure G-7 using the open symbols. The lower bound is around 0.12, also as shown in Figure G-7.

Figure G-6: Tip Resistance, q_{c1}, vs Depth below Original Ground

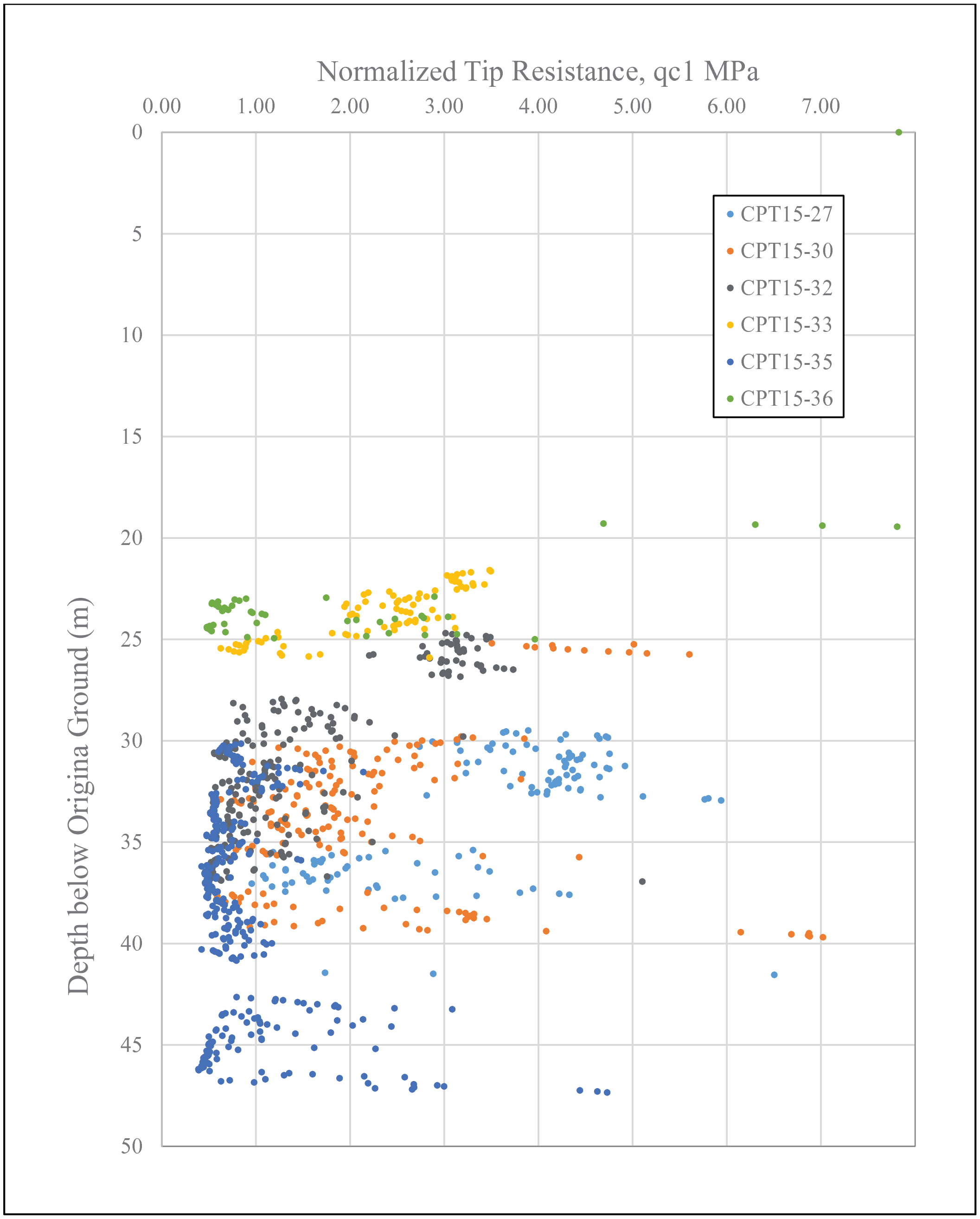
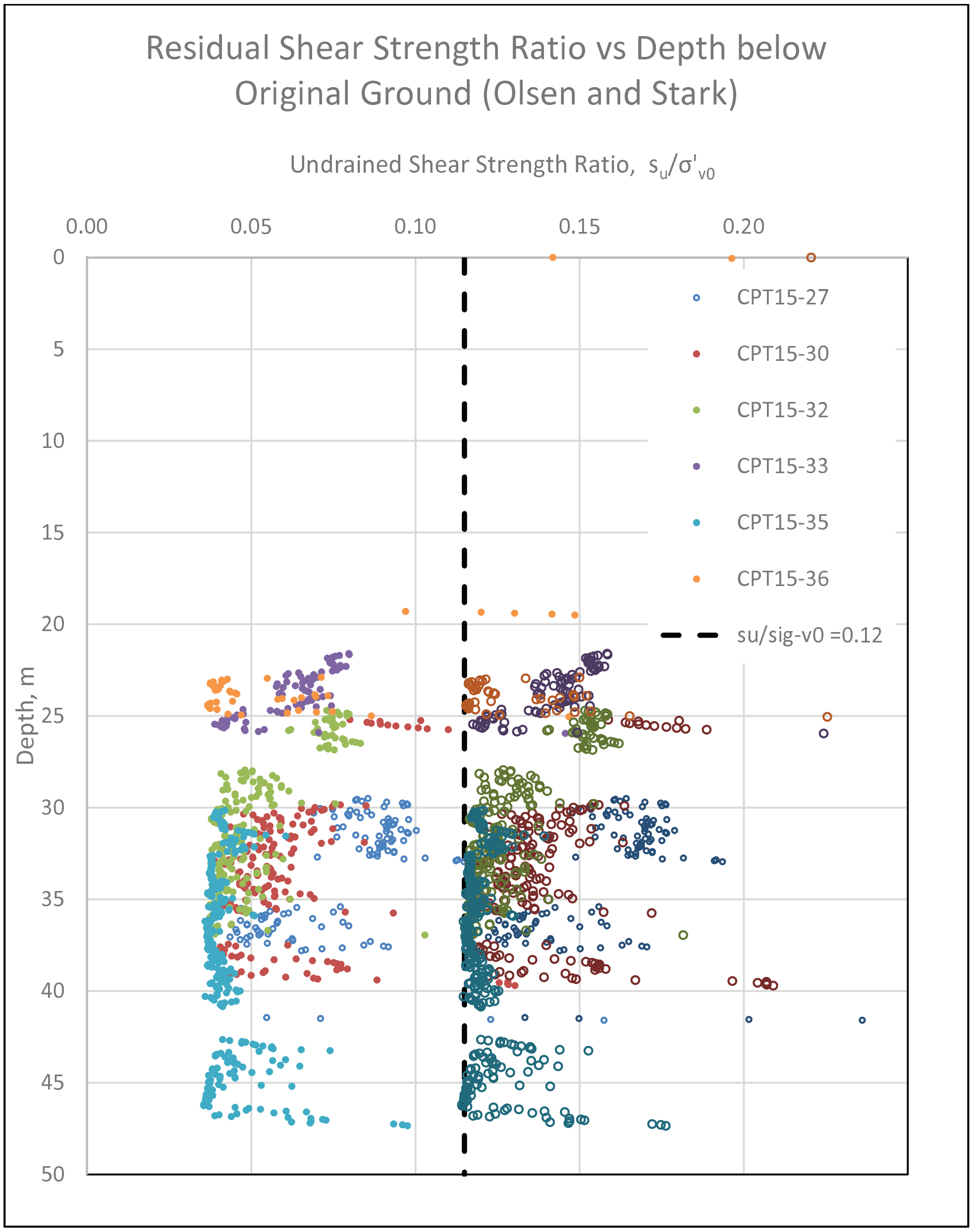


Figure G-7: Residual Shear Strength Ratio vs Depth below Original Ground



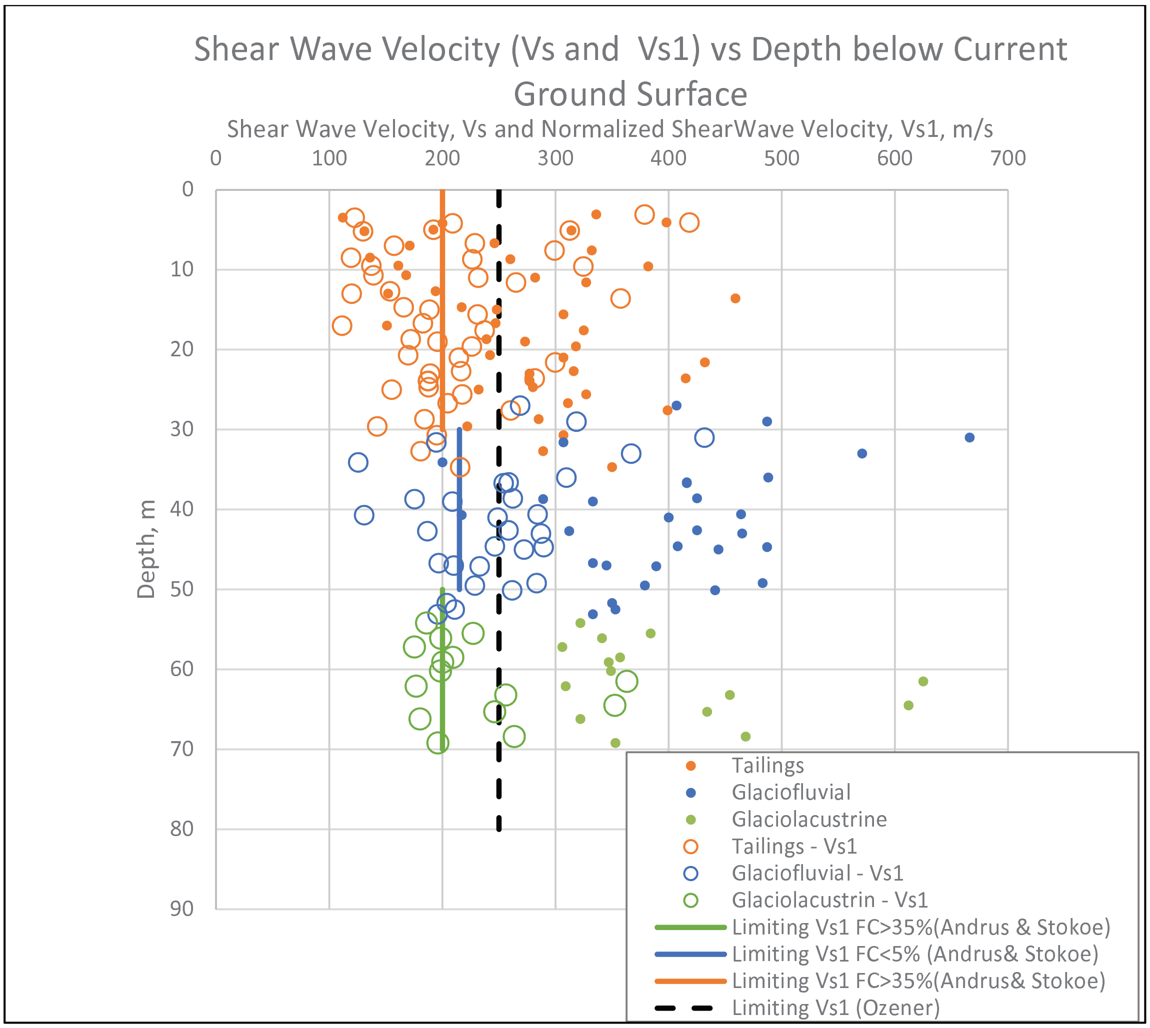
G.5 Assessment of Post-seismic Residual Shear Strength Ratios from Shear Wave Velocity

The residual shear strength ratio of liquefied soils can also be assessed using normalized shear wave velocity, V_{s1}, and the correlation proposed by Özener[vii].

$$\frac{S_{ur}}{\sigma'_{v0}} = 0.0218EXP(0.0103v_{s1})$$

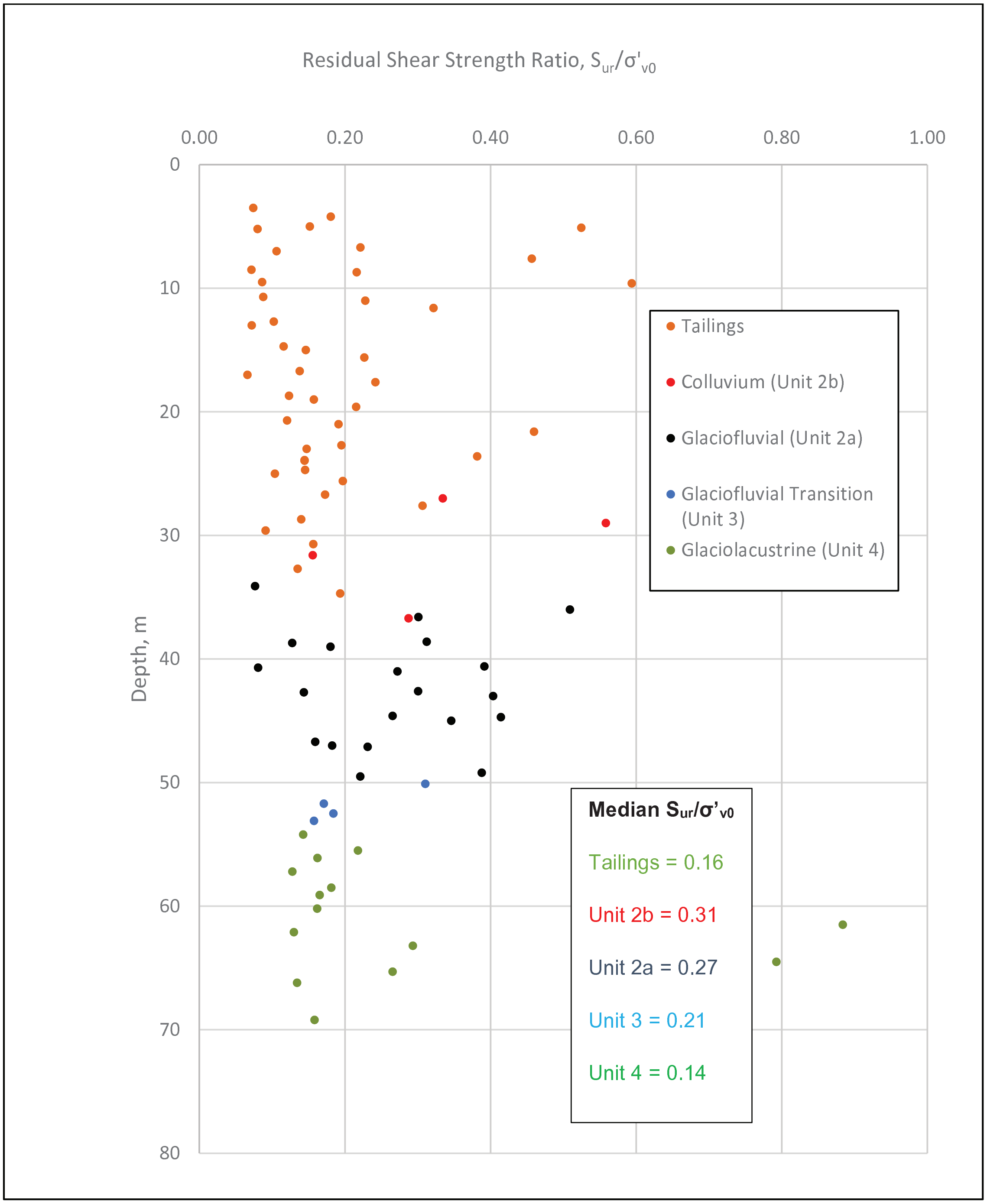
Shear wave velocities were measured from the existing ground surface through the tailings and foundation soils as discussed in Attachment 0. Soils with normalized shear wave velocities greater than 250 m/s are considered unlikely to be subject to transient liquefaction according to Özener. Limiting normalized shear wave velocities for liquefaction have also been proposed by Andrus & Stokoe[viii] as 215 m/s for clean sands and 200 m/s for soils with greater than 35% fines. The measured and normalized shear wave velocities with limiting values are shown in Figure G-8.

Figure G-8: Measured, Normalized and Limiting Shear Wave Velocities with Depth below Current Ground Surface



Based on Figure G-8, the tailings and glaciolacustrine soils (Unit 4) are potentially subject to transient liquefaction and there are potentially liquefiable layers in the granular foundation soils (Units 2a, 2b and 3). The calculated residual shear strength ratios are provided in Figure G-9. Median values for each unit also are shown in Figure G-9.

Figure G-9: Residual Shear Strength Ratios from V_{s1}



G.6 Comparison of Estimated Post-Seismic Residual Shear Strength and Proposed Values

A comparison of the estimated post-seismic residual shear strength ratios is provided in Table G-2. The values proposed for use in slope stability analyses also are shown.

Table G-2: Comparison of Post-seismic Residual Shear Strength Ratios

Unit	Olsen & Stark based on Average $N_{1(60)}$	Olsen & Stark based on q_{c1N} with fines corrections	Robertson, based on 20th Percentile $N_{1(60)\text{-CS}}$	Median based on Özener V_{s1}	Laboratory	Proposed
2b	0.21	--	0.28	0.31	--	0.28
2a	NA	--	NA	0.27	--	--
3	0.17	--	0.35	0.21	--	0.35
4	0.13	0.12	0.29	0.17	0.13	0.13

The residual shear strength ratio, $su/_{\sigma'v)}$ = 0.13, proposed for use in post-seismic stability analyses for assessing the impact of the presence of glaciolacustrine soils on the stability of the Old TDF is considered conservative. Cyclic simple direct shear tests were conducted on six samples of the glaciolacustrine soils (Unit 4). Cyclic mobility was triggered in three of the six samples. Post-cyclic monotonic direct shear tests were conducted on these samples. In addition, one sample in which cyclic mobility was not triggered was also tested. The post-seismic residual shear strength ratios ranged between 0.13 and 0.21. Excluding the low value of 0.13, the average of the remaining three values was 0.20, essentially equal to the static undrained shear strength ratio. This finding suggests that the glaciolacustrine soils might not experience significant strength loss after a seismic event and that the post-seismic residual strength ratio could be closer to 0.2 than to 0.13.

Additionally, the residual shear strength ratio based on Olsen & Stark provided in Table G-2 is based on the average $N_{1(60)}$ as discussed in Section G.2. Though Olsen & Stark suggest that $N_{1(60)}$ values should not be corrected for fines content, it is common practice to do so. If the average $N_{1(60)}$ value for Unit 4 is corrected for fines content, the corresponding residual shear strength ratio would be about 0.17. This value is equal to that using the relationship proposed by Özener which is based on shear wave velocity.

The assessment of the CPT data using the relationship proposed by Olsen & Stark resulted in a post-seismic residual shear strength ratio of about 0.05 for Unit 4. CPT was not conducted in Units 2a, 2b or 3. Again, Olsen & Stark suggest that the q_{c1} values should not be corrected for fines content. If these values are corrected for fines content, the corresponding residual shear strength ratio the lower bound is around 0.12. Without the fines content correction, the residual shear strength value appears to be unrealistically low when compared with that estimated using other methods and from laboratory testing. With the fines content correction, the lower bound residual shear strength value of 0.12 is consistent with the other estimates. Based on the foregoing, a residual shear strength ratio, $s_u/\sigma'_{v0)}$ = 0.13, for the glaciolacustrine soils (Unit 4) is considered a lower bound.

For Units 2a, 2b and 3, Amec Foster Wheeler has based the proposed residual shear strength ratios on analysis of SPT data. If a similar analyses were completed using the BPT data, the corresponding residual shear strength ratios for all units would be higher.

[ii] LRFD Design and Construction of Shallow Foundations for Highway Bridge Structures (NCRP Report 651), Transportation Research Board, 2010

[iii] Liquefaction Susceptibility of Fine-Grained Soils, Bray, J.D., Sancio, R.B, Journal of Geotechnical Engineering, Vol. 132, No. 9, 2006

[iv] Soil Liquefaction During Earthquakes, Idriss, I.M., Boulanger, R. W., Earthquake Engineering Research Institute, MNO-12, 2008

[v] Liquefied Strength Ratio from Liquefaction Flow Failure Case Histories, Olson, S.M., Stark, T.D., Canadian Geotechnical Journal, Vol. 39, May 2002

[vi]

[vii] Estimation of Residual Shear Strength Ratios of Liquefied Soil Deposits from Shear Wave Velocity, Ozener, P., Earthquake Engineering and Engineering Vibrations, Vol. 11, No. 4, December 2012

[viii] Liquefaction of Soils from Shear Wave Velocity, Andrus, R.D., Stokoe, K.H., Journal of Geotechnical and Geoenvironmental Engineering, Vol. 126, No. 11, November 2000.

Attachment A

Laboratory Test Results

MEG Technical Services (MTS)

(A Division of MEG Consulting Limited)

Marine + Earth MEG Geosciences

Form Nº MTS109

Project:	AMECFW - Myra Falls	Project No.:	15-MTS-029
Location:	BC	Date:	October 29, 2015
Borehole:	BH15-32A		

Specific Gravity (ASTM D854)

Sample Number	Depth (m)	Volumetric flask No.	Weight of flask and soil (g)	Weight of flask, water and soil (g)	Weight of dry solid (g)	Temperature (^{o}C)	Specific Gravity Gs
S7	65.14	2	221.89	702.03	52.40	21	2.83
S5	61.30	14	224.92	704.88	51.09	23	2.81

Comments :

Prepared by:	PC	Checked by:	MF	Approved by:	PS
Date:	October 29, 2015	Date:	October 29, 2015	Date:	October 29, 2015

MEG Technical Services (MTS)

(A Division of MEG Consulting Limited)

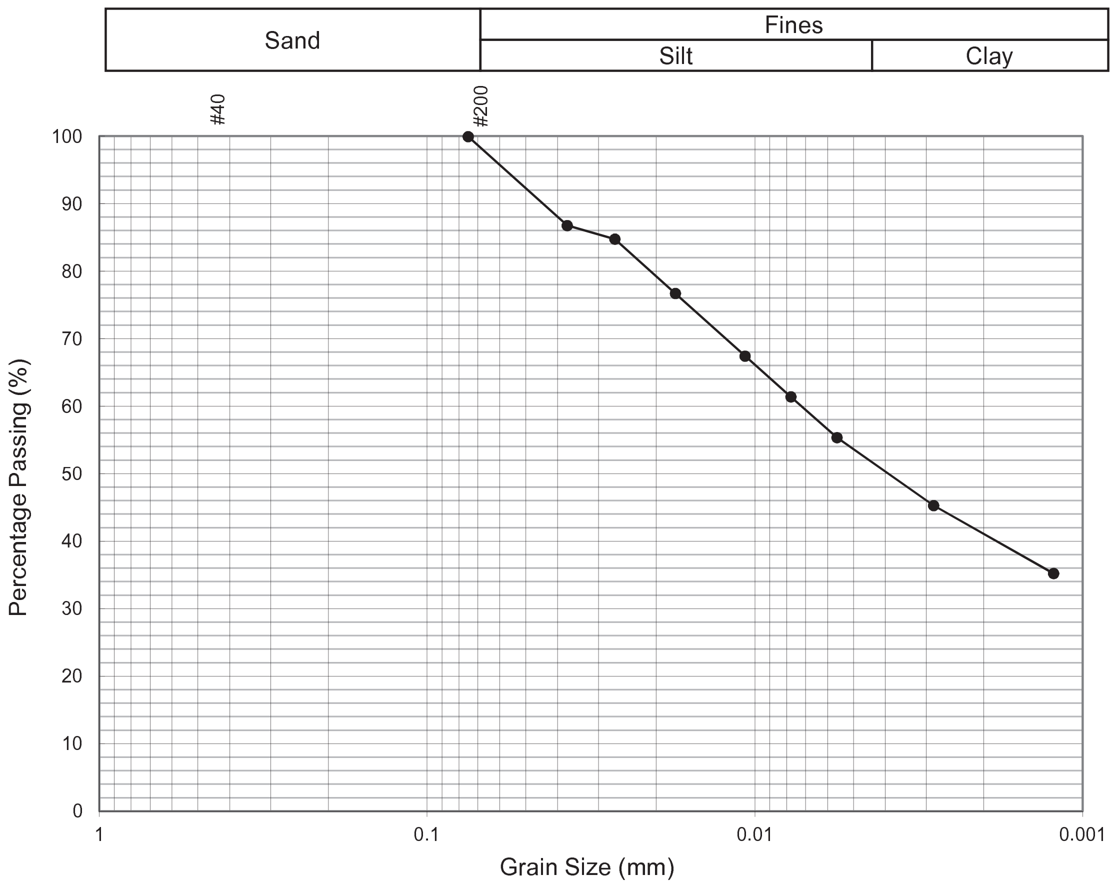
Form Nº MTS107

Project:	AMECFW - Myra Falls			Project No.:	15-MTS-029
Location:	BC			Date:	December 4, 2015
Borehole:	BH15-32A	Sample No.:	S3	Depth (m):	60.07

Hydrometer Test (ASTM D422)

Unified Soil Classification System (ASTM D 2487)

Description of Material: Pale green lean CLAY



Sample No.	Depth (m)	Percentage of Material by Weight (%)			
		Gravel	Sand	Fines	
				Silt	Clay
S3	60.07	-	0	48	52

Comments:

Prepared by:	PC	Checked by:	MF	Approved by:	PS
Date:	December 4, 2015	Date:	December 8, 2015	Date:	December 9, 2015

Marine + Earth
MEG Geosciences

MEG Technical Services (MTS)

(A Division of MEG Consulting Limited)

Form N° MTS104

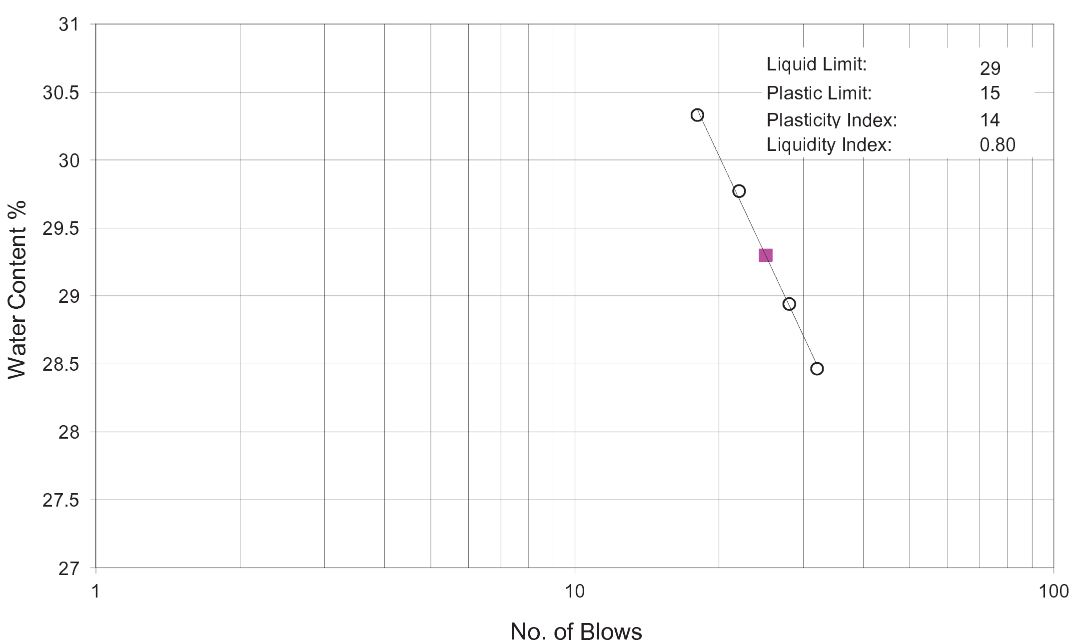
Project: AMECFW - Myra Falls		Project No.:	15-MTS-029
Location: BC		Date:	December 2, 2015
Borehole: BH15-32A	Sample No.: S3	Depth (m):	60.07

Liquid Limit, Plastic Limit and Plasticity Index of Soils (ASTM D4318)

LIQUID LIMIT								PLASTIC LIMIT						
TIN No.	Tare + Weight of Wet Soil (g)	Tare + Weight of Dry Soil (g)	Weight of Tin (g)	Weight of Water (g)	Weight of Dry Soil (g)	Water Content (%)	No. of Blows	TIN No.	Tare + Weight of Wet Soil (g)	Tare + Weight of Dry Soil (g)	Weight of Tin (g)	Weight of Water (g)	Weight of Dry Soil (g)	Water Content (%)
52A	32.04	30.11	23.33	1.93	6.78	28.5	32	18E	36.07	35.25	29.77	0.82	5.48	15.0
3	31.95	30.50	25.49	1.45	5.01	28.9	28	64C	36.75	35.93	30.53	0.82	5.40	15.2
86	33.18	31.48	25.77	1.70	5.71	29.8	22							
10A	39.76	38.48	34.26	1.28	4.22	30.3	18							

Classification of the material : CL

100 % with respect to the total of the material smaller than sieve No. 40



Observations:

Prepared by:	PC	Checked by:	MF	Approved by:	PS
Date:	December 2, 2015	Date:	December 3, 2015	Date:	December 4, 2015

OQM Organizational Quality Management Program

MEG Technical Services (MTS)

(A Division of MEG Consulting Limited)

Form Nº MTS217a

Marine + Earth MEG Geosciences

Project:	AMECFW - Myra Falls	**Borehole:**	BH15-32A	**Project No.:**	15-MTS-029
Client:	AMEC	**Sample No.:**	S3	**Date:**	December 7, 2015
Location:	BC	**Depth (m):**	60.07	**Station:**	DSS 1

Bender Element Velocity Measuring

WATER CONTENT & UNIT WEIGHT

	Initial	Final
Tin No.:	93	C106
Wt. of Tin (g):	23.66	150.44
Wet Weight (g):	102.56	334.23
Dry Weight (g):	86.17	299.29
Water Content (%):	26.2	23.5
Total Unit Weight (kN/m^3):	19.93	21.47
Dry Unit Weight (kN/m^3):	15.79	17.38

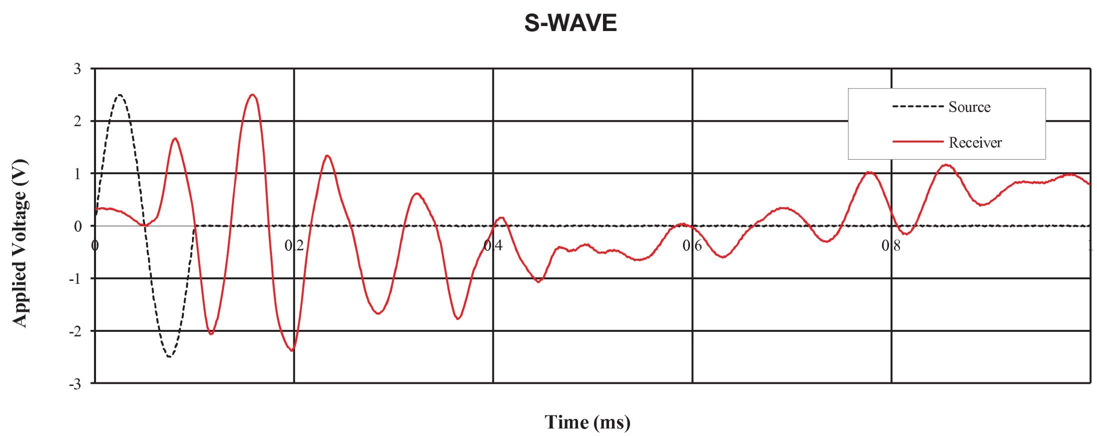
CONSOLIDATION

Specific Gravity, Gs:	2.81
Initial DSS Sample Height (mm):	23.5
Height after Consolidation (mm):	21.4
Initial Void Ratio, e_o	0.75
Initial Degree of Saturation (%)	98.7
Final Void Ratio, e_f	0.59
Final Degree of Saturation (%)	>100

BENDER ELEMENTS

	S-Wave
Initial Time, T_o (ms):	0.025
Final Time, T_f (ms):	0.081
Travel Time (ms):	0.056
Wave Velocity (m/s):	381
Shear Modulus, G (MPa)	295.4
Vertical Effective Stress, s'_v (kPa)	1000
Maximum to Present Stress Ratio	1.0

WAVE TRAVEL CURVES



Comments: *Vs is based on assessment of first shear wave arrival

Test performed on sample consolidated to 1100kPa and tested at a CSR=0.21 for 3 cycles

Prepared By:	PS	Checked By:	SR	Approved By:	EP
Date:	December 7, 2015	Date:	December 7, 2015	Date:	December 8, 2015

MEG Technical Services (MTS)

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Form Nº MTS217a

Marine + Earth MEG Geosciences

Project:	AMECFW - Myra Falls	**Borehole:**	BH15-32A	**Project No.:**	15-MTS-029
Client:	AMEC	**Sample No.:**	S3	**Date:**	November 24, 2015
Location:	BC	**Depth (m):**	60.10	**Station:**	DSS 1

Bender Element Velocity Measuring

WATER CONTENT & UNIT WEIGHT

	Initial	Final
Tin No.:	11	2
Wt. of Tin (g):	32.90	207.65
Wet Weight (g):	127.57	255.63
Dry Weight (g):	109.91	247.27
Water Content (%):	22.9	21.1
Total Unit Weight (kN/m^3):	20.06	21.71
Dry Unit Weight (kN/m^3):	16.31	17.93

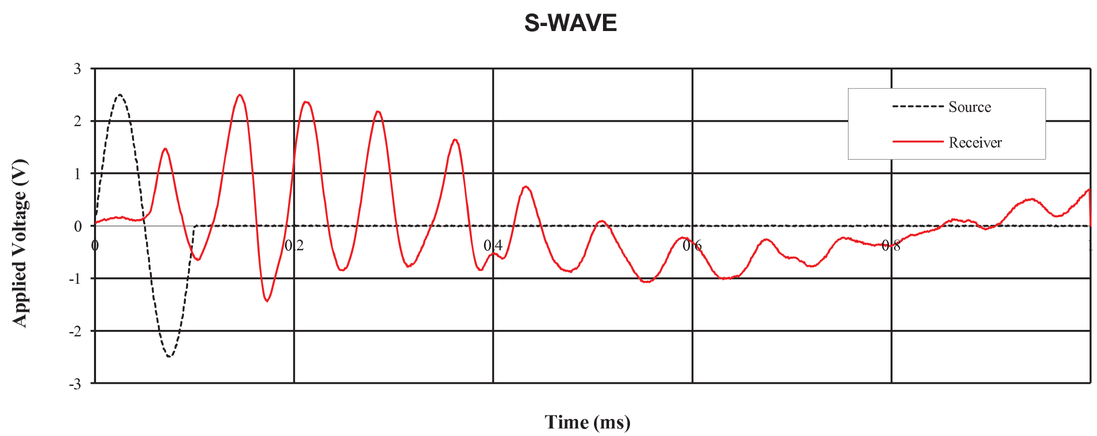
CONSOLIDATION

Specific Gravity, Gs:	2.81
Initial DSS Sample Height (mm):	23.5
Height after Consolidation (mm):	21.4
Initial Void Ratio, e_o	0.69
Initial Degree of Saturation (%)	93.4
Final Void Ratio, e_f	0.54
Final Degree of Saturation (%)	>100

BENDER ELEMENTS

	S-Wave
Initial Time, T_o (ms):	0.024
Final Time, T_f (ms):	0.072
Travel Time (ms):	0.048
Wave Velocity (m/s):	446
Shear Modulus, G (MPa)	405.9
Vertical Effective Stress, s'_v (kPa)	1100
Maximum to Present Stress Ratio	1.0

WAVE TRAVEL CURVES



Comments: *Vs is based on assessment of first shear wave arrival

Test performed on sample consolidated to 1100kPa and tested at a CSR=0.12 for 54 cycles

Prepared By:	PS	Checked By:	SR	Approved By:	EP
Date:	November 24, 2015	Date:	November 24, 2015	Date:	November 27, 2015

MEG Technical Services (MTS)

(A Division of MEG Consulting Limited)

Form Nº MTS210

Marine + Earth Geosciences

Project:	AMECFW - Myra Falls			Project No.:	15-MTS-029
Location:	BC	Borehole:	BH15-32A	Depth:	60.10 m
Sample:	S3	Station:	DSS 1	Date:	March 10, 2016

Stress Controlled Cyclic Direct Simple Shear Test

0.12 stress ratio (τ_{cyc}/ σ'_{vc}) @ 0.1 Hz for 54 cycles, σ'_{vc}=1100kPa

Initial sample Details		Final Sample Details	
Water Content (%):	22.9	Water Content (%):	21.1
Diameter (mm):	73.23	Diameter (mm):	73.23
Height (mm):	23.50	Change in Height, ΔH (mm):	2.12
Specific Gravity, Gs:	2.81	Final Height (mm):	21.38
Weight of Soil (g):	202.35	Weight of Soil (g):	199.34
Total Unit Weight (kN/m^3)	20.06	Total Unit Weight (kN/m^3)	21.71
Dry Unit Weight (kN/m^3)	16.31	Dry Unit Weight (kN/m^3)	17.93
Initial Void Ratio	0.69	Final Void Ratio	0.54

0.21 stress ratio (τ_{cyc}/ σ'_{vc}) @ 0.1 Hz for 3 cycles, σ'_{vc}=1000kPa

Initial sample Details		Final Sample Details	
Water Content (%):	26.2	Water Content (%):	23.5
Diameter (mm):	73.20	Diameter (mm):	73.20
Height (mm):	23.51	Change in Height, ΔH (mm):	2.16
Specific Gravity, Gs:	2.81	Final Height (mm):	21.35
Weight of Soil (g):	200.96	Weight of Soil (g):	196.62
Total Unit Weight (kN/m^3)	19.93	Total Unit Weight (kN/m^3)	21.47
Dry Unit Weight (kN/m^3)	15.79	Dry Unit Weight (kN/m^3)	17.38
Initial Void Ratio	0.75	Final Void Ratio	0.59

0.17 stress ratio (τ_{cyc}/ σ'_{vc}) @ 0.1 Hz for 10 cycles, σ'_{vc}=1000kPa

Initial sample Details		Final Sample Details	
Water Content (%):	23.4	Water Content (%):	19.7
Diameter (mm):	73.17	Diameter (mm):	73.17
Height (mm):	23.50	Change in Height, ΔH (mm):	1.95
Specific Gravity, Gs:	2.81	Final Height (mm):	21.55
Weight of Soil (g):	205.97	Weight of Soil (g):	199.72
Total Unit Weight (kN/m^3)	20.45	Total Unit Weight (kN/m^3)	21.62
Dry Unit Weight (kN/m^3)	16.57	Dry Unit Weight (kN/m^3)	18.07
Initial Void Ratio	0.67	Final Void Ratio	0.53

Sample Description:

Prepared By:	MF	Checked By:	PS	Approved By	JPS
Date:	March 10, 2016	Date:	March 10, 2016	Date:	March 11, 2016

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Marine + Earth
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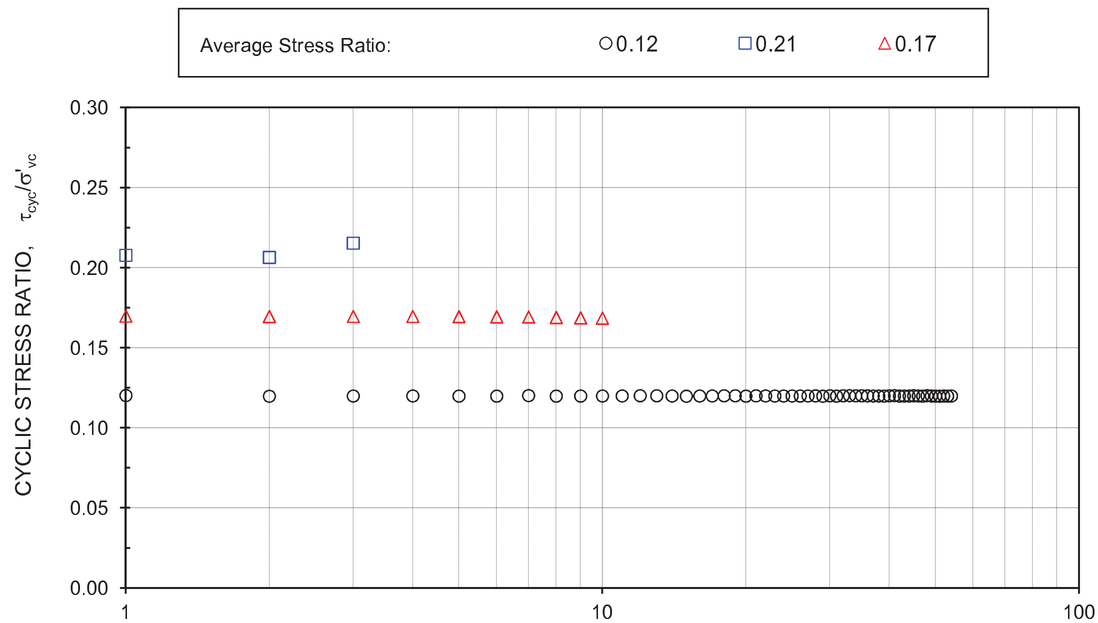
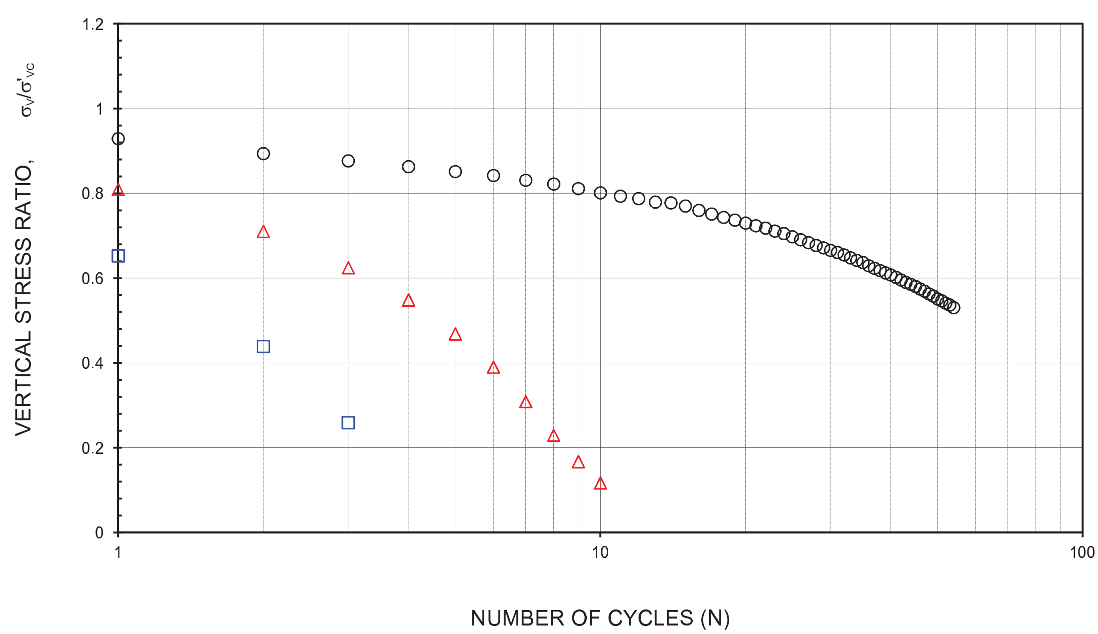
Project:	AMECFW - Myra Falls			Project No.:	15-MTS-029
Location:	BC	Borehole:	BH15-32A	Depth:	60.10 m
Sample:	S3	Station:	DSS 1	Date:	March 10, 2016

STRESS CONTROLLED CYCLIC DIRECT SIMPLE SHEAR TEST





Prepared By:	MF	Checked By:	PS	Approved By:	JPS
Date:	March 10, 2016	Date:	March 10, 2016	Date:	March 11, 2016

MEG TECHNICAL SERVICES
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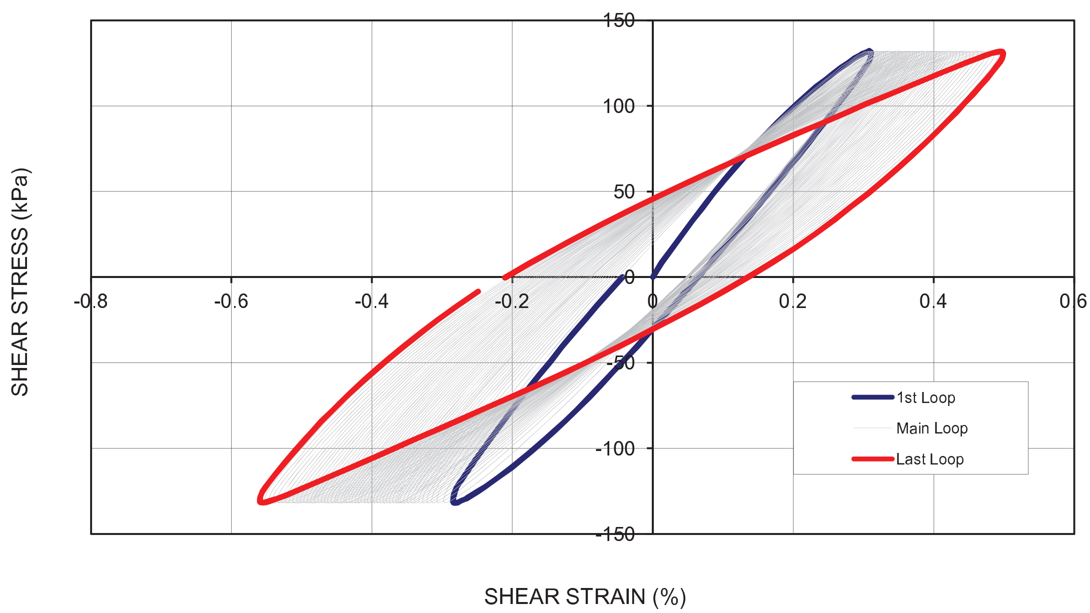
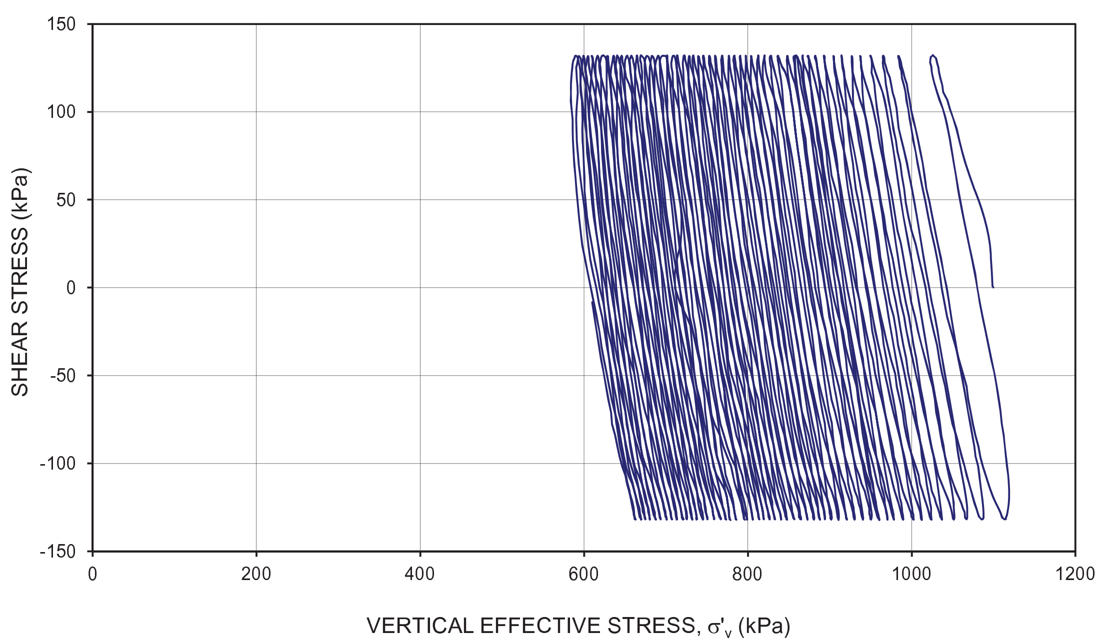
Marine + Earth Geosciences

Project:	AMECFW - Myra Falls			Project No.:	15-MTS-029
Location:	BC	Borehole:	BH15-32A	Depth:	60.10 m
Sample:	S3	Station:	DSS 1	Date:	March 10, 2016

STRESS CONTROLLED CYCLIC DIRECT SIMPLE SHEAR TEST

0.12 stress ratio (τ_{cyc}/ σ'_{vc}) @ 0.1 Hz for 54 cycles, σ'_{vc}=1100 kPa





Prepared By:	MF	Checked By:	PS	Approved By:	JPS
Date:	March 10, 2016	Date:	March 10, 2016	Date:	March 11, 2016

MEG TECHNICAL SERVICES

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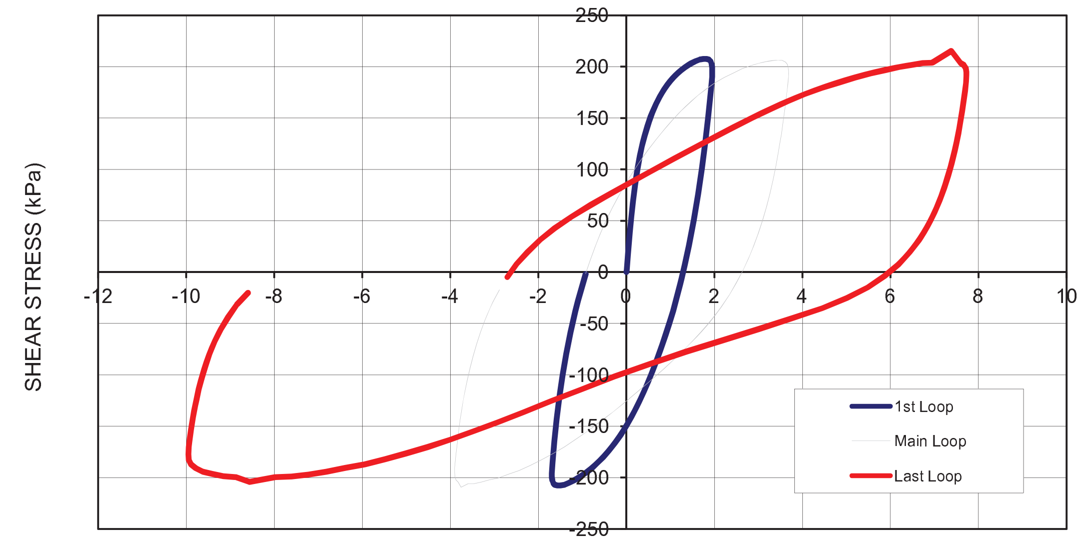
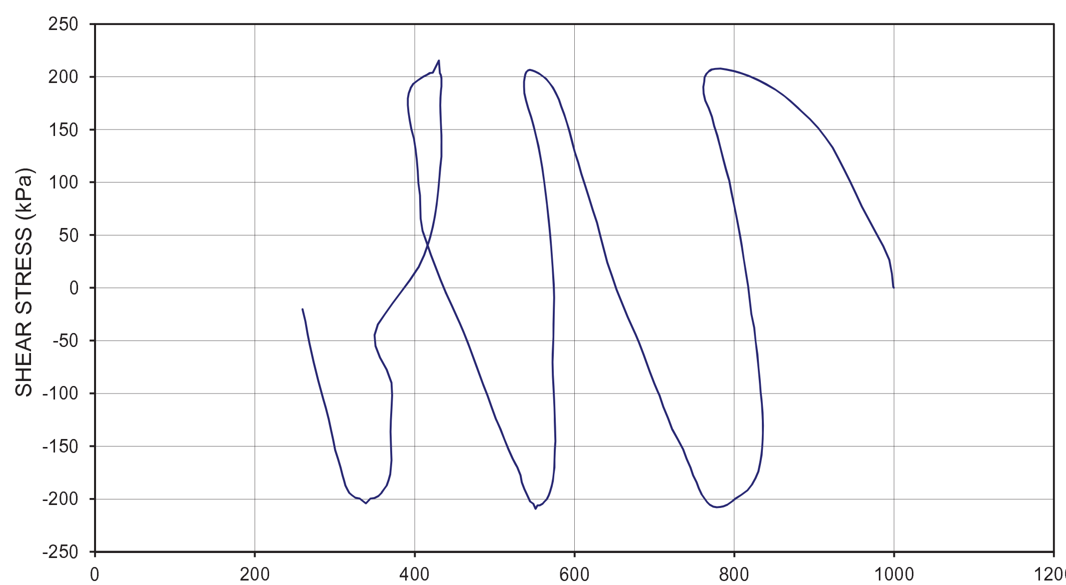
Project:	AMECFW - Myra Falls			Project No.:	15-MTS-029
Location:	BC	Borehole:	BH15-32A	Depth:	60.10 m
Sample:	S3	Station:	DSS 1	Date:	March 10, 2016

STRESS CONTROLLED CYCLIC DIRECT SIMPLE SHEAR TEST

0.21 stress ratio (τ_{cyc}/σ'_{vc}) @ 0.1 Hz for 3 cycles, σ'_{vc}=1000kPa



SHEAR STRAIN (%)



VERTICAL EFFECTIVE STRESS, σ'_v (kPa)

Prepared By:	MF	Checked By:	PS	Approved By:	JPS
Date:	March 10, 2016	Date:	March 10, 2016	Date:	March 11, 2016

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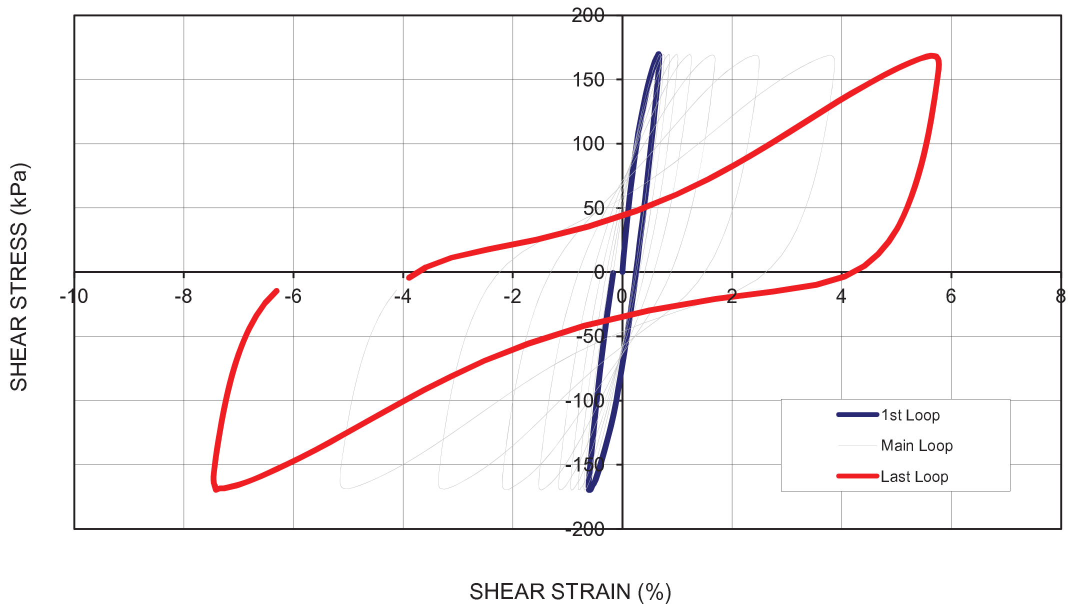
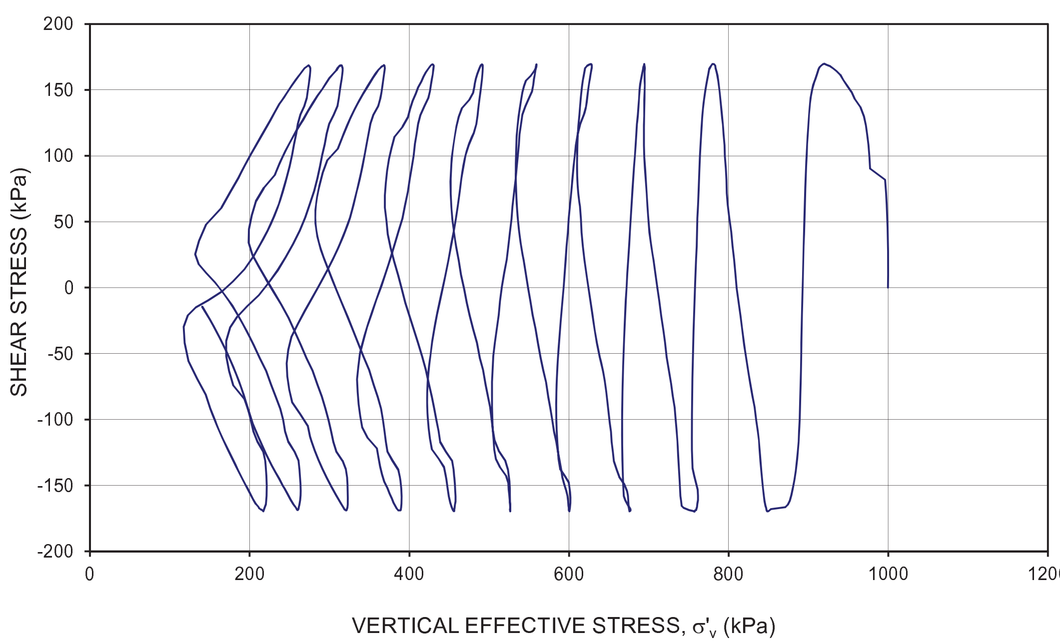
Project:	AMECFW - Myra Falls			Project No.:	15-MTS-029
Location:	BC	Borehole:	BH15-32A	Depth:	60.10 m
Sample:	S3	Station:	DSS 1	Date:	March 10, 2016

STRESS CONTROLLED CYCLIC DIRECT SIMPLE SHEAR TEST

0.17 stress ratio (τ_{cyc}/ σ'_{vc}) @ 0.1 Hz for 10 cycles, σ'_{vc}=1000kPa





Prepared By:	MF	Checked By:	PS	Approved By:	JPS
Date:	March 10, 2016	Date:	March 10, 2016	Date:	March 11, 2016

MEG Technical Services (MTS)
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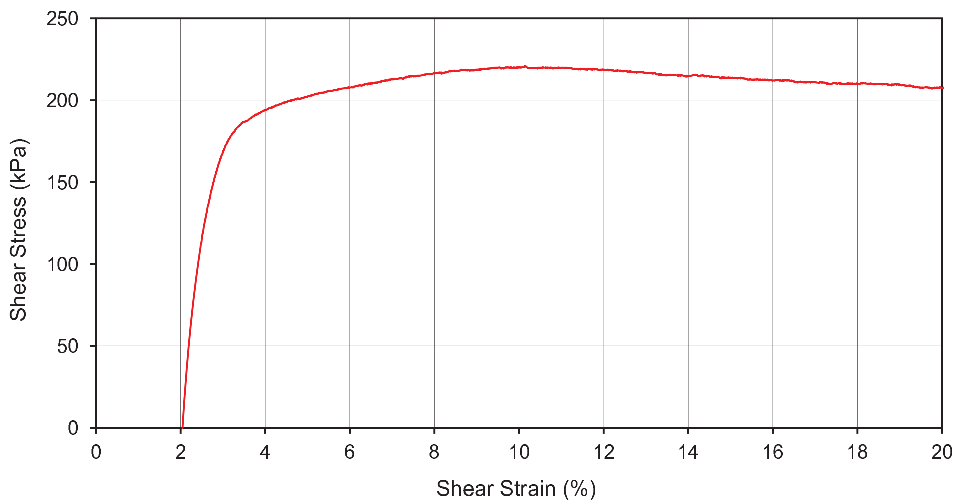
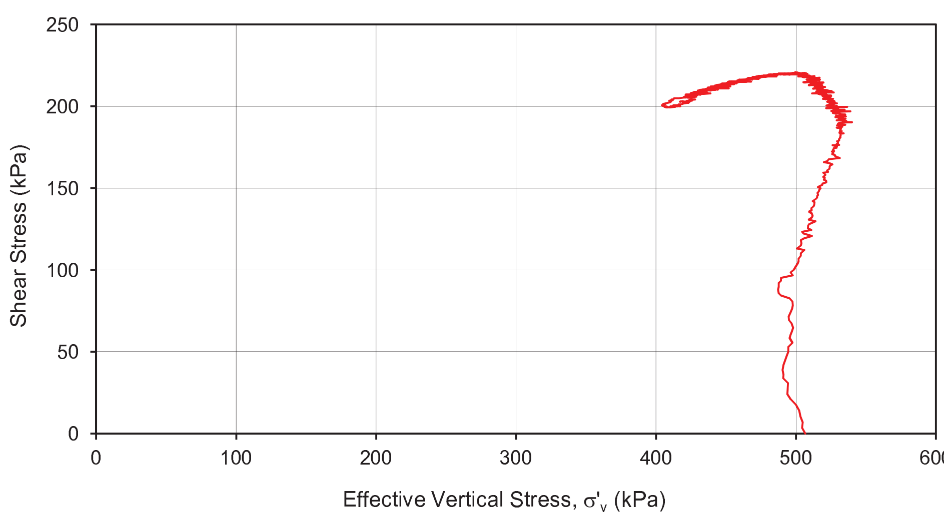
Form N° MTS214

Project:	AMECFW - Myra Falls			Project No.:	15-MTS-029
Location:	BC	Borehole:	BH15-32A	Depth:	60.10 m
Sample:	S3	Station:	DSS 1	Date:	March 10, 2016

Post-Cyclic Static Direct Simple Shear Test

POST-CYCLIC STATIC SHEAR TEST





Note: Test performed after stress-controlled DSS test at average cyclic stress ratio, CSR = 0.12 with 47% excess pore pressure

Prepared By:	MF	Checked By:	PS	Approved By:	JPS
Date:	March 10, 2016	Date:	March 10, 2016	Date:	March 11, 2016

MEG Technical Services (MTS)
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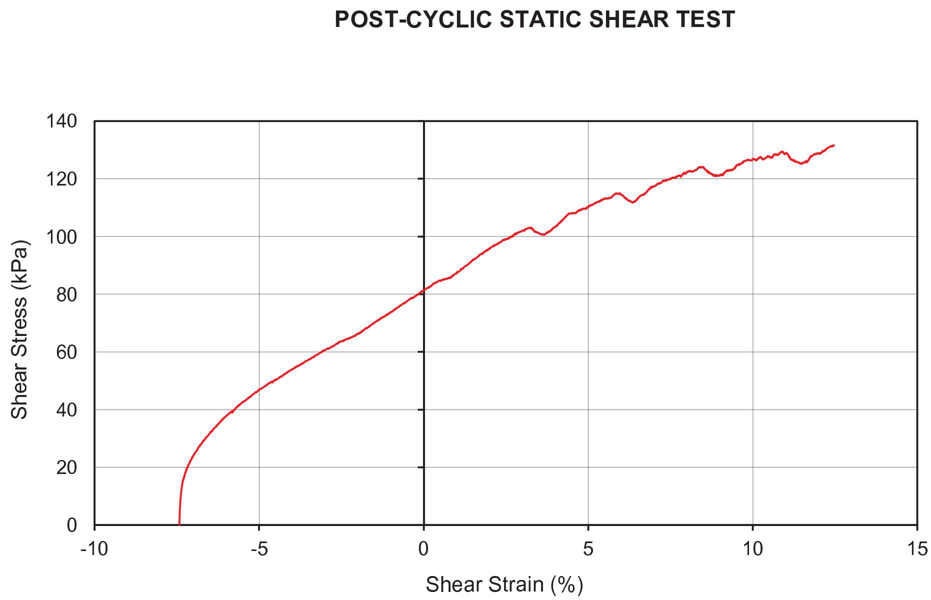
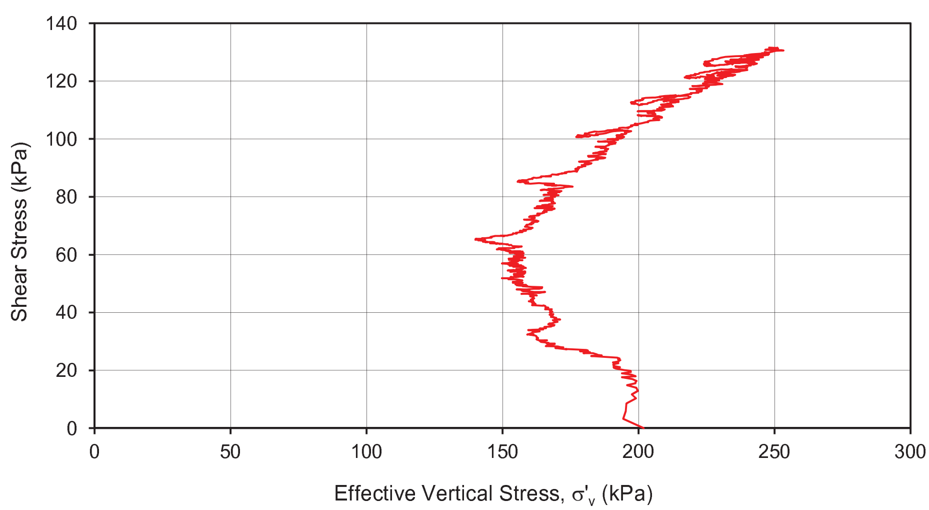
Project:	AMECFW - Myra Falls			Project No.:	15-MTS-029
Location:	BC	Borehole:	BH15-32A	Depth:	60.10 m
Sample:	S3	Station:	DSS 1	Date:	March 10, 2016

Post-Cyclic Static Direct Simple Shear Test





Note: Test performed after stress-controlled DSS test at average cyclic stress ratio, CSR = 0.21 with 74% excess pore pressure

Prepared By:	MF	Checked By:	PS	Approved By:	JPS
Date:	March 10, 2016	Date:	March 10, 2016	Date:	March 11, 2016

MEG Technical Services (MTS)

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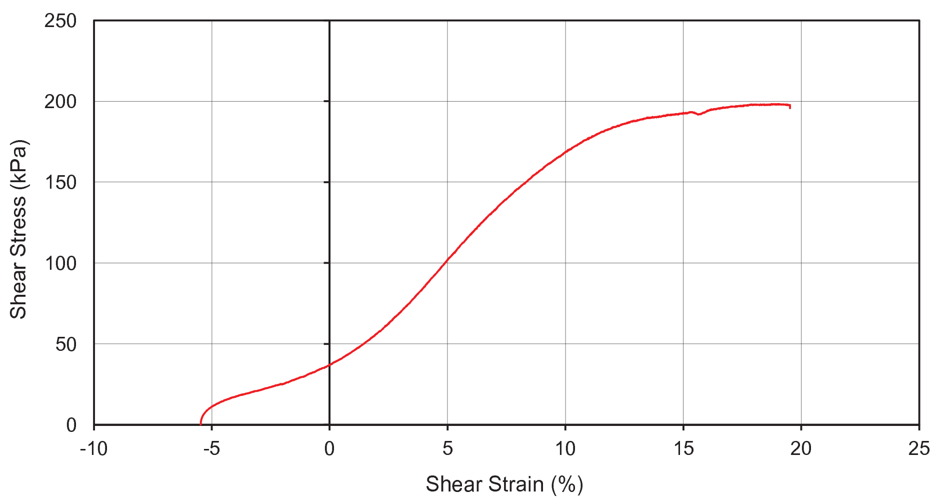
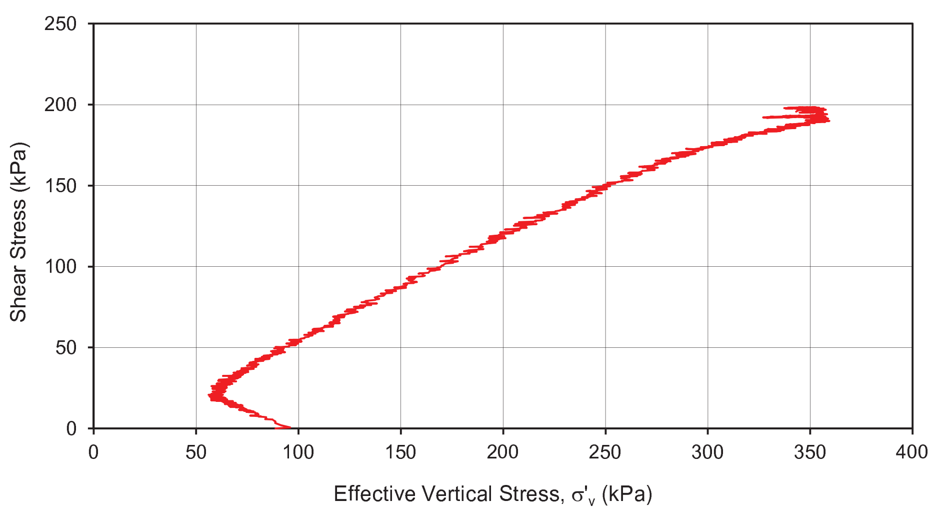
Marine + Earth MEG Geosciences

Project:	AMECFW - Myra Falls			Project No.:	15-MTS-029
Location:	BC	Borehole:	BH15-32A	Depth:	60.10 m
Sample:	S3	Station:	DSS 1	Date:	March 10, 2016

Post-Cyclic Static Direct Simple Shear Test

POST-CYCLIC STATIC SHEAR TEST





Note: Test performed after stress-controlled DSS test at average cyclic stress ratio, CSR = 0.17 with 88% excess pore pressure

Prepared By:	MF	Checked By:	PS	Approved By:	JPS
Date:	March 10, 2016	Date:	March 10, 2016	Date:	March 11, 2016

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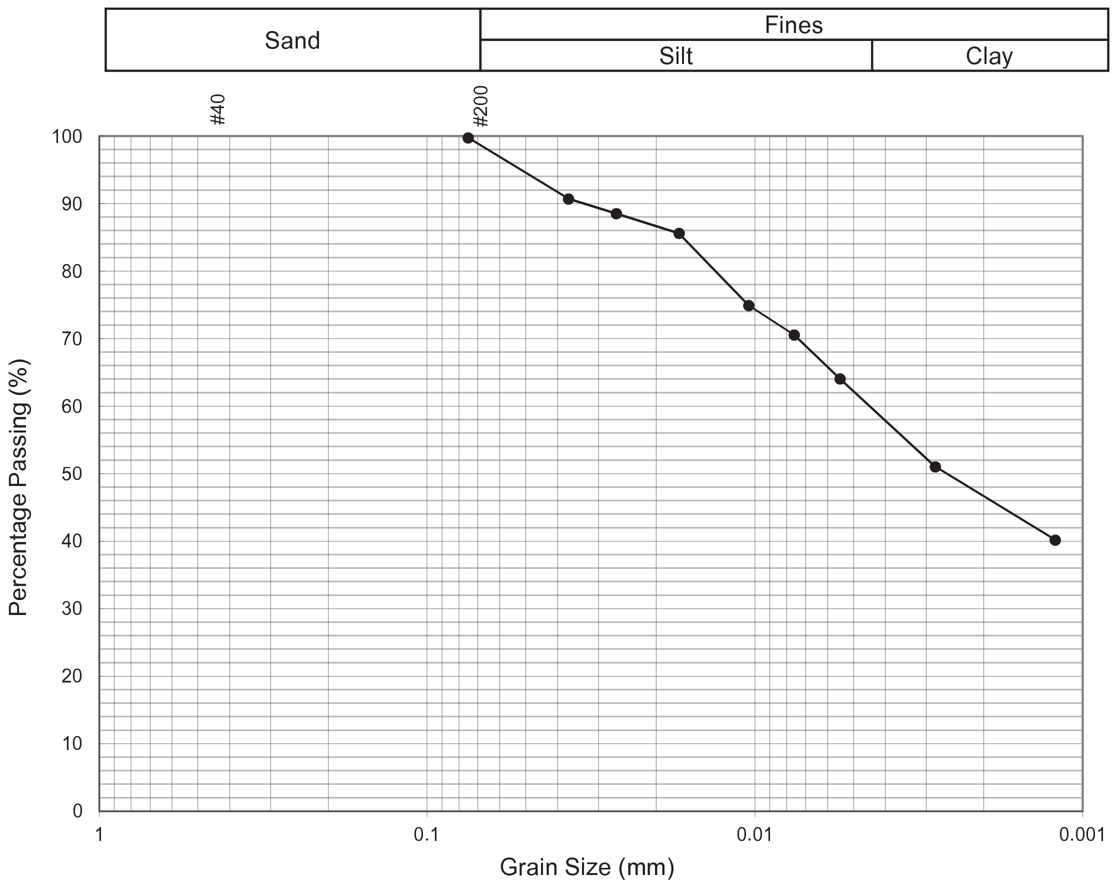
Form Nº MTS107

Project: AMECFW - Myra Falls		Project No.: 15-MTS-029	
Location: BC		Date: November 19, 2015	
Borehole: BH15-32A	Sample No.: S5	Depth (m): 61.30	

Hydrometer Test (ASTM D422)

Unified Soil Classification System (ASTM D 2487)

Description of Material: Pale green lean CLAY



Sample No.	Depth (m)	Percentage of Material by Weight (%)			
		Gravel	Sand	Fines	
				Silt	Clay
S5	61.30	-	0	38	62

Comments:

Prepared by: PC	Checked by: MF	Approved by: PS
Date: December 4, 2015	Date: December 8, 2015	Date: December 9, 2015

MEG Technical Services (MTS)

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Marine + Earth Geosciences

Form N° MTS104

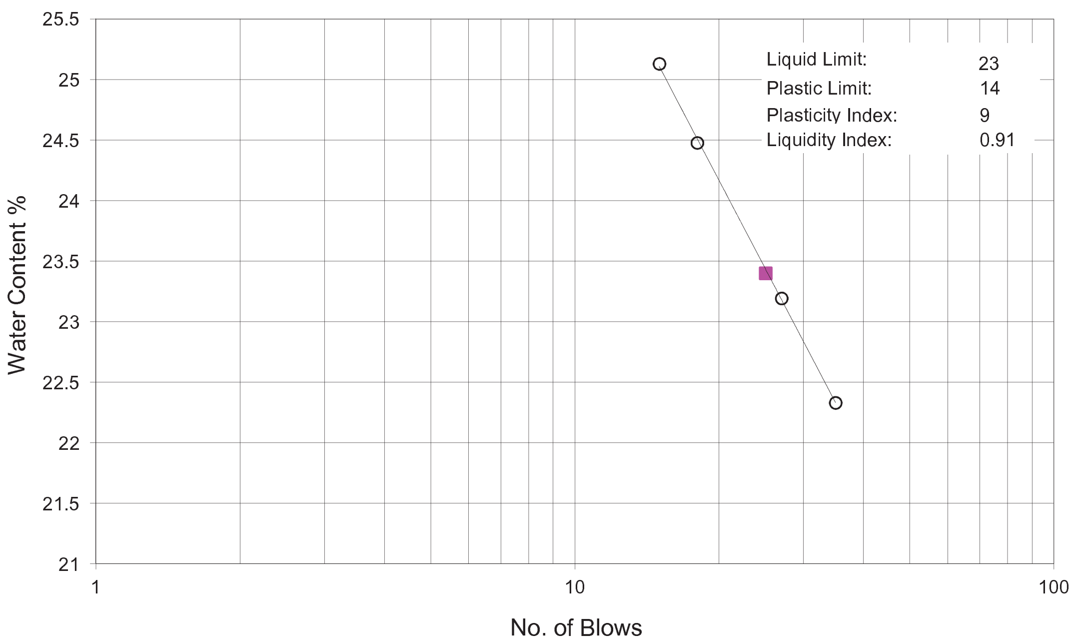
Project:	AMECFW - Myra Falls			Project No.:	15-MTS-029
Location:	BC			Date:	December 2, 2015
Borehole:	BH15-32A	Sample No.:	S5	Depth (m):	61.30

Liquid Limit, Plastic Limit and Plasticity Index of Soils (ASTM D4318)

LIQUID LIMIT								PLASTIC LIMIT						
TIN No.	Tare + Weight of Wet Soil (g)	Tare + Weight of Dry Soil (g)	Weight of Tin (g)	Weight of Water (g)	Weight of Dry Soil (g)	Water Content (%)	No. of Blows	TIN No.	Tare + Weight of Wet Soil (g)	Tare + Weight of Dry Soil (g)	Weight of Tin (g)	Weight of Water (g)	Weight of Dry Soil (g)	Water Content (%)
10A	44.34	42.50	34.26	1.84	8.24	22.3	35	30E	36.24	35.47	30.03	0.77	5.44	14.2
86	38.04	35.73	25.77	2.31	9.96	23.2	27	33E	36.36	35.55	29.97	0.81	5.58	14.5
68	45.18	43.19	35.06	1.99	8.13	24.5	18							
52A	35.33	32.92	23.33	2.41	9.59	25.1	15							

Classification of the material : CL

_____ % with respect to the total of the material smaller than sieve No. 40



Observations:

Prepared by:	PC	Checked by:	MF	Approved by:	PS
Date:	December 2, 2015	Date:	December 3, 2015	Date:	December 4, 2015

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MEG Technical Services (MTS)

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Form Nº MTS217a

Marine + Earth MEG Geosciences

Project:	AMECFW - Myra Falls	**Borehole:**	BH15-32A	**Project No.:**	15-MTS-029
Client:	AMEC	**Sample No.:**	S5	**Date:**	November 23, 2015
Location:	BC	**Depth (m):**	61.30	**Station:**	DSS 1

Bender Element Velocity Measuring

WATER CONTENT & UNIT WEIGHT

	Initial	Final
Tin No.:	3	D1
Wt. of Tin (g):	25.50	116.91
Wet Weight (g):	62.65	308.91
Dry Weight (g):	55.89	278.31
Water Content (%):	22.2	19.0
Total Unit Weight (kN/m^3):	20.50	21.42
Dry Unit Weight (kN/m^3):	16.78	18.01

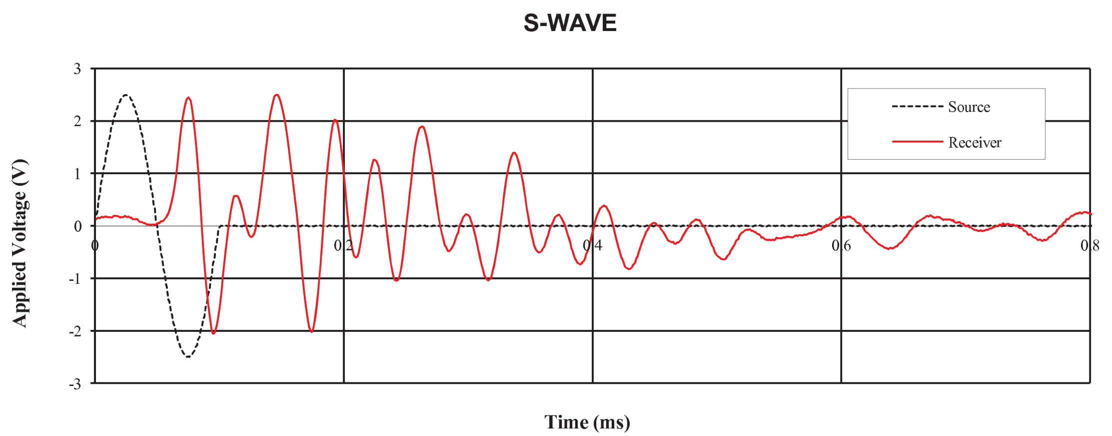
CONSOLIDATION

Specific Gravity, Gs:	2.81
Initial DSS Sample Height (mm):	23.6
Height after Consolidation (mm):	21.9
Initial Void Ratio, e_o	0.64
Initial Degree of Saturation (%)	97.0
Final Void Ratio, e_f	0.53
Final Degree of Saturation (%)	82.7

BENDER ELEMENTS

	S-Wave
Initial Time, T_o (ms):	0.024
Final Time, T_f (ms):	0.074
Travel Time (ms):	0.050
Wave Velocity (m/s):	439
Shear Modulus, G (MPa)	402.3
Vertical Effective Stress, s'_v (kPa)	1100
Maximum to Present Stress Ratio	1.0

WAVE TRAVEL CURVES



Comments: *Vs is based on assessment of first shear wave arrival

Prepared By:	PS	Checked By:	SR	Approved By:	EP
Date:	November 23, 2015	Date:	November 23, 2015	Date:	November 23, 2015

MEG Technical Services (MTS)

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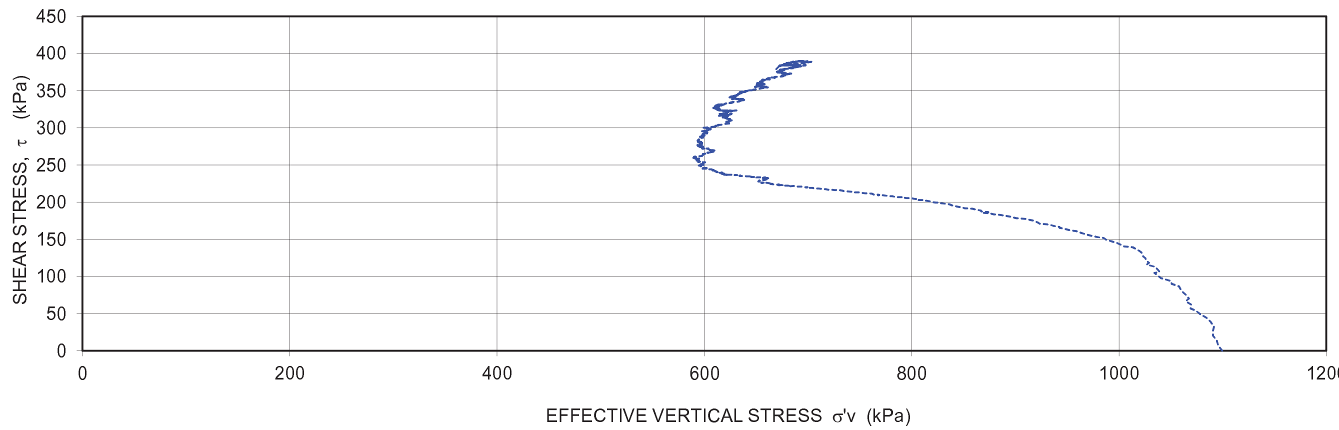
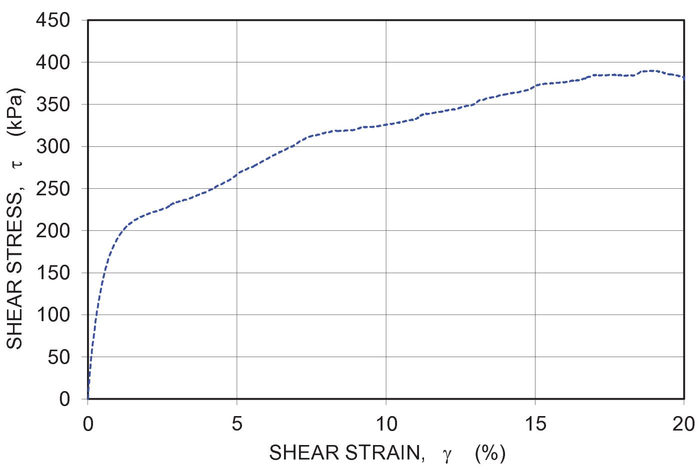
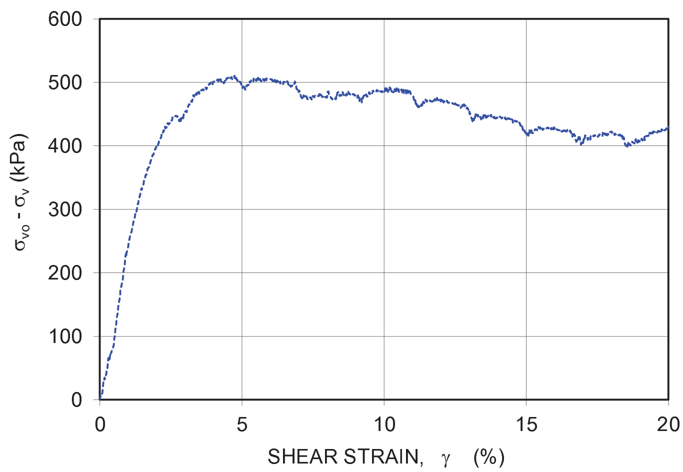
Project:	AMECFW - Myra Falls	Project No.:	15-MTS-029
Location:	BC	Date:	November 16, 2015
Borehole:	BH15-32A	Depth (m):	61.37
Sample No.:	S5		

Direct Simple Shear (ASTM D6528)

Initial Height (mm):	23.6	**Weight of Specimen (g):**	208.86	**Initial Void Ratio, e_o:**	0.62
Diameter of Ring (mm):	73.1	**Total Unit Weight (kN/m^3):**	20.66	**Final Void Ratio, e_f:**	0.53
Specific Gravity, Gs:	2.81	**Dry Unit Weight (kN/m^3):**	16.99	**Natural Water Content (%):**	21.6
Final Water Content (%):	17.8	**Initial Degree of Saturation, Sr (%):**	97.4	**Final Degree of Saturation, Sr (%):**	94.6







Type of Test: Constant Volume					
Sample No.	Depth (m)	Total Unit Weight (kN/m^3)	Effective Vertical Stress, σ'v (kPa)	Strain Rate (%/hour)	Test OCR
S5	61.37	20.7	1100	5	-

Comments:

Prepared By:	MF	Checked By:	PS	Approved By:	EP
Date:	November 16, 2015	Date:	November 16, 2015	Date:	November 23, 2016

OQM | Organizational Quality Management Program

MEG Technical Services (MTS)

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Form Nº MTS210

Project:	AMECFW - Myra Falls			Project No.:	15-MTS-029
Location:	BC	Borehole:	BH15-32A	Depth:	61.30 m
Sample:	S5	Station:	DSS 1	Date:	March 10, 2016

Stress Controlled Cyclic Direct Simple Shear Test

0.10 stress ratio (τ_{cyc}/ σ'_{vc}) @ 0.1 Hz for 30 cycles, σ'_{vc}=1000kPa

Initial sample Details		Final Sample Details	
Water Content (%):	23.3	Water Content (%):	21.2
Diameter (mm):	73.26	Diameter (mm):	73.26
Height (mm):	23.57	Change in Height, ΔH (mm):	2.16
Specific Gravity, Gs:	2.81	Final Height (mm):	21.41
Weight of Soil (g):	203.19	Weight of Soil (g):	199.75
Total Unit Weight (kN/m^3)	20.06	Total Unit Weight (kN/m^3)	21.71
Dry Unit Weight (kN/m^3)	16.27	Dry Unit Weight (kN/m^3)	17.91
Initial Void Ratio	0.70	Final Void Ratio	0.54

0.14 stress ratio (τ_{cyc}/ σ'_{vc}) @ 0.1 Hz for 30 cycles, σ'_{vc}=1000kPa

Initial sample Details		Final Sample Details	
Water Content (%):	23.9	Water Content (%):	20.0
Diameter (mm):	73.19	Diameter (mm):	73.19
Height (mm):	23.52	Change in Height, ΔH (mm):	1.89
Specific Gravity, Gs:	2.81	Final Height (mm):	21.62
Weight of Soil (g):	206.40	Weight of Soil (g):	199.78
Total Unit Weight (kN/m^3)	20.47	Total Unit Weight (kN/m^3)	21.54
Dry Unit Weight (kN/m^3)	16.51	Dry Unit Weight (kN/m^3)	17.96
Initial Void Ratio	0.67	Final Void Ratio	0.54

0.18 stress ratio (τ_{cyc}/ σ'_{vc}) @ 0.1 Hz for 6 cycles, σ'_{vc}=1100kPa

Initial sample Details		Final Sample Details	
Water Content (%):	22.2	Water Content (%):	19.0
Diameter (mm):	73.28	Diameter (mm):	73.28
Height (mm):	23.55	Change in Height, ΔH (mm):	1.61
Specific Gravity, Gs:	2.81	Final Height (mm):	21.94
Weight of Soil (g):	207.60	Weight of Soil (g):	202.02
Total Unit Weight (kN/m^3)	20.50	Total Unit Weight (kN/m^3)	21.42
Dry Unit Weight (kN/m^3)	16.77	Dry Unit Weight (kN/m^3)	18.01
Initial Void Ratio	0.64	Final Void Ratio	0.53

Sample Description:

Prepared By:	MF	Checked By:	PS	Approved By:	JPS
Date:	March 10, 2016	Date:	March 10, 2016	Date:	March 11, 2016

MEG TECHNICAL SERVICES

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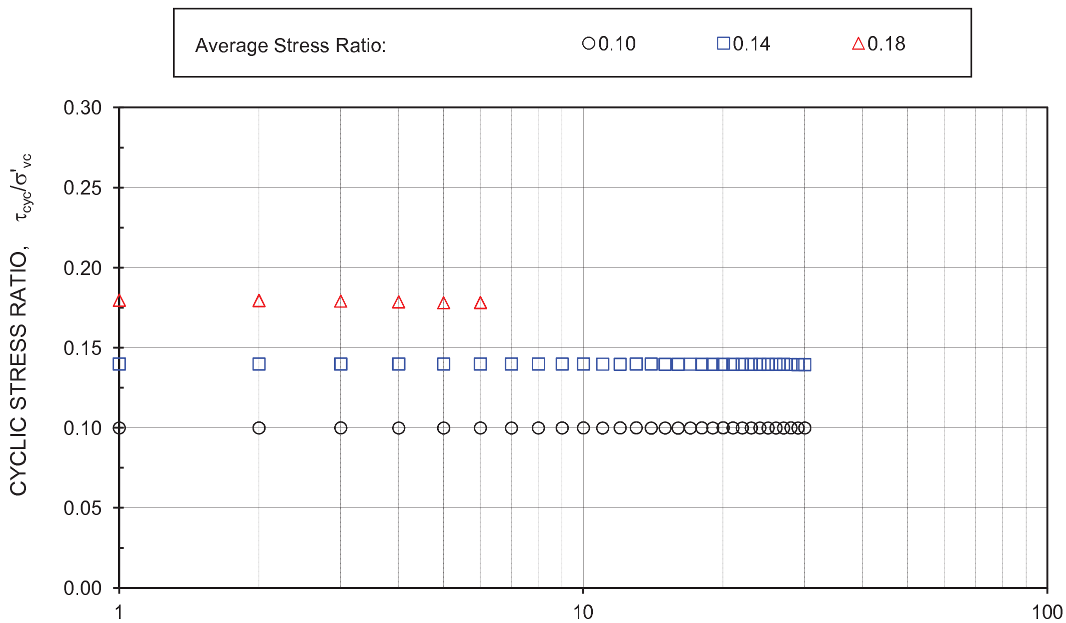
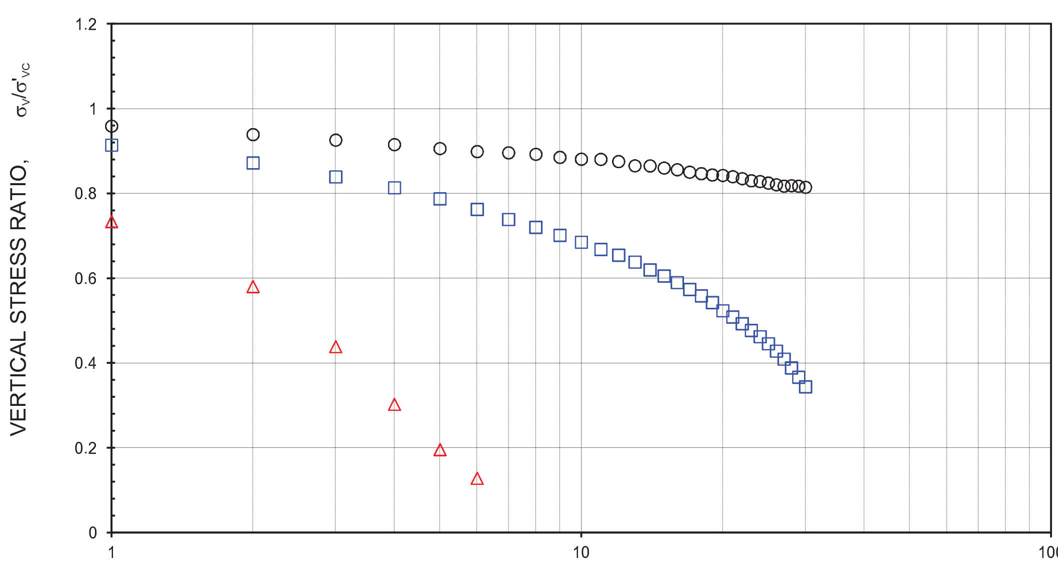
Marine + Earth Geosciences

Project:	AMECFW - Myra Falls			Project No.:	15-MTS-029
Location:	BC	Borehole:	BH15-32A	Depth:	61.30 m
Sample:	S5	Station:	DSS 1	Date:	March 10, 2016

STRESS CONTROLLED CYCLIC DIRECT SIMPLE SHEAR TEST





Prepared By:	MF	Checked By:	PS	Approved By:	JPS
Date:	March 10, 2016	Date:	March 10, 2016	Date:	March 11, 2016

MEG TECHNICAL SERVICES
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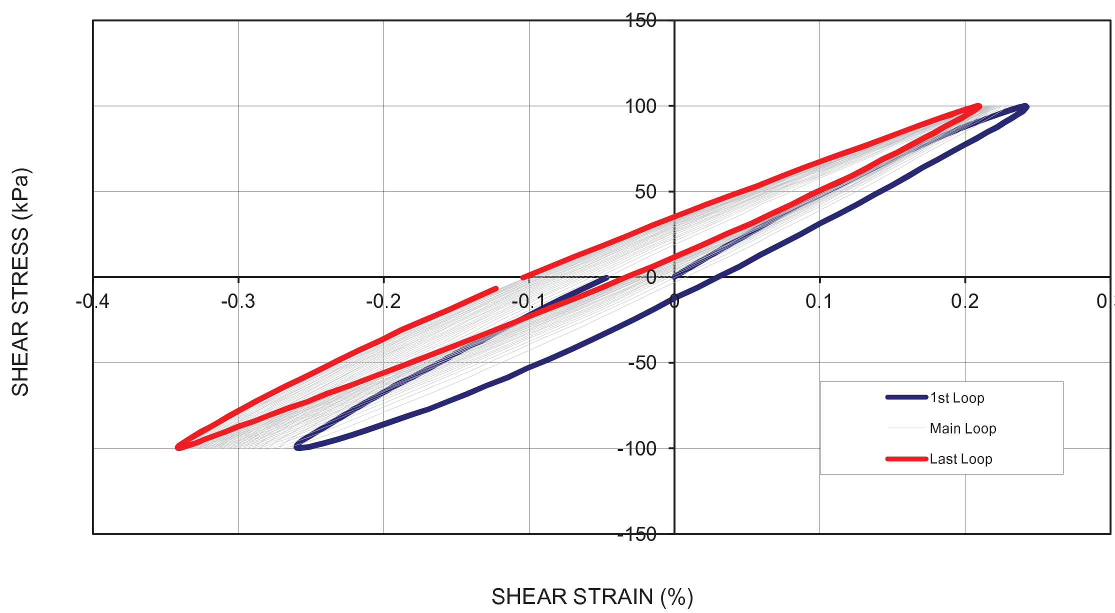
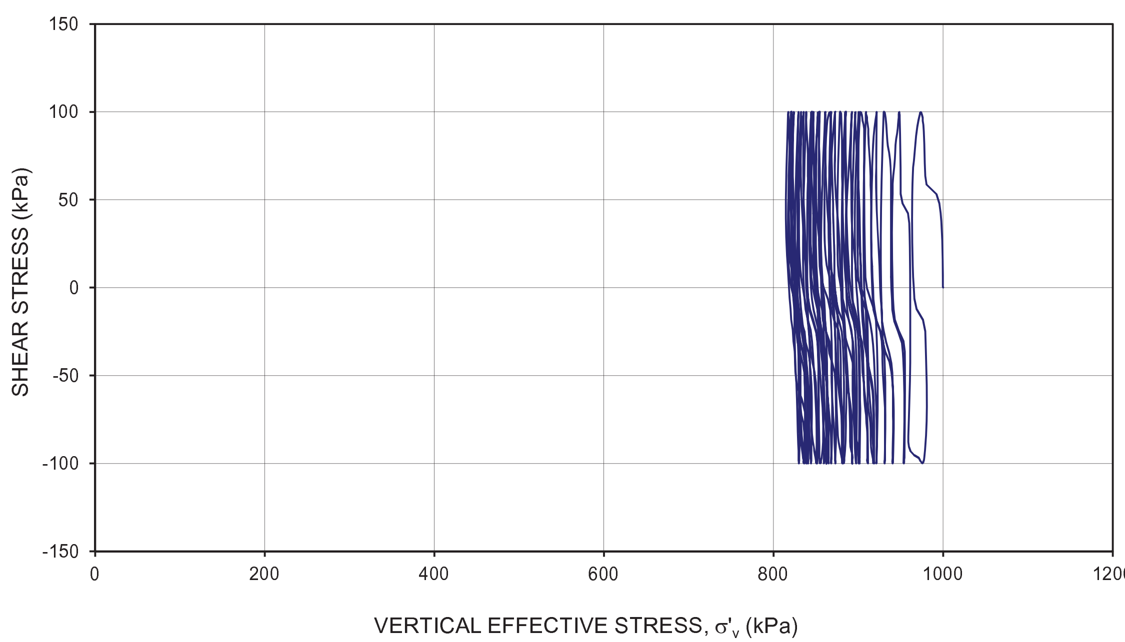
Project:	AMECFW - Myra Falls			Project No.:	15-MTS-029
Location:	BC	Borehole:	BH15-32A	Depth:	61.30 m
Sample:	S5	Station:	DSS 1	Date:	March 10, 2016

STRESS CONTROLLED CYCLIC DIRECT SIMPLE SHEAR TEST

0.10 stress ratio (τ_{cyc}/ σ'_{vc}) @ 0.1 Hz for 30 cycles, σ'_{vc}=1000kPa





Prepared By:	MF	Checked By:	PS	Approved By:	JPS
Date:	March 10, 2016	Date:	March 10, 2016	Date:	March 11, 2016

MEG TECHNICAL SERVICES
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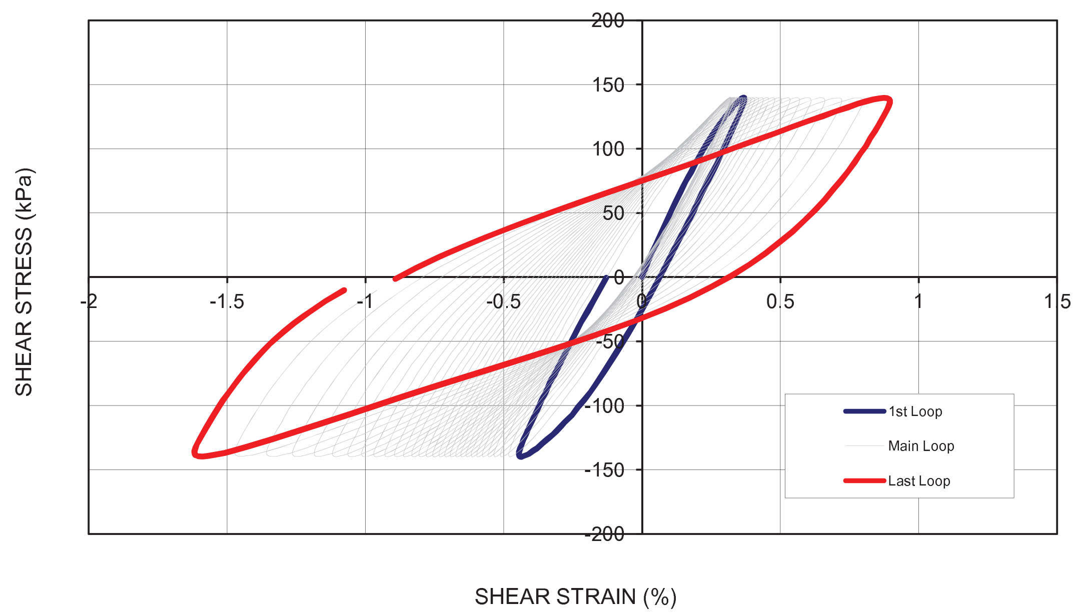
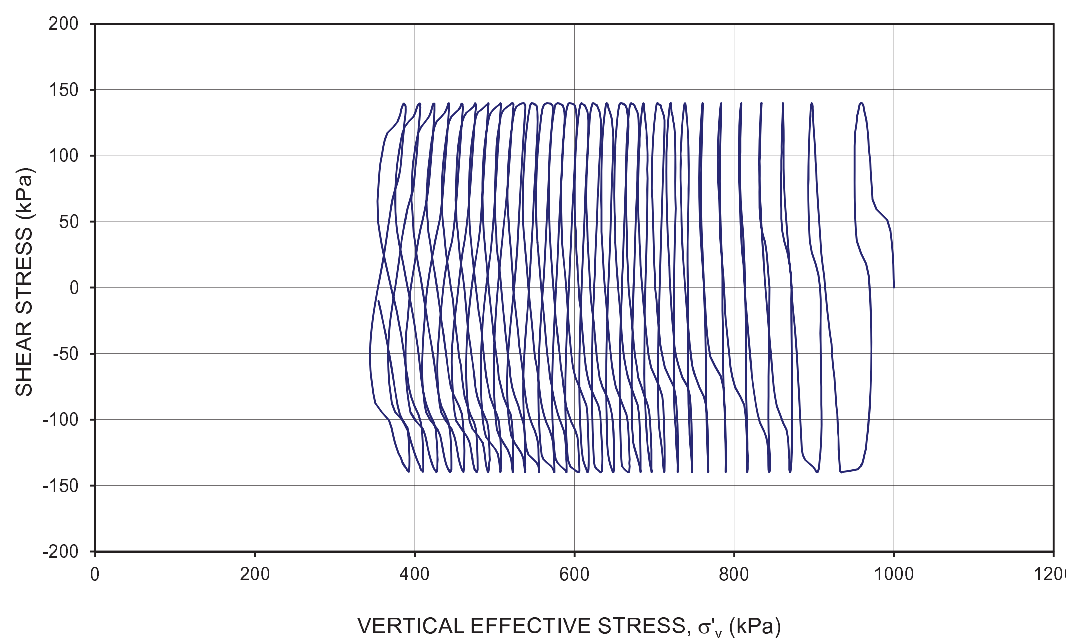
Project:	AMECFW - Myra Falls			Project No.:	15-MTS-029
Location:	BC	Borehole:	BH15-32A	Depth:	61.30 m
Sample:	S5	Station:	DSS 1	Date:	March 10, 2016

STRESS CONTROLLED CYCLIC DIRECT SIMPLE SHEAR TEST

0.14 stress ratio (τ_{cyc}/ σ'_{vc}) @ 0.1 Hz for 30 cycles, σ'_{vc}=1000kPa





Prepared By:	MF	Checked By:	PS	Approved By:	JPS
Date:	March 10, 2016	Date:	March 10, 2016	Date:	March 11, 2016

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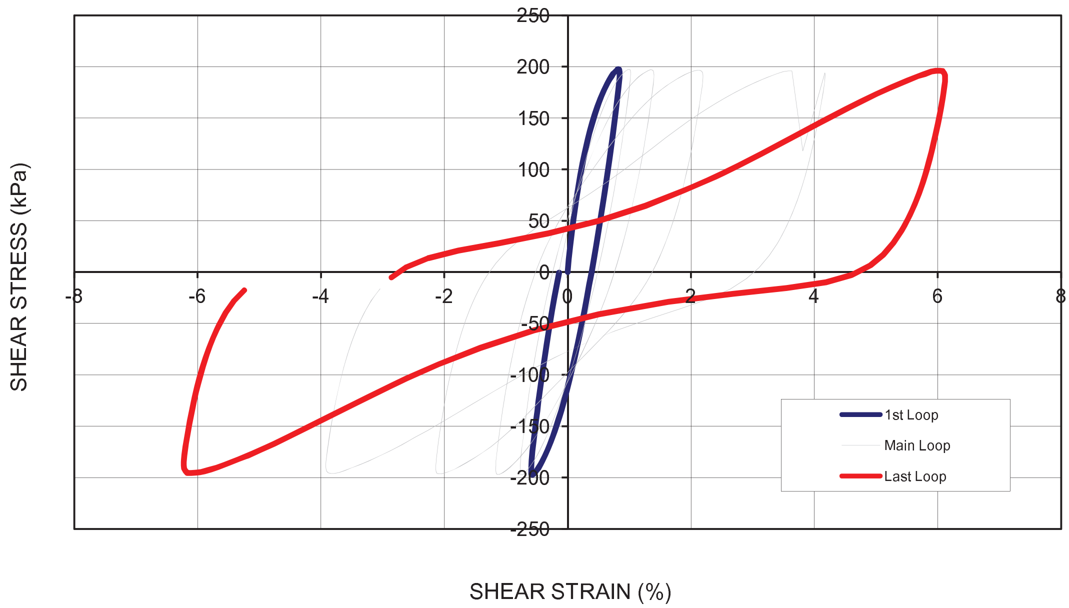
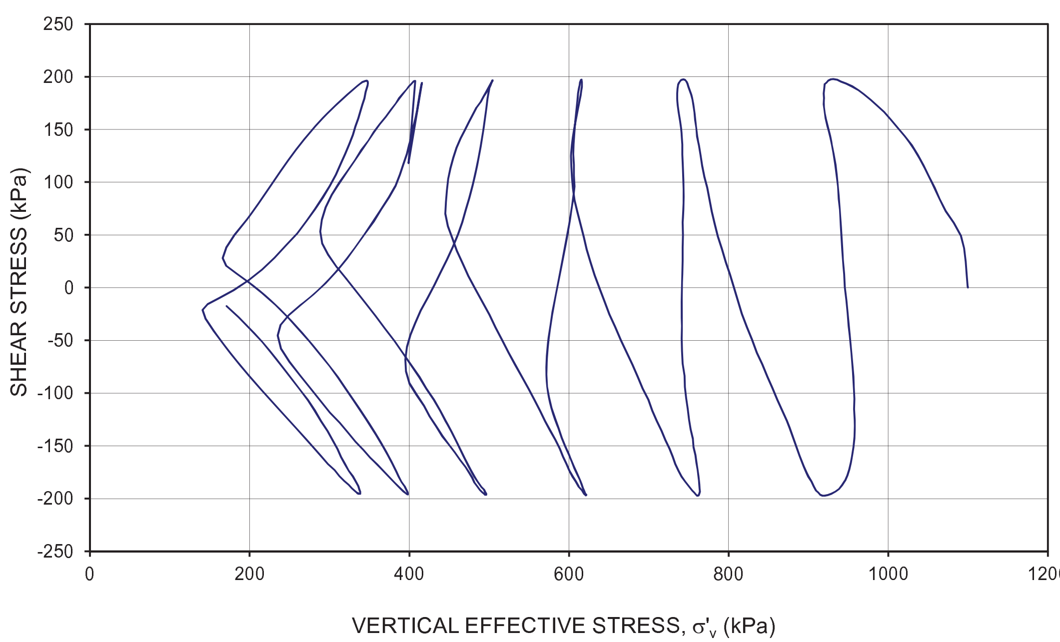
Project:	AMECFW - Myra Falls			Project No.:	15-MTS-029
Location:	BC	Borehole:	BH15-32A	Depth:	61.30 m
Sample:	S5	Station:	DSS 1	Date:	March 10, 2016

STRESS CONTROLLED CYCLIC DIRECT SIMPLE SHEAR TEST

0.18 stress ratio (τ_{cyc}/ σ'_{vc}) @ 0.1 Hz for 6 cycles, σ'_{vc}=1100kPa





Prepared By:	MF	Checked By:	PS	Approved By:	JPS
Date:	March 10, 2016	Date:	March 10, 2016	Date:	March 11, 2016

MEG Technical Services (MTS)

(A Division of MEG Consulting Limited)

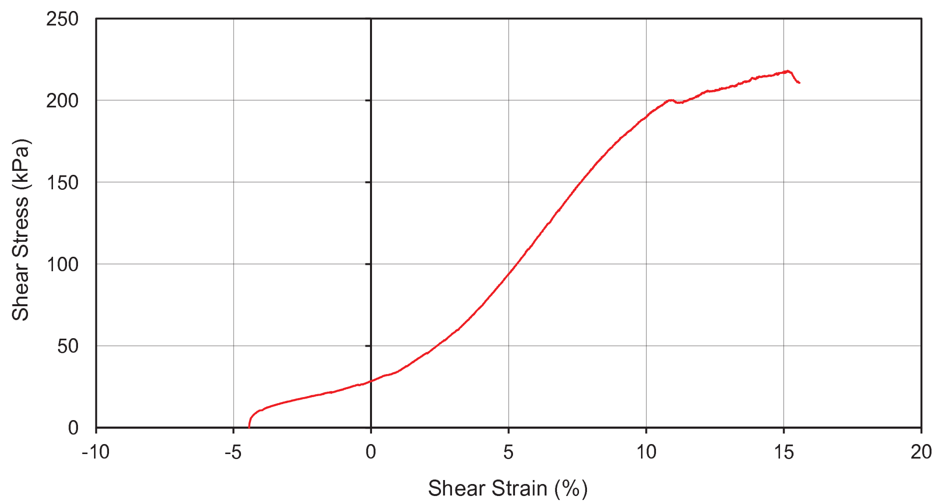
Form Nº MTS214

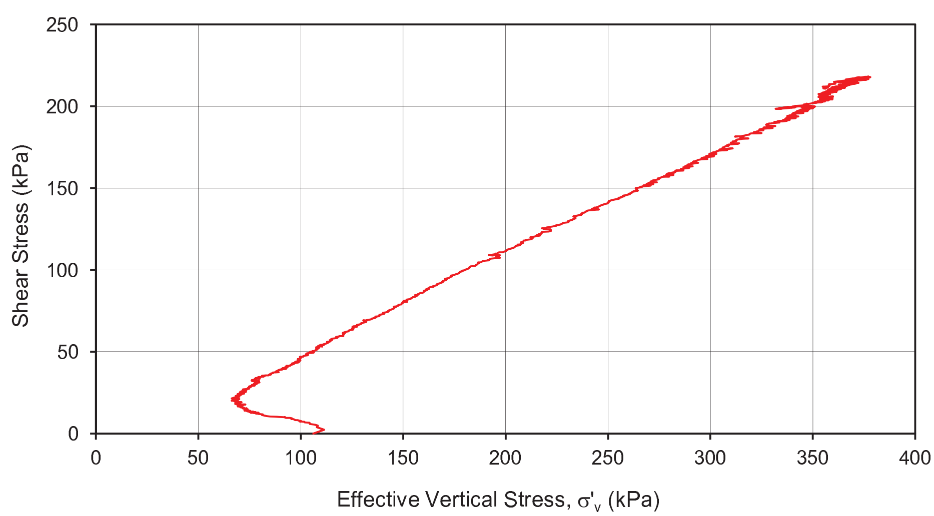
Marine + Earth MEG Geosciences

Project:	AMECFW - Myra Falls			Project No.:	15-MTS-029
Location:	BC	Borehole:	BH15-32A	Depth:	61.30 m
Sample:	S5	Station:	DSS 1	Date:	March 10, 2016

Post-Cyclic Static Direct Simple Shear Test

POST-CYCLIC STATIC SHEAR TEST





Note: Test performed after stress-controlled DSS test at average cyclic stress ratio, CSR = 0.18
with 87% excess pore pressure

Prepared By:	MF	Checked By:	PS	Approved By:	JPS
Date:	March 10, 2016	Date:	March 10, 2016	Date:	March 11, 2016

MEG Technical Services (MTS)

(A Division of MEG Consulting Limited)

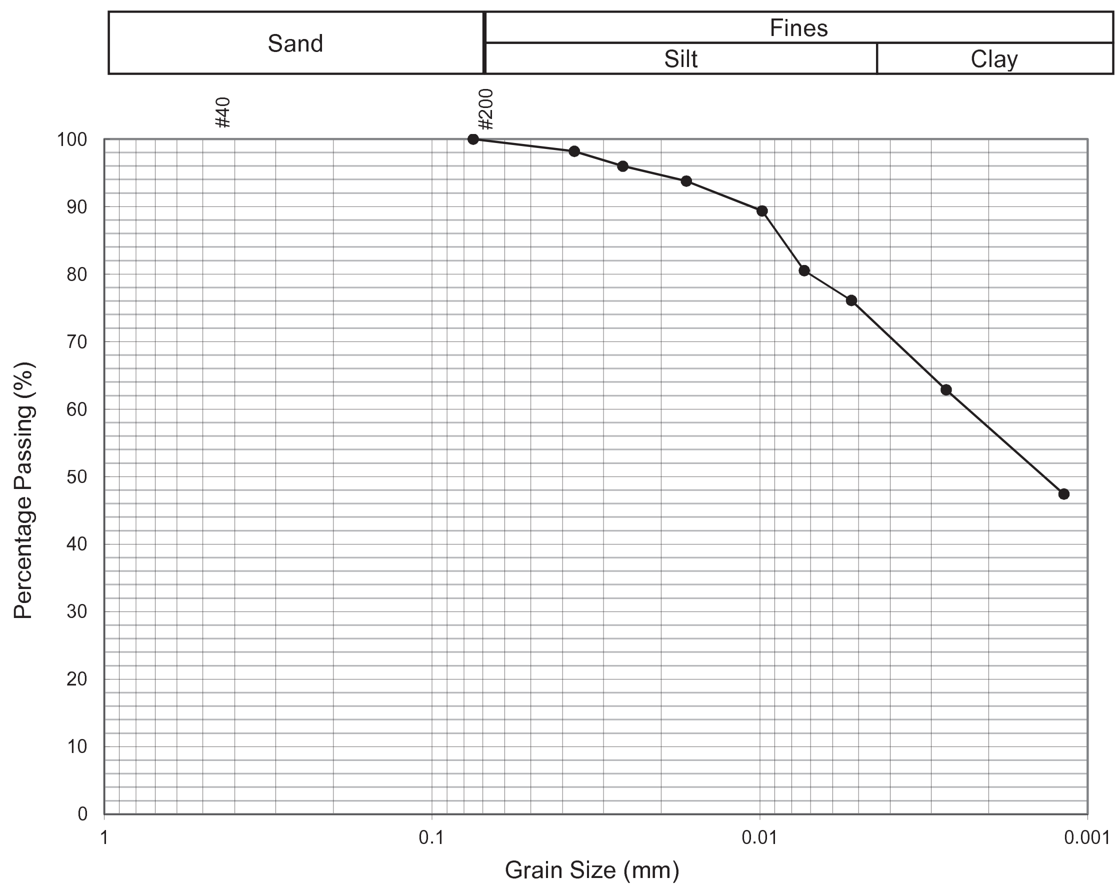
Form Nº MTS107

Project:	AMECFW - Myra Falls			Project No.:	15-MTS-029
Location:	BC			Date:	October 29, 2015
Borehole:	BH15-32A	Sample No.:	S7	Depth (m):	65.14

Hydrometer Test (ASTM D422)

Unified Soil Classification System (ASTM D 2487)

Description of Material: Pale green lean CLAY



Sample No.	Depth (m)	Percentage of Material by Weight (%)			
		Gravel	Sand	Fines	
				Silt	Clay
S7	65.14	-	0	28	72

Comments:

Prepared by:	PC	Checked by:	MF	Approved by:	PS
Date:	October 29, 2015	Date:	October 29, 2015	Date:	October 29, 2015

MEG Technical Services (MTS)

(A Division of MEG Consulting Limited)

Marine + Earth Geosciences

Form N° MTS104

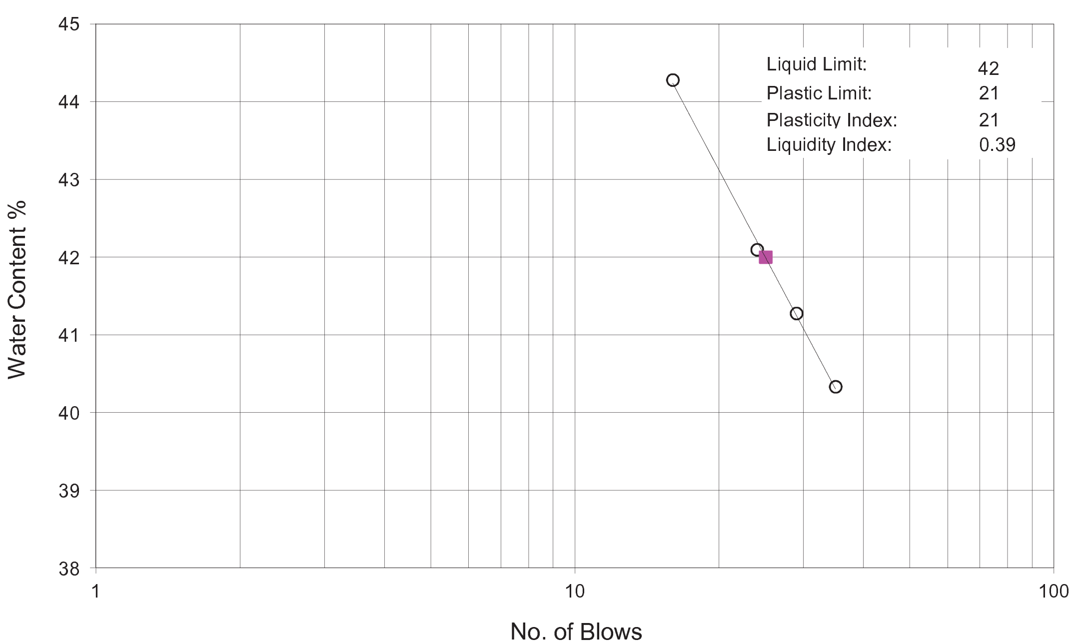
Project:	AMECFW - Myra Falls			Project No.:	15-MTS-029
Location:	BC			Date:	October 27, 2015
Borehole:	BH15-32A	Sample No.:	S7	Depth (m):	65.13

Liquid Limit, Plastic Limit and Plasticity Index of Soils (ASTM D4318)

LIQUID LIMIT								PLASTIC LIMIT						
TIN No.	Tare + Weight of Wet Soil (g)	Tare + Weight of Dry Soil (g)	Weight of Tin (g)	Weight of Water (g)	Weight of Dry Soil (g)	Water Content (%)	No. of Blows	TIN No.	Tare + Weight of Wet Soil (g)	Tare + Weight of Dry Soil (g)	Weight of Tin (g)	Weight of Water (g)	Weight of Dry Soil (g)	Water Content (%)
38	48.08	44.20	34.58	3.88	9.62	40.3	35	1E	36.45	35.28	29.73	1.17	5.55	21.1
15	43.58	40.41	32.73	3.17	7.68	41.3	29	30E	36.65	35.50	30.04	1.15	5.46	21.1
64	41.09	38.80	33.36	2.29	5.44	42.1	24							
61	33.78	30.49	23.06	3.29	7.43	44.3	16							

Classification of the material : CL

100 % with respect to the total of the material smaller than sieve No. 40



Observations:

Prepared by:	PC	Checked by:	MF	Approved by:	PS
Date:	October 27, 2015	Date:	October 27, 2015	Date:	October 28, 2015

OQM Organizational Quality Management Program

MEG Technical Services (MTS)

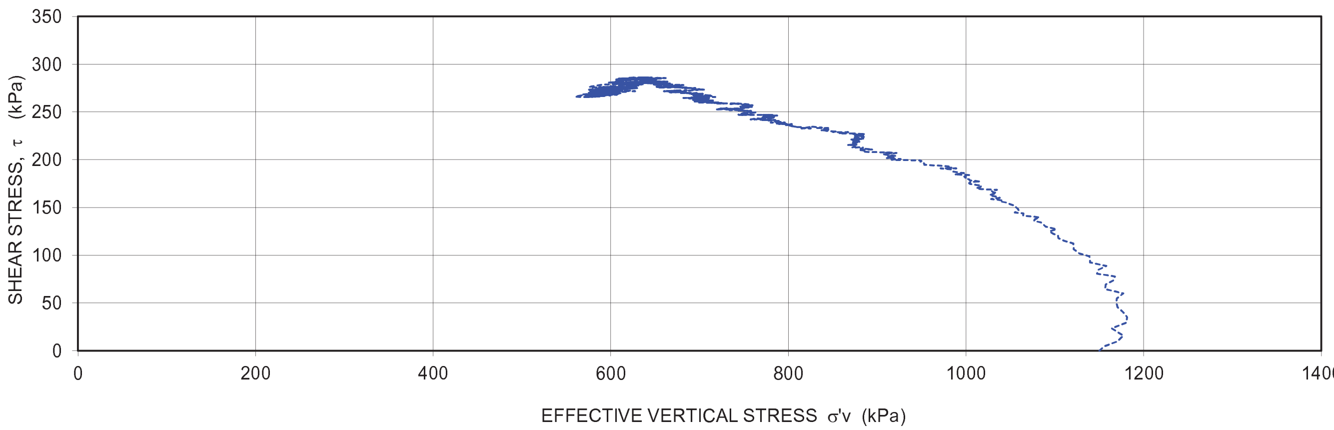
(A Division of MEG Consulting Limited)

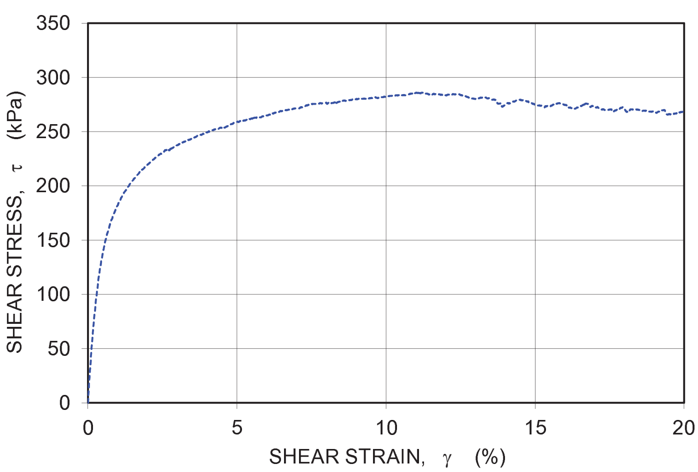
Marine + Earth MEG Geosciences

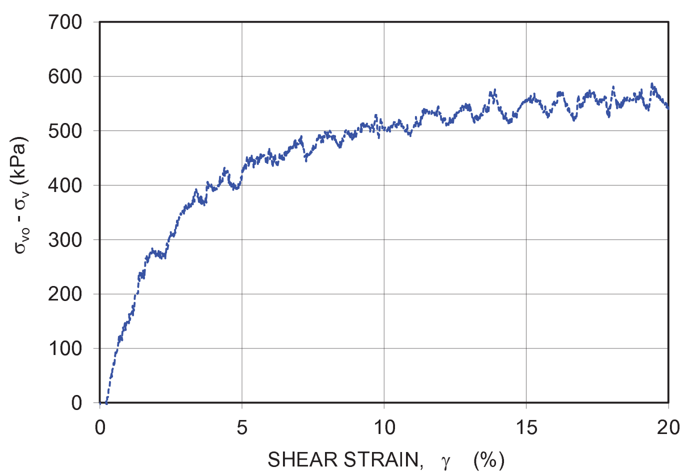
Project:	AMECFW - Myra Falls	Project No.:	15-MTS-029
Location:	BC	Date:	October 27, 2015
Borehole:	BH15-32A	Depth (m):	65.13
Sample No.:	S7		

Direct Simple Shear (ASTM D6528)

Initial Height (mm):	23.5	**Weight of Specimen (g):**	197.40	**Initial Void Ratio, e_o:**	0.83
Diameter of Ring (mm):	73.2	**Total Unit Weight (kN/m^3):**	19.60	**Final Void Ratio, e_f:**	0.61
Specific Gravity, Gs:	2.83	**Dry Unit Weight (kN/m^3):**	15.19	**Natural Water Content (%):**	29.1
Final Water Content (%):	23.6	**Initial Degree of Saturation, Sr (%):**	99.3	**Final Degree of Saturation, Sr (%):**	>100







Type of Test: Constant Volume					
Sample No.	Depth (m)	Total Unit Weight (kN/m^3)	Effective Vertical Stress, σ'v (kPa)	Strain Rate (%/hour)	Test OCR
S7	65.13	19.6	1500	5	-

Comments:

Prepared By:	MF	Checked By:	PS	Approved By:	JPS
Date:	October 27, 2015	Date:	October 27, 2015	Date:	October 29, 2015

OQM Organizational Quality Management Program

MEG Technical Services (MTS)

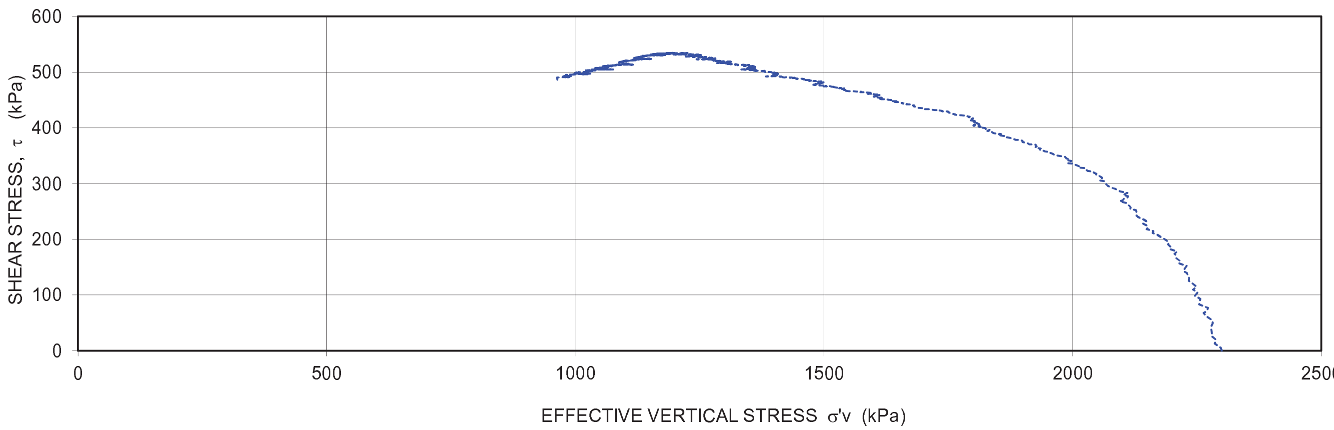
(A Division of MEG Consulting Limited)

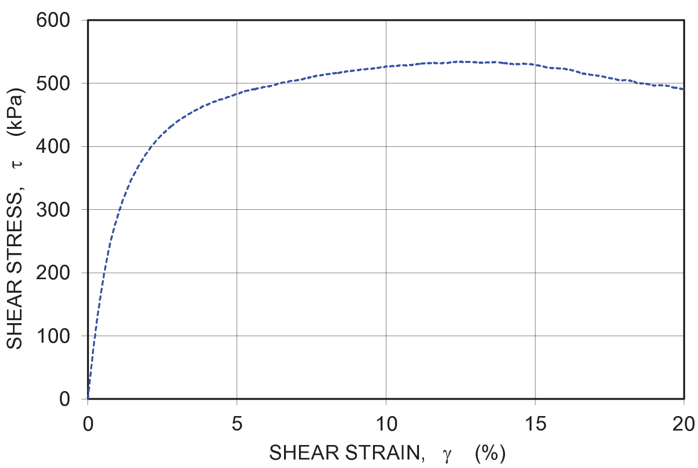
Marine + Earth MEG Geosciences

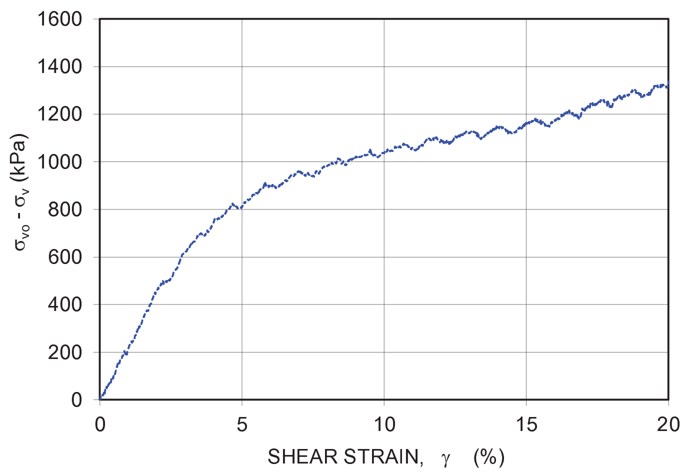
Project:	AMECFW - Myra Falls	Project No.:	15-MTS-029
Location:	BC	Date:	October 27, 2015
Borehole:	BH15-32A	Depth (m):	65.15
Sample No.:	S7		

Direct Simple Shear (ASTM D6528)

Initial Height (mm):	23.5	**Weight of Specimen (g):**	197.98	**Initial Void Ratio, e_o:**	0.83
Diameter of Ring (mm):	73.2	**Total Unit Weight (kN/m^3):**	19.65	**Final Void Ratio, e_f:**	0.50
Specific Gravity, Gs:	2.83	**Dry Unit Weight (kN/m^3):**	15.19	**Natural Water Content (%):**	29.3
Final Water Content (%):	22.5	**Initial Degree of Saturation, Sr (%):**	100.0	**Final Degree of Saturation, Sr (%):**	>100







Type of Test: Constant Volume					
Sample No.	Depth (m)	Total Unit Weight (kN/m^3)	Effective Vertical Stress, σ'v (kPa)	Strain Rate (%/hour)	Test OCR
S7	65.15	19.6	2300	5	-

Comments:

Prepared By:	MF	Checked By:	PS	Approved By:	JPS
Date:	October 27, 2015	Date:	October 27, 2015	Date:	October 29, 2015

APPENDIX D

Nyrstar Myra Falls Old TDF Tailings Properties[2]

[2] Amec Foster Wheeler, 2016. "Myra Falls Old Tailings Disposal Facility Glaciolacustrine Evaluation Report - Appendix B", GLU Evaluation Report, March 2016.

Memo

To:	File		
From:	Dixie Ann Simon (AmecFW)		
Tel:	1 (250) 847-8783	**File No:**	NX14001F.4
Date:	31 March 2016	**Cc:**	Ed McRoberts (AmecFW)

Subject: **Old TDF Tailings**
Geotechnical Material Properties

A. INTRODUCTION AND BACKGROUND

The purpose of this memorandum is to summarize the geotechnical material properties of the tailings that have been used in historical Old Tailings Disposal Facility (Old TDF) stability analyses and the data that are basis for the values used in the current Old TDF stability analyses as well as published technical papers. Not all historical test results were made available to Amec Foster Wheeler. Only the results contained in reports that were provided to Amec Foster Wheeler or publically available are discussed herein.

The Old TDF was designed by Knight Piesold Ltd. (KPL) in 1982. The original design report was not made available to Amec Foster Wheeler; however, the tailings properties used in the design of the Old TDF were summarized in a paper authored by the designers of the Old TDF[1]. The summary of the tailings properties, excerpted from the paper, is provided in Table A-1.

KPL conducted cyclic Triaxial tests on 12 reconstituted samples of the tailings in 1982 as part of their study of the seismic stability of the Old TDF. The purpose of the testing was to determine the residual shear strength ratio, s_u/σ'_{v0}, for use in post-seismic slope stability analyses. The samples were tested at the University of British Columbia's Soil Mechanics Library by Dr. Y.P. Vaid. Detailed descriptions of the sample preparation and testing procedures are provided in KPL's 1982 report[2]. The summary of the results, excerpted from the report, is provided in Table A-2.

---

[1] Design and Operation of the Myra Falls Tailings Disposal Facility, Haile, J.P. and Kerr, T.F., 1989
[2] Myra Falls Tailings Disposal Facility, Report on Seismic Stability Considerations, Knight and Piesold Ltd., 07 July 1982.

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Table A-1: Tailings Properties Used In Design of Old TDF

Tailings Type	G_s	LL %	PI %	Dry Density t/m³	Coefficient of Vertical Permeability cm/s		Coefficient of Consolidation m²/year	Shear Strength			
					After Full Drainage	After Consolidation to Average In-situ stress		Friction Angle (remolded) degrees	Friction Angle (undisturbed) degrees	Effective Cohesion kPa	Shear Strength Ratio at 2% Strain
Myra-Lynx	2.95	45	8	1.2	$2.5x10^{-5}$	$1x10^{-6}$	14 to 153	27.2	32.8	0	0.35
HW	3.95	23	NP	2.0	$3.5x10^{-5}$	$3x10^{-6}$	78 to 147	30.0	34.5	0	0.32

Table A-2: Summary of KPL Cyclic Triaxial Testing on Reconstituted Tailings Samples

Sample No.	Initial Void Ratio %	Initial Saturation Ratio	Effective Confining Stress kPa	Over Consolidation Ratio	Void Ratio after Consolidation %	Saturation Ratio after Consolidation	Cyclic Stress Ratio	Number of Cycles for 2.5% Axial Strain	Post Cyclic Deviator Stress at 8% Axial Strain kPa	Comments
1	2.01	0.89	50	1	1.86	0.88	0.13	19	19	
2	2.05	0.89	50	1	1.80	0.89	0.13	>100	26	Did not liquefy in 100 cycles; load attenuated
3	1.84	0.94	100	1	1.44	0.92	0.12	50	61	
4	1.80	0.92	100	1	1.43	0.92	0.12	48	70	Cyclic load was non symmetric
5	1.56	0.87	40	4.5	1.45	0.87	0.37	11	80	
6	1.49	0.90	40	4.5	1.32	0.89	0.35	18	100	Did not develop strain in 76 cycles at stress ratio = 0.16 and additional 76 cycles of stress ratio 0.24
7	1.56	1,00	40	4.5	1.16	1.00	0.40	14	65	
8	1,54	1.00	40	4.5	1.20	1.00	0.32	30	60	
9	1.06	0.95	40	2	0.95	0.96	0.27	14	50	Did not liquefy under 76 cycles of stress ration=0.20
10	1.08	0.87	40	2	1.02	0.92	0.29	6	35	
11	0.91	1.00	320	1	1,000.74	1.00	0.14	30	76	
12	0.90	0.99	320	1	0.76	0.98	0.18	7	42	

KPL analyzed the results of the testing and selected a post-seismic residual shear strength ratio, S_{ur}/σ'_{v0}, of 0.2 for use in the design of the Old TDF. KPL selected the shear strength at an arbitrary strain level of 8% to calculate the residual strength ratio. The arbitrary strain level was selected because KPL observed 'strain hardening' in all of the post-cyclic, monotonic undrained shear tests. They concluded the strain hardening was the result of 'consistent dilative response' during undrained shear.

Knight Piesold conducted cyclic direct simple shear tests on thin-walled tube samples obtained during three investigations in 1995. The samples were obtained from an area of sandy tailings located approximately in the vicinity of the existing pipe bridge. The sandy tailings reportedly were the result of deposition of 'raw' tailings. KPL suspected that the sandier tailings might 'exhibit much poorer post-liquefaction behaviour than the silt tailings'. Samples were obtained prior and after completion of a trial densification program. Samples numbers 5 through 9 were obtained after the completion of the trial densification program.

The samples were also tested at UBC by Dr. Vaid. Detailed descriptions of the field program and testing procedures are provided in KPL's 1996 report[3]. A summary of the results, excerpted from the appended report prepared by Dr. Vaid, is provided in Table A-3. A summary of the index property and static direct simple shear testing of the tailings samples conducted by KPL is provided in Table A-4. The order that the samples are listed in Table A-3 and Table A-4 is not the same. The order for both tables is as provided in the referenced reports.

[3] Myra Falls Operations, Tailings Storage Facility, Report on 1995 Site Investigations and Trial Drainage/Densification Program (REF. NO. 1288D/2A), Knight Piesold Ltd., 13 March 1996

Table A-3: Summary of KPL Cyclic and Post Cyclic Direct Simple Shear Tests

Test No.	Bore Hole ID	Depth (ft)	Initial Water Content	Over Consolidation Ratio	Vertical Effective Consolidation Stress kPa	Cyclic Tests			Post Cyclic Tests	
						Cyclic Stress Ratio	Number of Cycles	Maximum Shear Strain during Cyclic Loading %	Peak Shear Strength kPa	Peak Strain %
1	SPT95-1	25	0.162	1	159	0.105	10	0.2	159	19
						0.116	10	0.3		
						0.134	15	3.8		
2	SPT95-2	53	0.361	1	357	0.130	18	3.8	156	15
3	SPT95-2	51	0.189	1	357	0.100	10	0.5	134	19
						0.111	20	4.6		
1	SPT95-3	25	0.189	1	157	0.115	14	6.3	97	25
2	SPT95-3	31	0.352	1	198	0.117	15	0.9	35	20
						0.128	10	4.2		
3	SPT95-4	56	0.163	1	366	0.130	6	6.5	70	20
4	CPT95-23	55.5	0.183	1	396	0.121	17	4.6	170	20
5[1]	SPT95-5	19.5	0.192	1	115	0.125	13	4.4	123	18
6[1]	SPT95-5	29	0.378	1	183	0.128	9	5.4	62	22
7[1]	SPT95-5	47.5	0.162	1	314	0.129	17	3.8	165	22
8[1]	SPT95-6	45	0.323	1	295	0.137	9	3.9	60	20
9[1]	SPT95-5	38	0.368	1	236	0.137	11	5.0	40	18

Notes: 1. Samples obtained after densification

Table A-4: Summary of KPL Index Properties and Static Direct Simple Shear Testing

Penetration No.	Soil Type	Phase	Depth of Sample m	Vertical Effective Consolidation Stress kPa	Shear Strength Ratio	G_s	Initial Water Content	Unit Weight g/cm^2	Initial Void Ratio	Void Ratio after Consolidation	% Sand	% Silt
SPT95-2	silt	I	15.5	357	0.17	3.56	36.1	1.51	1.361	1.045	0.6	99.4
SPT95-3	silt	II	9.5	198	0.18	3.65	35.2	1.42	1.570	1.209	0.0	100
SPT95-5[1]	silt	III	8.8	183	0.34	4.17	37.8	2.11	0.976	0.890	0.0	100
SPT95-5[1]	silt	III	11.6	236	0.17	3.78	36.8	1.39	1.719	1.432	0.0	100
SPT95-6[1]	silt	III	13.7	295	0.21	3.57	32.3	1.50	1.380	1.107	3.3	96.7
SPT95-1	sand	I	7.6	159	1.00	3.62	16.2	2.05	0.765	0.672	39.9	60.1
SPT95-2	sand	I	15.5	357	0.39	3.85	18.9	2.32	0.751	0.662	40.3	59.7
SPT95-3	sand	II	7.6	157	0.62	3.72	18.9	2.12	0.869	0.757	62.0	38.0
SPT95-4	sand	II	17.1	366	0.19	4.01	16.3	2.29	0.919	0.752	62.6	37.4
CPT95-23	sand	II	16.9	396	0.44	3.76	18.3	2.33	0.733	0.614	51.4	48.6
SPT95-5[1]	sand	III	5.9	115	1.07	3.75	19.2	2.12	0.904	0.770	9.3	90.7
SPT95-5[1]	sand	III	14.5	314	0.52	4.00	16.2	2.41	0.732	0.658	5.3	94.7

Notes: 1. Samples obtained after densification

The seismic upgrade berm was designed in 1998 by Klohn Crippen Ltd (KCL). KCL did not conduct any independent testing of the tailings materials, relying on their interpretation of post-cyclic shear strength testing conducted at the University of British Columbia, case histories and their experience. KCL selected a drained effective friction angle of 28° and a post-seismic residual shear strength ratio, S_{ur}/σ'_{v0}, of 0.14.

The cyclic shear response of fine-grained mine tailings was studied by Wijewickreme et al[4]. The database used in the study consisted of the results of constant volume cyclic direct simple shear tests conducted at geotechnical research laboratory at the University of British Columbia. Six out of the 20 cyclic DSS tests in the data base of tests conducted on copper-gold-zinc (CGZ) tailings are samples that were tested by KPL during their 1995 investigations. Two of the six tests were on samples obtained after densification. Nonetheless, the database provides a range of values for residual shear strength ratios for comparison with historical and current values used in analyses. Excluding four values for CGZ tailings that seem to be unrealistically high, the average post-cyclic residual shear, S_{ur}/σ'_{v0}, strength ratio is 0.21. The minimum is 0.13.

B. 2015 AMALGAMATED PASTE AREA BERM INVESTIGATION

Amec Foster Wheeler completed the field program for the required APA berm stabilization in 2015. The investigation consisted of borehole drilling using the sonic method of drilling, SPT and thin-walled tube sampling, in-situ vane shear testing and cone penetration testing with pore pressure measurements (CPTu). Seismic CPTu and pore pressure dissipation tests were also completed. The mobilized or peak shear strength, s_u, and the shear strength ratio, s_u/σ'_{v0}, as interpreted from CPT data are shown in Figure B-1 and Figure B-2. The vane shear data also is shown in the figures.

Figure B-1: Undrained Shear Strength from CPT and Vane Shear Tests

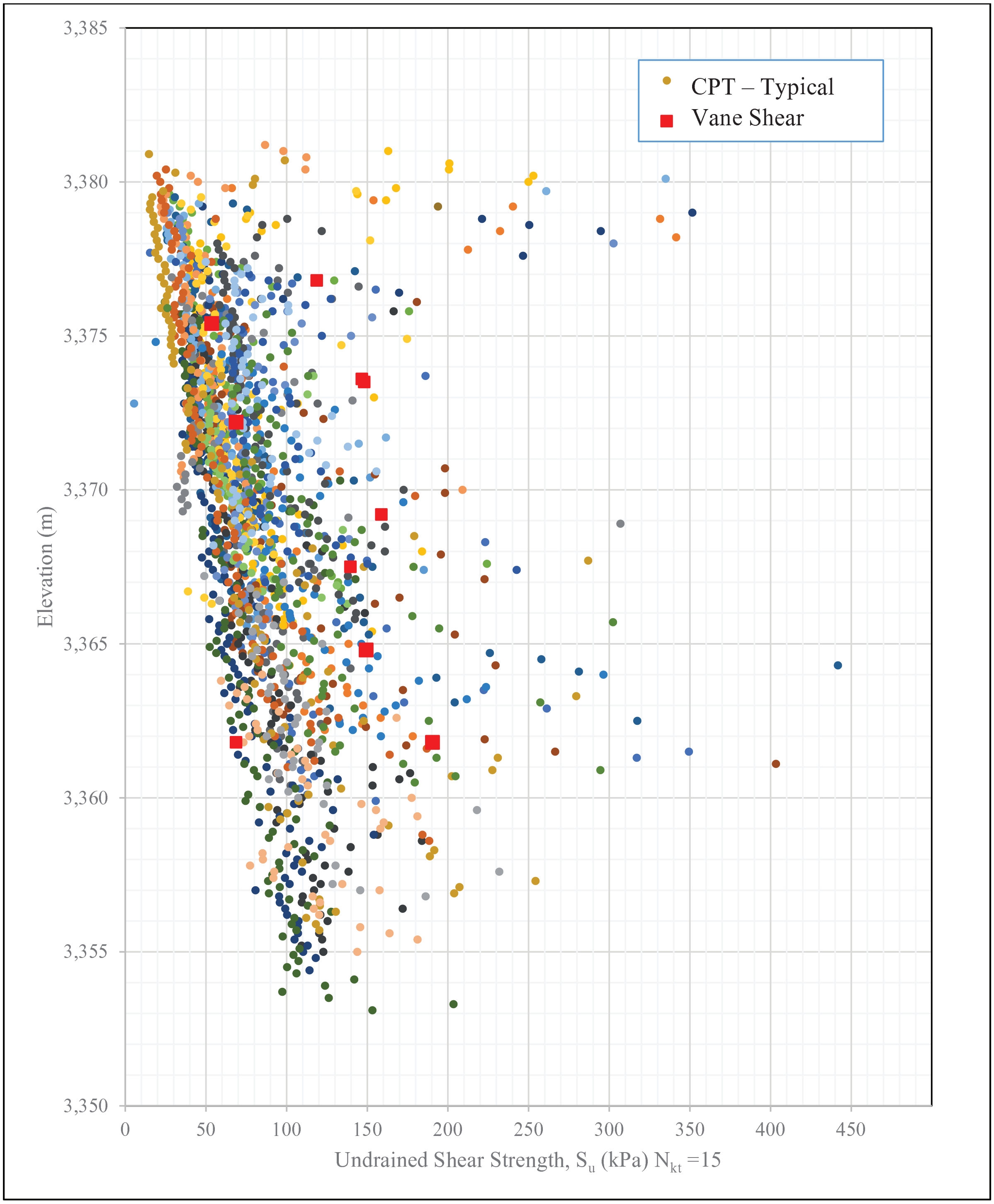
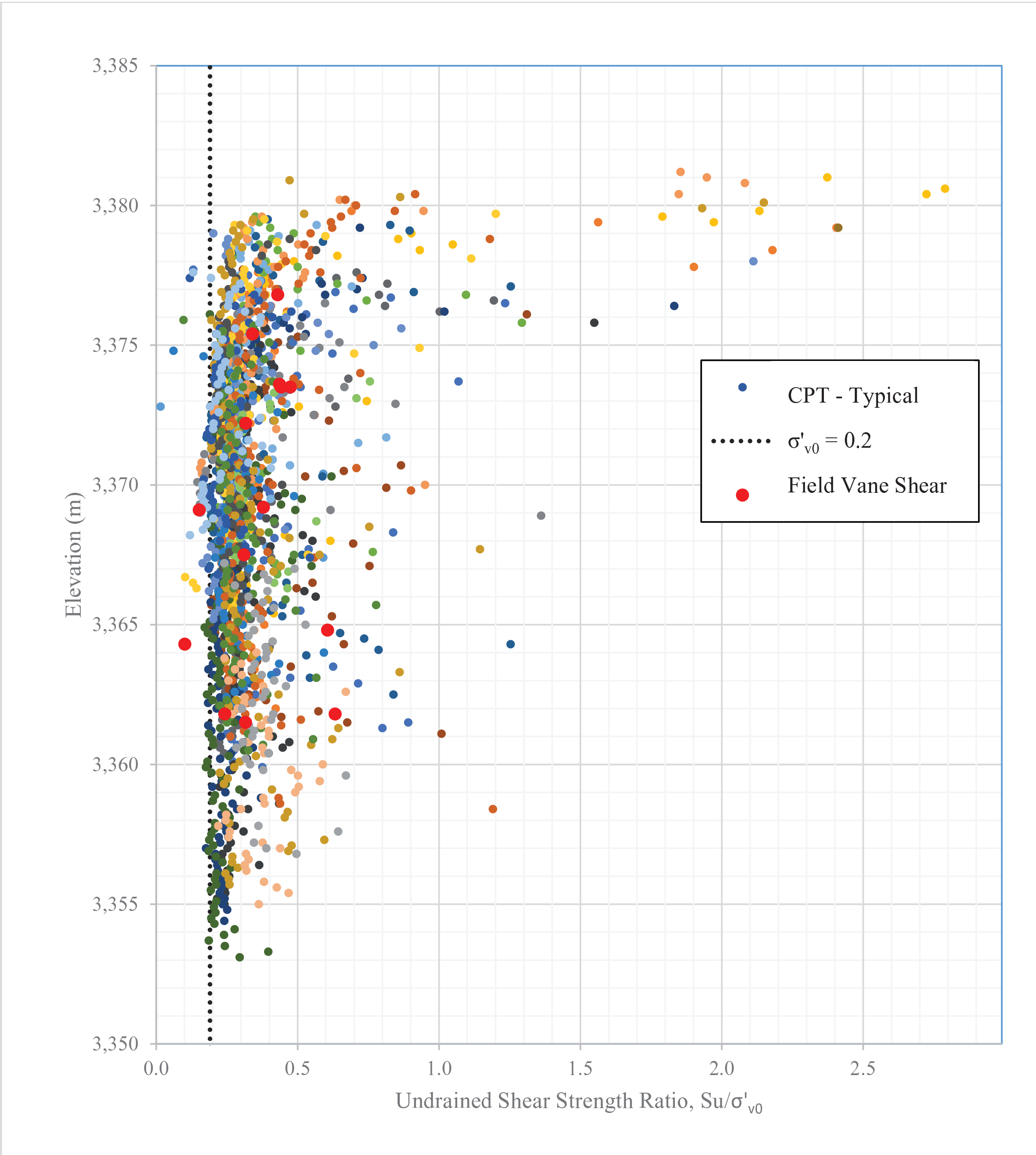
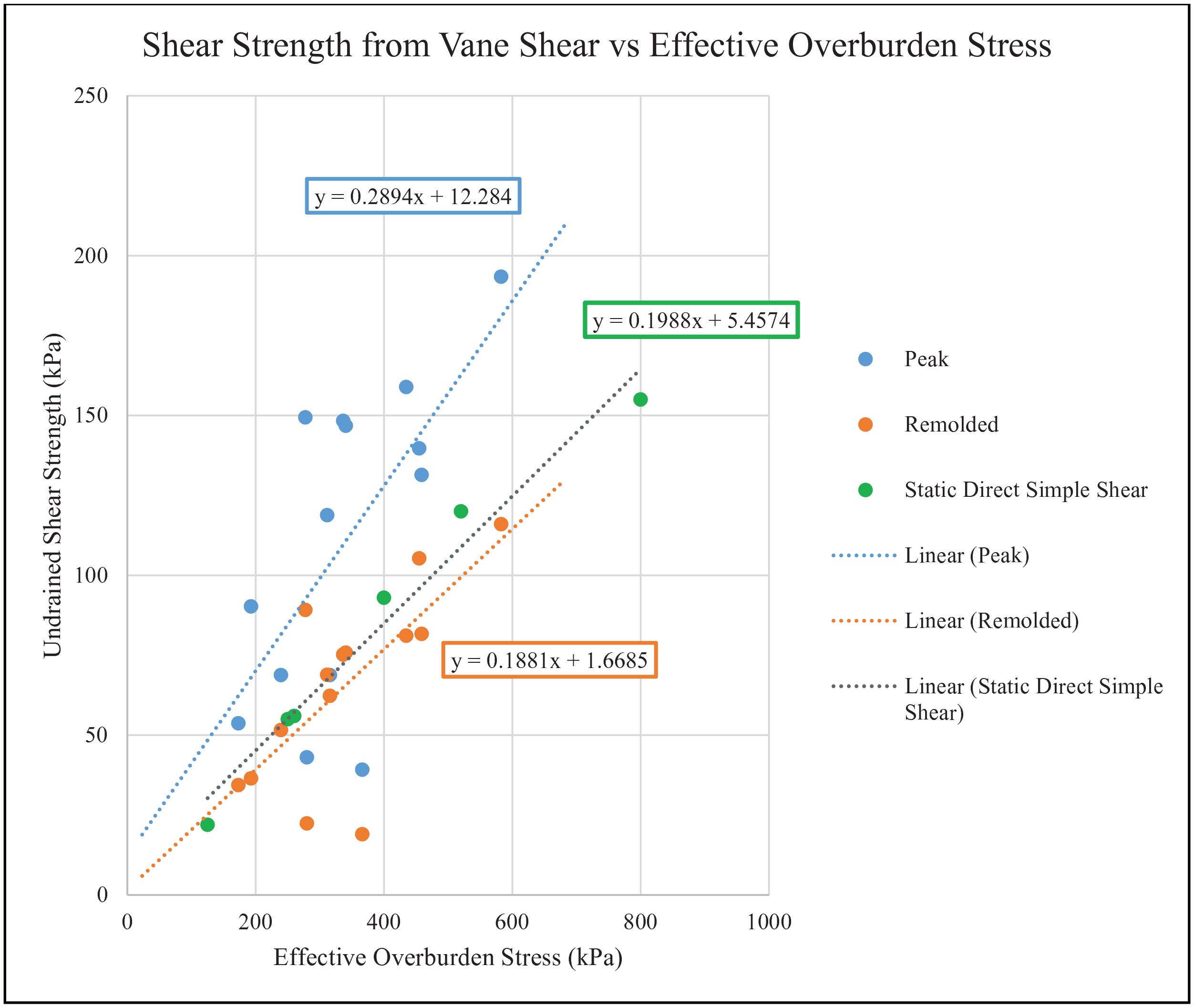


Figure B-2: Undrained Shear Strength Ratio



The vane shear data is shown graphically in Figure B-3. The results of the static direct simple shear tests, discussed in Section C, are shown for comparison. The detailed descriptions and all of the results of the investigation are provided in the 2015 APA berm data report[5].

Figure B-3: Summary of Vane Shear Testing



Selected grab samples were tested for index properties. In total, 57 samples were tested to determine water content and 52 samples were tested to determine the percentage of clay and silt size particles. Water content ranged between 11% and 42% with the majority between 24% and 37%. The percentage of clay and silt sized particles in samples of the tailings was generally greater than 90%.

Three selected thin walled tube samples were selected for index testing. A summary of the index property testing of thin-walled tube samples is provided in Table B-1. Analysis of the CPTu data is in process.

Table B-1: Summary of Tailings Index Properties

Borehole Number	Sample ID	Depth m	Atterberg Limits			USC	Specific Gravity	Grain Size Distribution			Total Unit Weight kN/m^3	Water Content %
			LL %	PL %	PI %			% Sand	% Silt	% Clay		
BH15-A05	SH02	17.9	25	19	6	CL-ML	3.54	1	74	25	20.5	26.4
BH15-A17	SH02	24.1	21	17	4	CL	3.52		97.2[1]		21.3	31.2
BH15-A25	SH02	17.9	28	18	10	CL	3.53		93.8[1]		21.3	28.1

Notes: 1. Result from grab sample.

Additional thin walled tube samples were obtained of the tailings encountered in the mud-rotary bore holes that were advanced during the 2015 glaciolacustrine field program. The samples were obtained specifically for static and cyclic direct simple shear testing as discussed in Section C.

C. STATIC AND CYCLIC DIRECT SIMPLE SHEAR TESTING

All thin-walled tube samples obtained of the tailings encountered in the mud rotary boreholes were submitted to MEG Technical Services (MTS) laboratory in Richmond, BC for gamma ray scanning. Three tubes were selected for subsequent testing. The tubes were selected based on depth within the tailings deposit, recovery, and visual evidence of uniformity based on the gamma ray scans.

The selected samples were tested for index properties also by MTS including Atterberg Limits, grain size distribution and specific gravity. Sample disturbance was assessed by making shear wave velocity measurements. Field shear wave velocity measurements were not available at the location of the samples because of the borehole from which these thin-walled tube samples were obtained was abandoned prior to installing the required casing in the borehole as discussed in the data report. The results are provided in Table C-1.

Laboratory shear wave velocity measurements provide an indication of sample disturbance when compared with field shear wave velocity measurements; however, shear wave velocity measurements were not available for the borehole from which the tube samples were obtained. The borehole from which these samples were obtained was abandoned prior to installation of the casing required for downhole shear wave velocity measurements. The borehole was abandoned because a thin-walled tube sampler was lost during a sampling attempt in the foundation soils. The range of field shear wave velocities (downhole and SCPTu) measured in the tailings deposit is provided in the table.

Table C-1: Summary of Index Properties

Borehole Number	Sample ID	Depth m	Atterberg Limits			USCS	Specific Gravity	Grain Size Distribution			Laboratory Shear Wave Velocity m/s	Range of Downhole Shear Wave Velocity m/s
			LL %	PL %	PI %			% Sand	% Silt	% Clay		
BH15-3	SH1	6.35	28	15	13	CL	3.34	3	66	31	196	112 to 459
	SH2	12.52	32	15	17	CL	3.44	1	69	30	282	
	SH3	19.33	25	17	8	CL	3.62	1	71	28	325	

The selected tube samples were also tested for static and post-cyclic residual shear strength in direct simple shear under constant volume conditions. The testing was completed by MTS. Static direct simple shear tests were conducted with consolidation pressures equal to the estimated effective overburden stress and again at twice the estimated overburden stress. The purpose of the second test at twice the estimated effected over burden stress was to assess whether or not the tailings were under (unlikely), normally or over (unlikely) consolidated. The results generally indicate that the tailings are normally consolidated.

Cyclic direct simple shear (DSS) tests were conducted on samples consolidated to the estimated effective overburden stress. Cyclic stress ratios, number of cycles and frequency were specified by Amec Foster Wheeler. Post-cyclic, monotonic DSS were completed on samples that developed transient liquefaction (cyclic mobility) or experience more than 3.5% strain during cyclic testing. The results of the static, cyclic and post-cyclic testing are summarized in Table C-2. The laboratory test results are provided in Attachment B.

4 Cyclic Shear Response of Fine-grained Mine Tailings, Wijewickreme, D, Sanin, M.V., Greenaway, G.R., NRC Research Press Website at HTTP://CGI.nrc.ca. October 2005

5 2015 Geotechnical Site Investigation Data Report, Amalgamated Paste Area (APA) Berm, Amec Foster Wheeler, Project NO. NX1400D.6.200, June 2015

Table C-2: Summary of Static, Cyclic and Post-Cyclic Constant Volume Direct Simple Shear Testing

Borehole Number	Sample ID	Depth m	Total Unit Weight γ kN/m^3	Effective Overburden Stress σ'_{v0} kPa	Static Shear Strength S_u kPa	Static Shear Strength Ratio S_u/σ'_{v0}	Cyclic Stress Ratio τ_{cyc}/σ'_{v0}	Number of Cycles to Transient Liquefaction at 0.1 Hz	Peak Cyclic Shear Strength S_{ur} kPa	Estimated Excess Pore Pressure %	Post Cyclic Shear Strength Ratio S_{ur}/σ'_{v0}
BH15-30	S3	6.32	20.2 to 22.3	125 (250)	22 (53)	0.18 (0.21)	0.095	>30	--	--	--
							0.13	>30	--	--	--
							0.17	6	29	90	0.23
	S5	12.52	20.2 to 22.1	260 (520)	56 (113)	0.22 (0.22)	0.11	>30	--	--	--
							0.15	15	45	87	0.18
							0.19	3	53	77	0.20
	S7	19.33	21 to 23.3	400 (800)	82 (150)	0.21 (0.19)	0.095	>30	--	--	--
							0.13	>30	55	88	0.14
							0.17	4	75	83	0.19

Note: '—' Post-cyclic testing was not conducted because the sample did not exhibit transient liquefaction nor more than 3.5% strain. Post-cyclic strength can be assumed to be at least equal to the static shear strength.

D. TAILINGS MATERIAL PROPERTIES USED IN HISTORICAL SLOPE STABILITY ANALYSES

A summary of the tailings strength properties used in historical slope stability analyses are provided in Table D-1.

Table D-1: Summary of Historical and Tailings Strength Properties Used in Design of the Old TDF

Consultant	Total Unit Weight kN/m³	Static Strength Parameters		Post-seismic Residual Shear Strength Ratio S_{ur}/σ'_{v0}
		Angle of Internal Friction Degrees	Shear Strength Ratio S_{ur}/σ'_{v0}	
Knight Piesold	Not provided	33[1]	0.35	0.20
Klohn Crippen	22.5	28	--	0.14
AMEC (2010)[6]	22	28	--	0.14

Notes: 1. Undisturbed angle of internal friction from Table A-1.

[6] Myra Falls Mine Tailings Disposal Facility, Slope Stability Results for TDF/APA Closure, AMEC Earth and Environmental, Project No. NX10011.2010.2, 07 September 2010.

Attachment A

Laboratory Test Reports

MEG Technical Services (MTS)

(A Division of MEG Consulting Limited)

Form Nº MTS109

Project:	AMECFW - Myra Falls	Project No.:	15-MTS-029
Location:	BC	Date:	January 8, 2016
Borehole:	BH15-39		

Specific Gravity (ASTM D854)

Sample Number	Depth (m)	Volumetric flask No.	Weight of flask and soil (g)	Weight of flask, water and soil (g)	Weight of dry solid (g)	Temperature (°C)	Specific Gravity Gs
SH-01	6.32	7	241.49	722.92	54.89	25	3.34
SH-02	15.48	1	214.20	698.92	48.31	25	3.44
SH-03	19.38	7	237.52	721.31	50.92	25	3.62

Comments :

Prepared by:	PC	Checked by:	MF	Approved by:	PS
Date:	January 8, 2016	Date:	January 8, 2016	Date:	January 11, 2016

MEG Technical Services (MTS)

(A Division of MEG Consulting Limited)

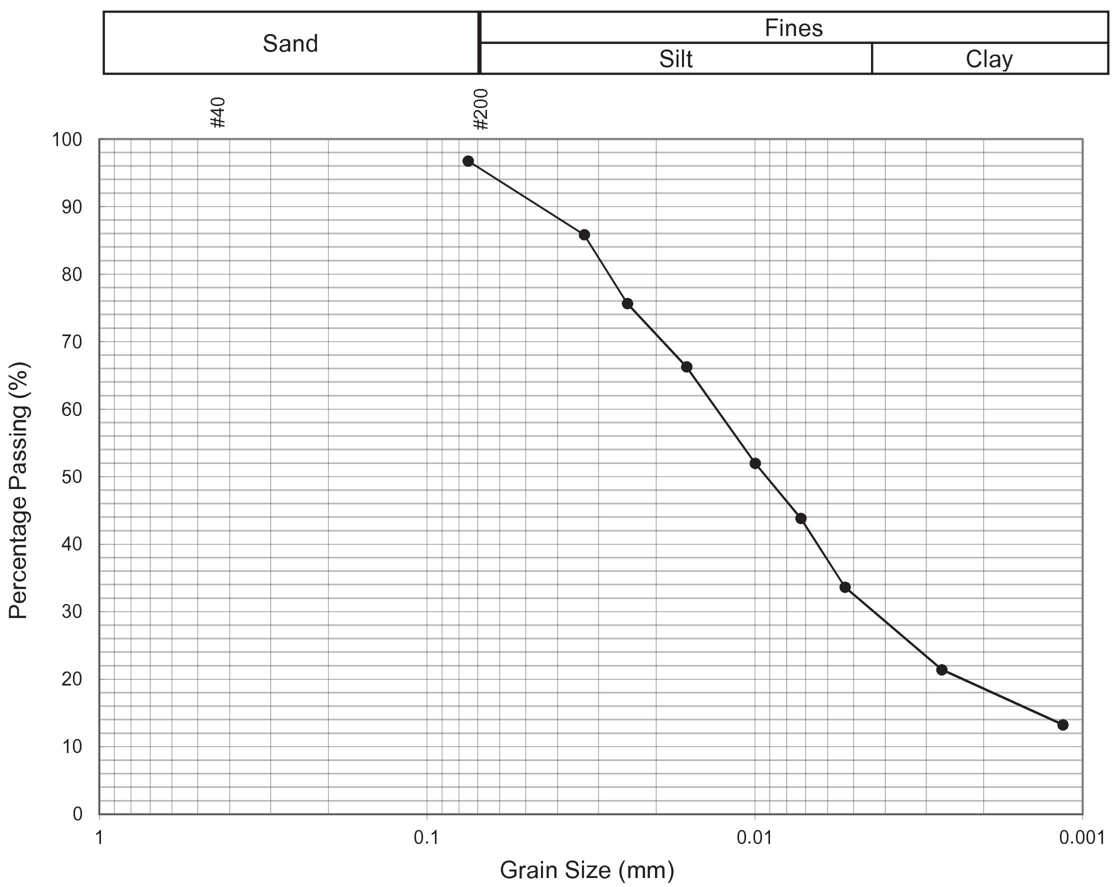
Form Nº MTS107

Project: AMECFW - Myra Falls			Project No.: 15-MTS-029
Location: BC			Date: January 14, 2016
Borehole: BH15-39	Sample No.: SH-01		Depth (m): 6.35

Hydrometer Test (ASTM D422)

Unified Soil Classification System (ASTM D 2487)

Description of Material: Very dark gray lean CLAY



Sample No.	Depth (m)	Percentage of Material by Weight (%)			
		Gravel	Sand	Fines	
				Silt	Clay
SH-01	6.35	-	3	66	31

Comments: Clay description based on Atterberg limits result and ASTM flow chart

Prepared by: PC	Checked by: MF	Approved by: PS
Date: January 14, 2016	Date: January 14, 2016	Date: January 15, 2016

MEG Technical Services (MTS)

(A Division of MEG Consulting Limited)

Marine + Earth MEG Geosciences

Form N° MTS104

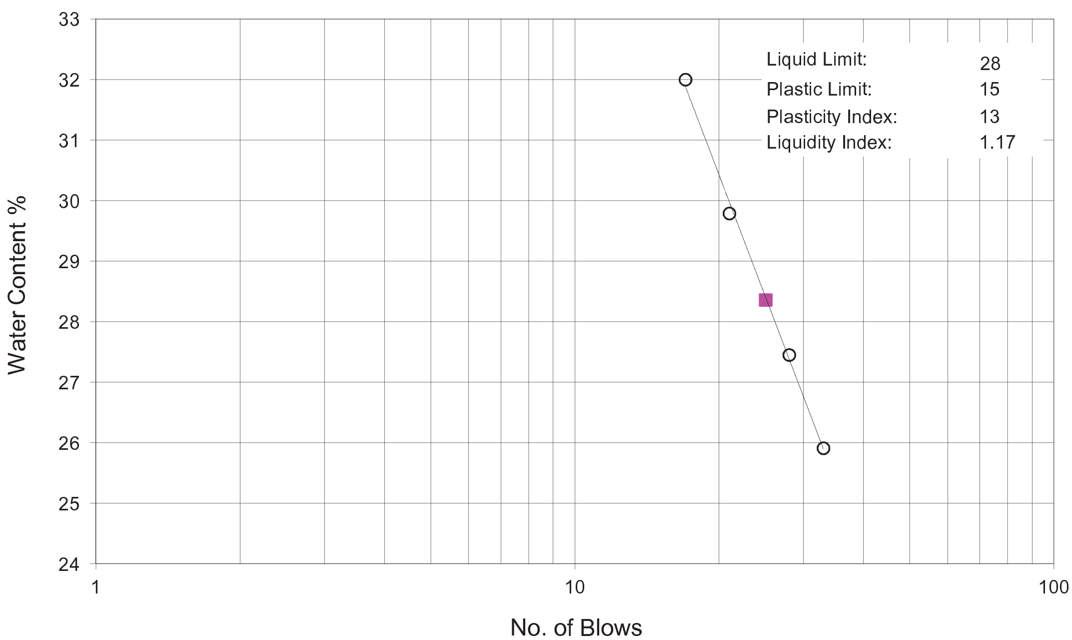
Project:	AMECFW - Myra Falls		Project No.:	15-MTS-029
Location:	BC		Date:	January 13, 2016
Borehole:	BH15-39	Sample No.: SH-01	Depth (m):	6.35

Liquid Limit, Plastic Limit and Plasticity Index of Soils (ASTM D4318)

LIQUID LIMIT								PLASTIC LIMIT						
TIN No.	Tare + Weight of Wet Soil (g)	Tare + Weight of Dry Soil (g)	Weight of Tin (g)	Weight of Water (g)	Weight of Dry Soil (g)	Water Content (%)	No. of Blows	TIN No.	Tare + Weight of Wet Soil (g)	Tare + Weight of Dry Soil (g)	Weight of Tin (g)	Weight of Water (g)	Weight of Dry Soil (g)	Water Content (%)
2A	44.25	41.53	33.03	2.72	8.50	32.0	17	64C	38.47	37.41	30.51	1.06	6.90	15.4
93	37.21	34.10	23.66	3.11	10.44	29.8	21	12E	37.85	36.81	30.06	1.04	6.75	15.4
18	45.87	43.35	34.17	2.52	9.18	27.5	28							
17	40.79	37.52	24.90	3.27	12.62	25.9	33							

Classification of the material : CL

100 % with respect to the total of the material smaller than sieve No. 40



Observations:

Prepared by:	PC	Checked by:	MF	Approved by:	PS
Date:	January 13, 2016	Date:	January 13, 2016	Date:	January 15, 2016

MEG Technical Services (MTS)

(A Division of MEG Consulting Limited)

Form Nº MTS217a

Marine + Earth Geosciences

Project:	AMECFW - Myra Falls	**Borehole:**	BH15-39	**Project No.:**	15-MTS-029
Client:	AMEC	**Sample No.:**	SH01	**Date:**	March 10, 2016
Location:	BC	**Depth (m):**	6.47	**Station:**	DSS2

Bender Element Velocity Measuring

WATER CONTENT & UNIT WEIGHT

	Initial	Final
Tin No.:	93	C34
Wt. of Tin (g):	23.67	195.00
Wet Weight (g):	129.23	389.47
Dry Weight (g):	103.43	342.65
Water Content (%):	32.3	31.7
Total Unit Weight (kN/m^3):	19.60	20.62
Dry Unit Weight (kN/m^3):	14.81	15.64

CONSOLIDATION

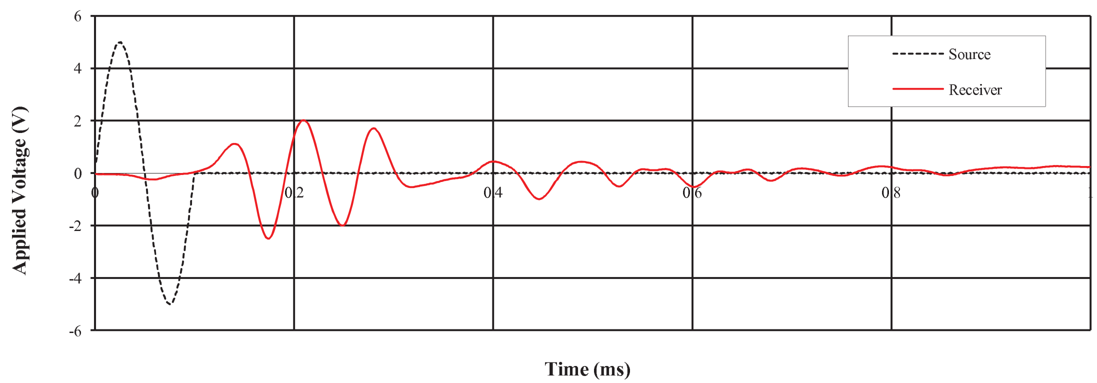
Specific Gravity, Gs:	3.34
Initial DSS Sample Height (mm):	23.5
Height after Consolidation (mm):	22.3
Initial Void Ratio, e_o	1.21
Initial Degree of Saturation (%)	89.1
Final Void Ratio, e_f	1.10
Final Degree of Saturation (%)	97.2

BENDER ELEMENTS

	S-Wave
Initial Time, T_o (ms):	0.026
Final Time, T_f (ms):	0.140
Travel Time (ms):	0.114
Wave Velocity (m/s):	196
Shear Modulus, G (MPa)	76.4
Vertical Effective Stress, s'$_v$ (kPa)	125
Maximum to Present Stress Ratio	-

WAVE TRAVEL CURVES

S-WAVE



Comments: Test performed on sample consolidated to 125kPa and tested at a CRS of 0.13

*Vs is based on assessment of first shear wave arrival

Prepared By:	MF	Checked By:	PS	Approved By:	JPS
Date:	March 10, 2016	Date:	March 10, 2016	Date:	March 11, 2016

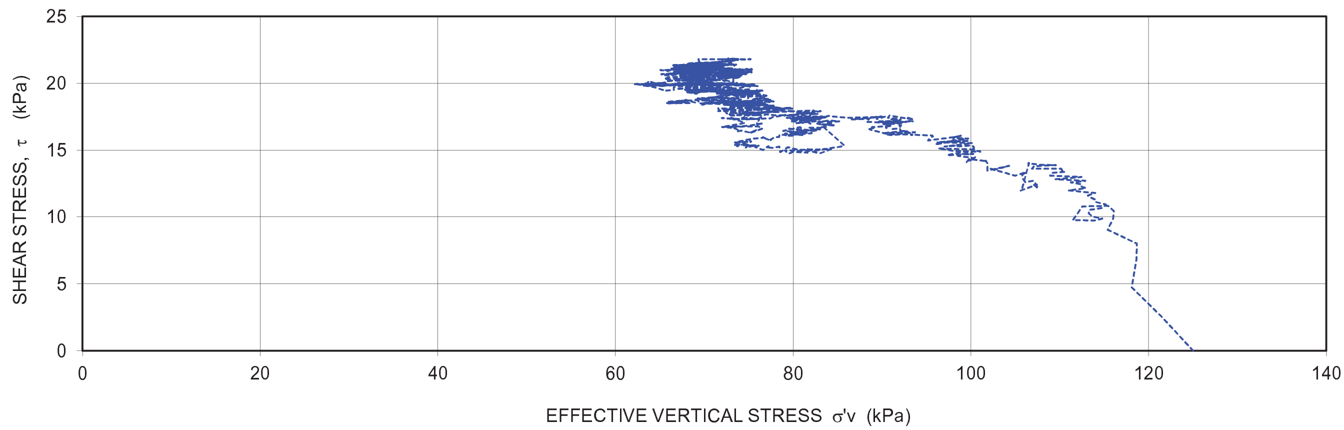
MEG Technical Services (MTS)
(A Division of MEG Consulting Limited)

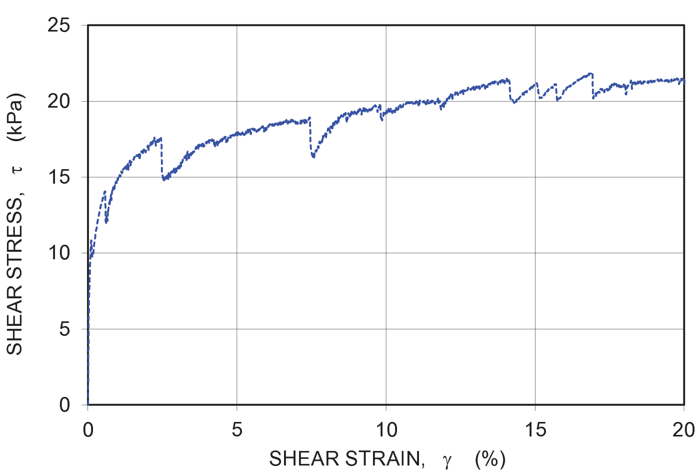
Marine + Earth MEG Geosciences

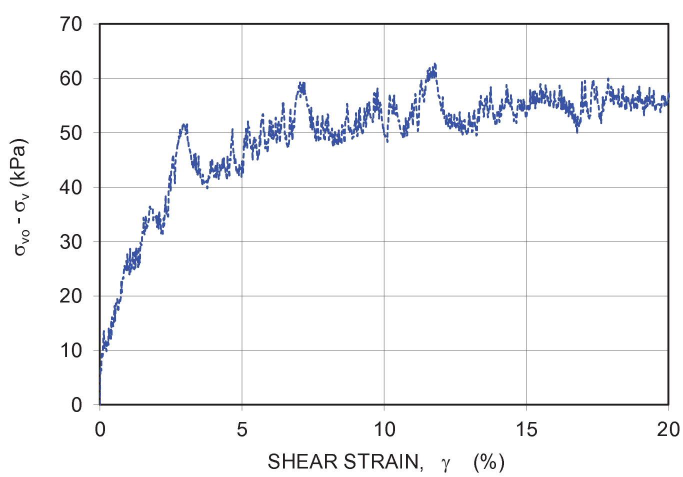
Project:	AMECFW - Myra Falls	Project No.:	15-MTS-029
Location:	BC	Date:	January 13, 2016
Borehole:	BH15-39	Depth (m):	6.32
Sample No.:	SH01		

Direct Simple Shear (ASTM D6528)

Initial Height (mm):	23.5	**Weight of Specimen (g):**	202.83	**Initial Void Ratio, e_o:**	1.21
Diameter of Ring (mm):	73.1	**Total Unit Weight (kN/m^3):**	20.20	**Final Void Ratio, e_f:**	1.03
Specific Gravity, Gs:	3.34	**Dry Unit Weight (kN/m^3):**	14.81	**Natural Water Content (%):**	36.4
Final Water Content (%):	35.4	**Initial Degree of Saturation, Sr (%):**	100.1	**Final Degree of Saturation, Sr (%):**	>100







Type of Test: Constant Volume					
Sample No.	Depth (m)	Total Unit Weight (kN/m^3)	Effective Vertical Stress, σ'v (kPa)	Strain Rate (%/hour)	Test OCR
SH01	6.32	20.2	125	5	-

Comments:

Prepared By:	MF	Checked By:	PS	Approved By:	JPS
Date:	January 13, 2016	Date:	January 13, 2016	Date:	January 15, 2016

MEG Technical Services (MTS)

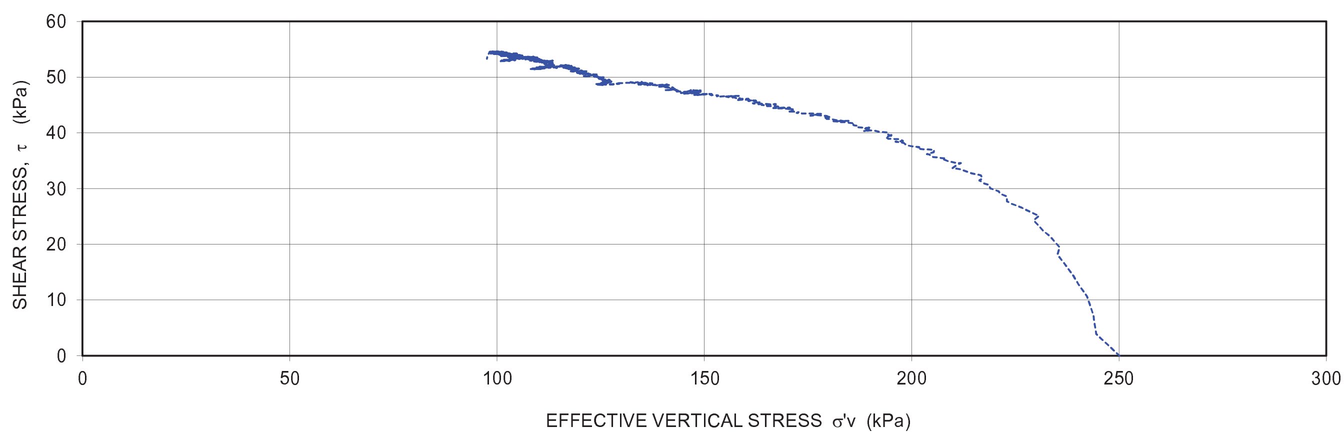
(A Division of MEG Consulting Limited)

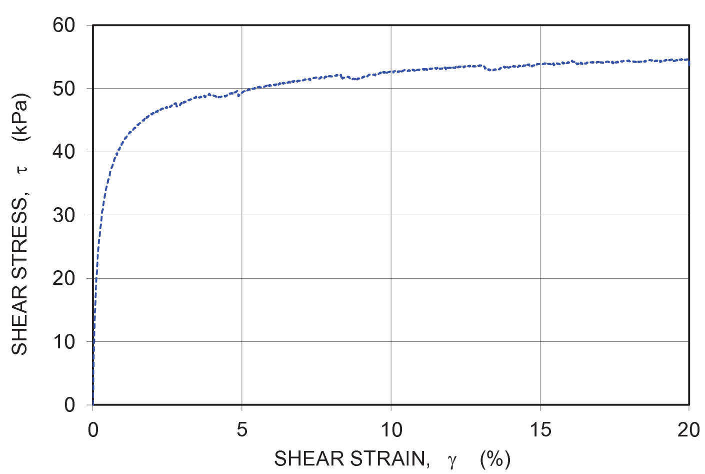
Marine + Earth MEG Geosciences

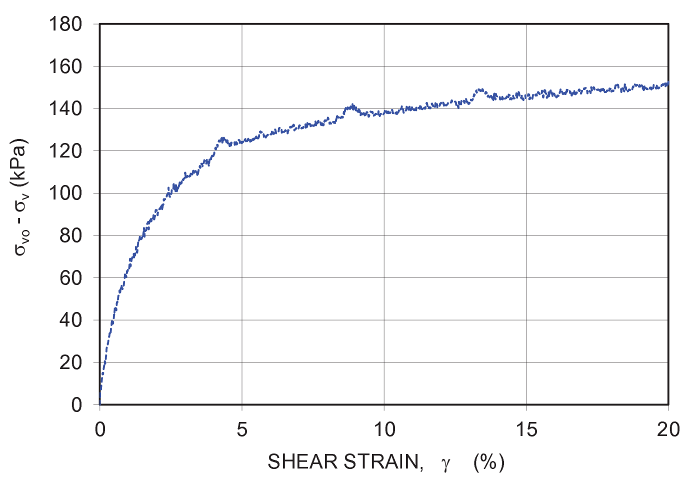
Project:	AMECFW - Myra Falls	Project No.:	15-MTS-029
Location:	BC	Date:	January 13, 2016
Borehole:	BH15-39	Depth (m):	6.35
Sample No.:	SH01		

Direct Simple Shear (ASTM D6528)

Initial Height (mm):	23.4	**Weight of Specimen (g):**	204.41	**Initial Void Ratio, e_o:**	1.20
Diameter of Ring (mm):	73.2	**Total Unit Weight (kN/m^3):**	20.40	**Final Void Ratio, e_f:**	0.96
Specific Gravity, Gs:	3.34	**Dry Unit Weight (kN/m^3):**	14.93	**Natural Water Content (%):**	36.6
Final Water Content (%):	35.2	**Initial Degree of Saturation, Sr (%):**	100.0	**Final Degree of Saturation, Sr (%):**	>100







Type of Test: Constant Volume					
Sample No.	Depth (m)	Total Unit Weight (kN/m^3)	Effective Vertical Stress, σ'v (kPa)	Strain Rate (%/hour)	Test OCR
SH01	6.35	20.4	250	5	-

Comments:

Prepared By:	MF	Checked By:	PS	Approved By:	JPS
Date:	January 13, 2016	Date:	January 13, 2016	Date:	January 15, 2016

OQM Organizational Quality Management Program

MEG Technical Services (MTS)

(A Division of MEG Consulting Limited)

Form Nº MTS210

Marine + Earth MEG Geosciences

Project:	AMECFW - Myra Falls			Project No.:	15-MTS-029
Location:	BC	Borehole:	BH15-39	Depth:	6.47 m
Sample:	SH-01	Station:	DSS2	Date:	March 8, 2016

Stress Controlled Cyclic Direct Simple Shear Test

0.095 stress ratio (τ_{cyc}/ σ'_{vc}) @ 0.1 Hz for 30 cycles, σ'_{vc}=125kPa

Initial sample Details		Final Sample Details	
Water Content (%):	29.7	Water Content (%):	23.4
Diameter (mm):	73.23	Diameter (mm):	73.23
Height (mm):	23.57	Change in Height, ΔH (mm):	0.89
Specific Gravity, Gs:	3.34	Final Height (mm):	22.68
Weight of Soil (g):	225.79	Weight of Soil (g):	214.92
Total Unit Weight (kN/m^3)	22.32	Total Unit Weight (kN/m^3)	22.08
Dry Unit Weight (kN/m^3)	17.21	Dry Unit Weight (kN/m^3)	17.88
Initial Void Ratio	0.91	Final Void Ratio	0.83

0.17 stress ratio (τ_{cyc}/ σ'_{vc}) @ 0.1 Hz for 6 cycles, σ'_{vc}=125kPa

Initial sample Details		Final Sample Details	
Water Content (%):	33.3	Water Content (%):	29.2
Diameter (mm):	73.23	Diameter (mm):	73.23
Height (mm):	23.48	Change in Height, ΔH (mm):	0.96
Specific Gravity, Gs:	3.34	Final Height (mm):	22.52
Weight of Soil (g):	214.54	Weight of Soil (g):	208.02
Total Unit Weight (kN/m^3)	21.28	Total Unit Weight (kN/m^3)	21.51
Dry Unit Weight (kN/m^3)	15.97	Dry Unit Weight (kN/m^3)	16.65
Initial Void Ratio	1.05	Final Void Ratio	0.97

0.13 stress ratio (τ_{cyc}/ σ'_{vc}) @ 0.1 Hz for 30 cycles, σ'_{vc}=125kPa

Water Content (%):	35.8	Water Content (%):	37.5
Diameter (mm):	73.18	Diameter (mm):	73.18
Height (mm):	23.54	Change in Height, ΔH (mm):	1.24
Specific Gravity, Gs:	3.34	Final Height (mm):	22.30
Weight of Soil (g):	212.82	Weight of Soil (g):	215.60
Total Unit Weight (kN/m^3)	21.09	Total Unit Weight (kN/m^3)	22.55
Dry Unit Weight (kN/m^3)	15.53	Dry Unit Weight (kN/m^3)	16.40
Initial Void Ratio	1.11	Final Void Ratio	1.00

Sample Description:

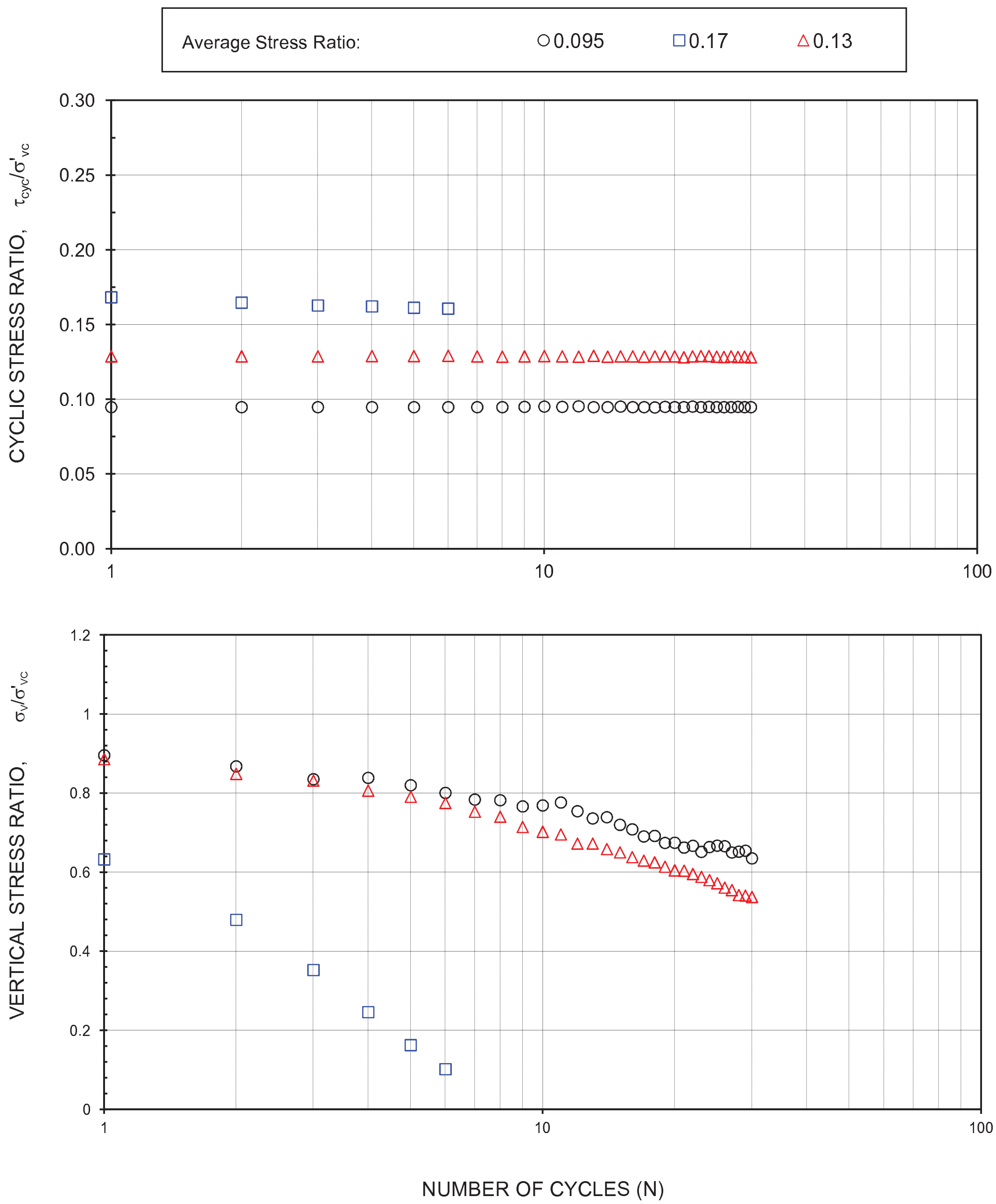
Prepared By:	MF	Checked By:	PS	Approved By:	JPS
Date:	March 8, 2016	Date:	March 8, 2016	Date:	March 11, 2016

MEG TECHNICAL SERVICES
(A Division of MEG Consulting Limited)

Marine + Earth
MEG
Geosciences

Project:	AMECFW - Myra Falls			Project No.:	15-MTS-029
Location:	BC	Borehole:	BH15-39	Depth:	6.47 m
Sample:	SH-01	Station:	DSS2	Date:	March 8, 2016

STRESS CONTROLLED CYCLIC DIRECT SIMPLE SHEAR TEST



Prepared By:	MF	Checked By:	PS	Approved By:	JPS
Date:	March 8, 2016	Date:	March 8, 2016	Date:	March 11, 2016

MEG TECHNICAL SERVICES

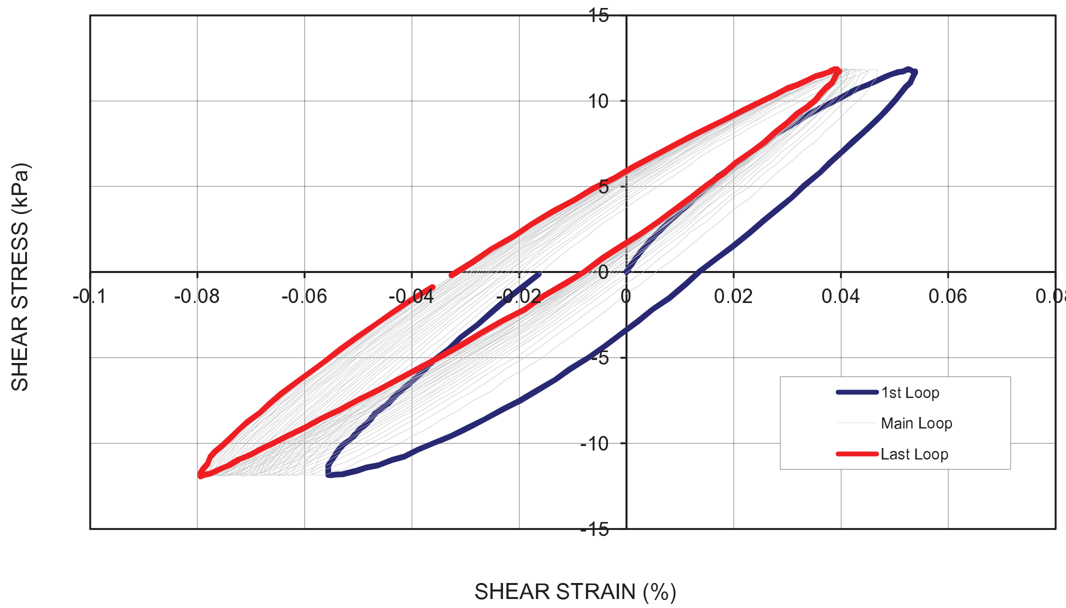
(A Division of MEG Consulting Limited)

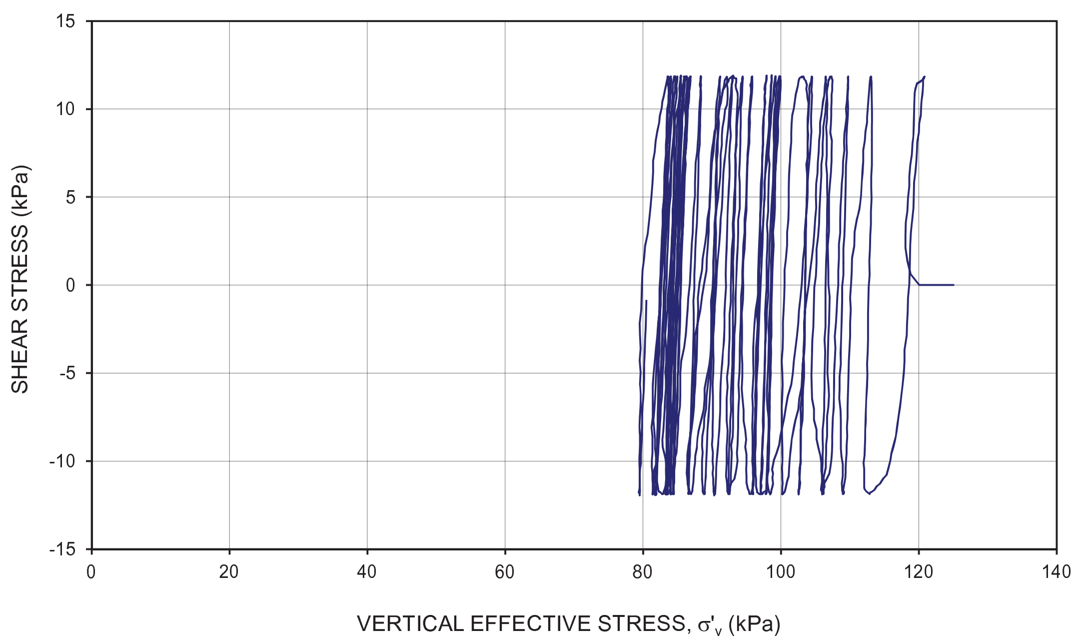
Marine + Earth Geosciences

Project:	AMECFW - Myra Falls			Project No.:	15-MTS-029
Location:	BC	Borehole:	BH15-39	Depth:	6.47 m
Sample:	SH-01	Station:	DSS2	Date:	March 8, 2016

STRESS CONTROLLED CYCLIC DIRECT SIMPLE SHEAR TEST

0.095 stress ratio (τ_{cyc}/ σ'_{vc}) @ 0.1 Hz for 30 cycles, σ'_{vc}=125kPa





Prepared By:	MF	Checked By:	PS	Approved By:	JPS
Date:	March 8, 2016	Date:	March 8, 2016	Date:	March 11, 2016

MEG TECHNICAL SERVICES

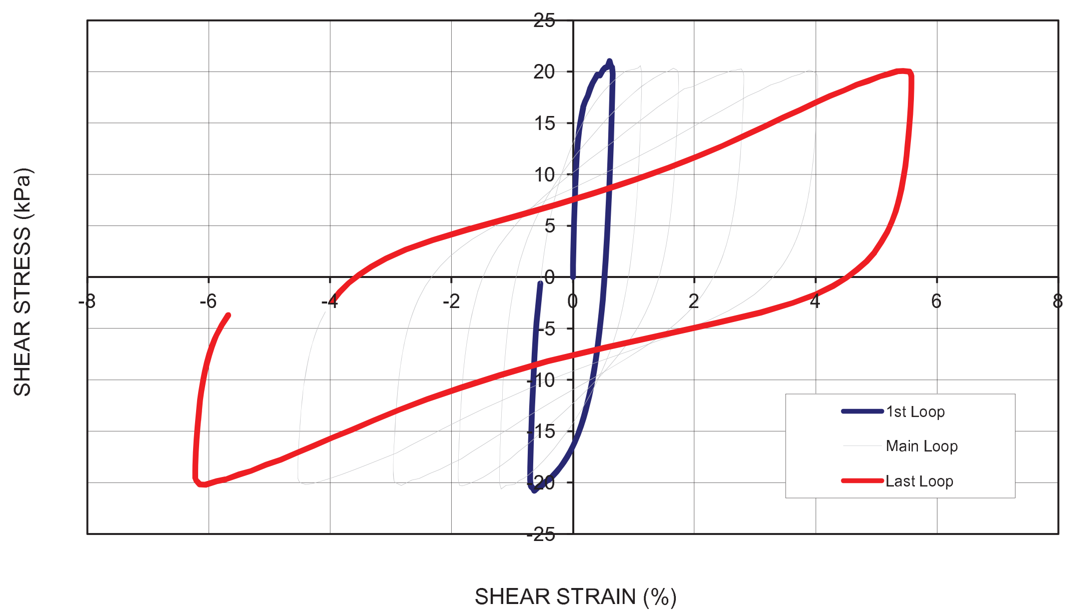
(A Division of MEG Consulting Limited)

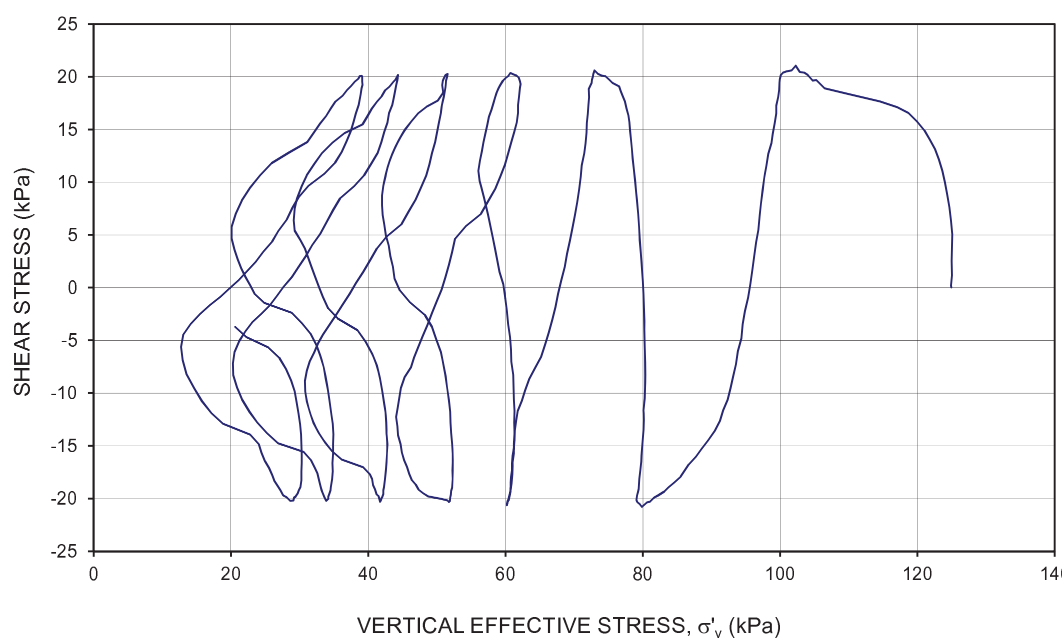
Marine + Earth MEG Geosciences

Project:	AMECFW - Myra Falls			Project No.:	15-MTS-029
Location:	BC	Borehole:	BH15-39	Depth:	6.47 m
Sample:	SH-01	Station:	DSS2	Date:	March 8, 2016

STRESS CONTROLLED CYCLIC DIRECT SIMPLE SHEAR TEST

0.17 stress ratio (τ_{cyc}/ σ'_{vc}) @ 0.1 Hz for 6 cycles, σ'_{vc}=125kPa





Prepared By:	MF	Checked By:	PS	Approved By:	JPS
Date:	March 8, 2016	Date:	March 8, 2016	Date:	March 11, 2016

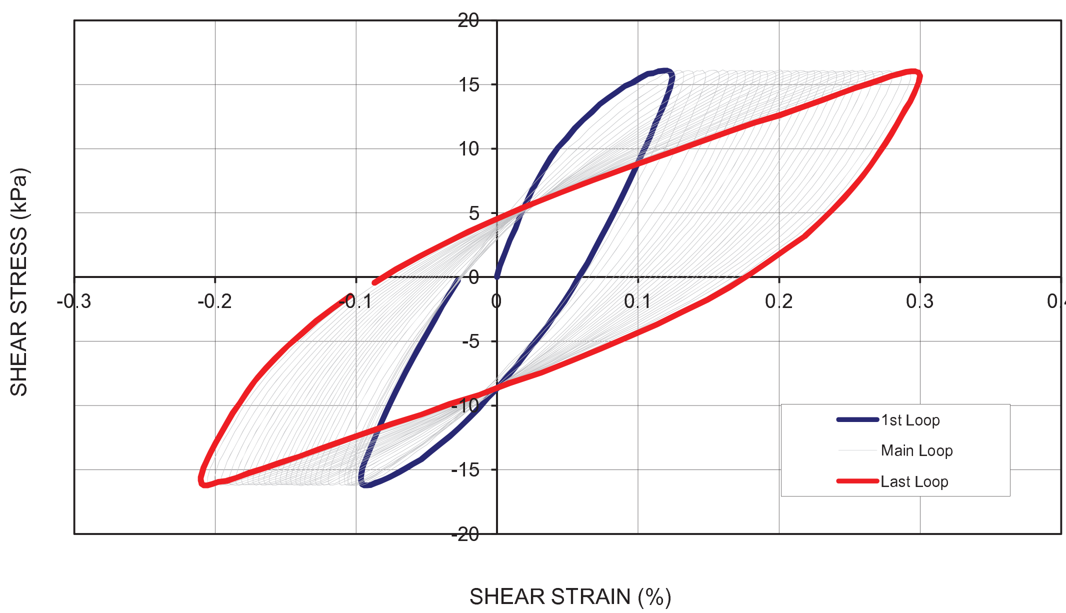
MEG TECHNICAL SERVICES
(A Division of MEG Consulting Limited)

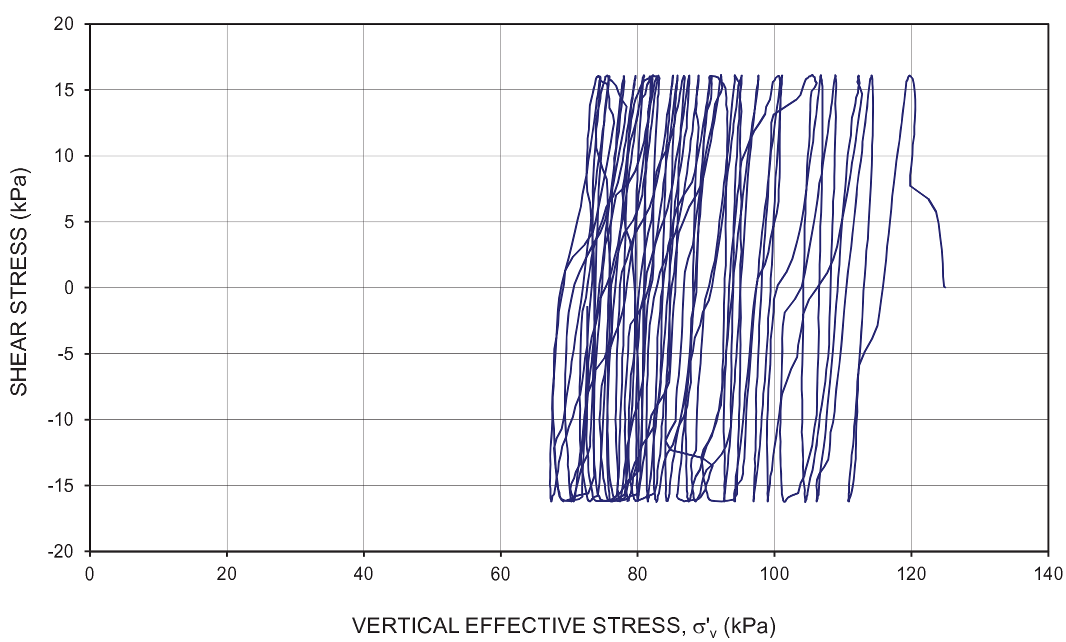
Marine + Earth MEG Geosciences

Project:	AMECFW - Myra Falls			Project No.:	15-MTS-029
Location:	BC	Borehole:	BH15-39	Depth:	6.47 m
Sample:	SH-01	Station:	DSS2	Date:	March 8, 2016

STRESS CONTROLLED CYCLIC DIRECT SIMPLE SHEAR TEST

0.13 stress ratio (τ_{cyc}/ σ'_{vc}) @ 0.1 Hz for 30 cycles, σ'_{vc}=125kPa





Prepared By:	MF	Checked By:	PS	Approved By:	JPS
Date:	March 8, 2016	Date:	March 8, 2016	Date:	March 11, 2016

OQM | Organizational Quality Management Program

MEG Technical Services (MTS)

(A Division of MEG Consulting Limited)

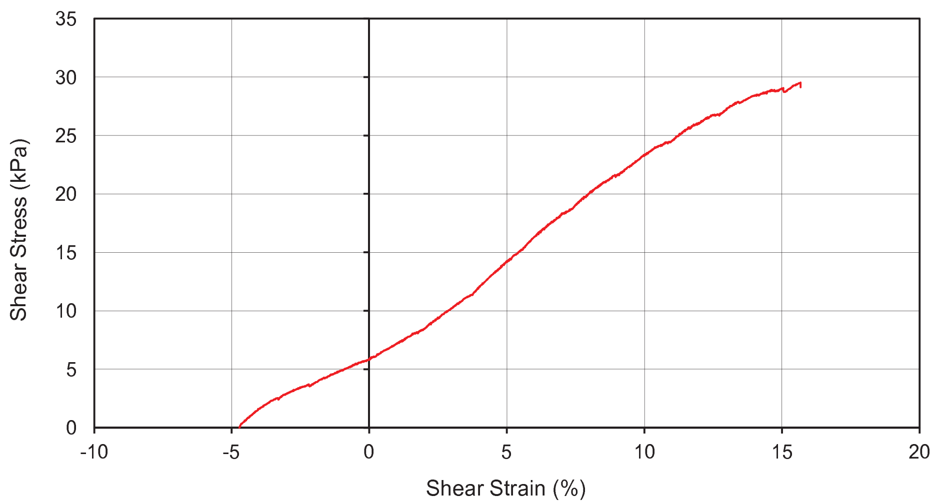
Form Nº MTS214

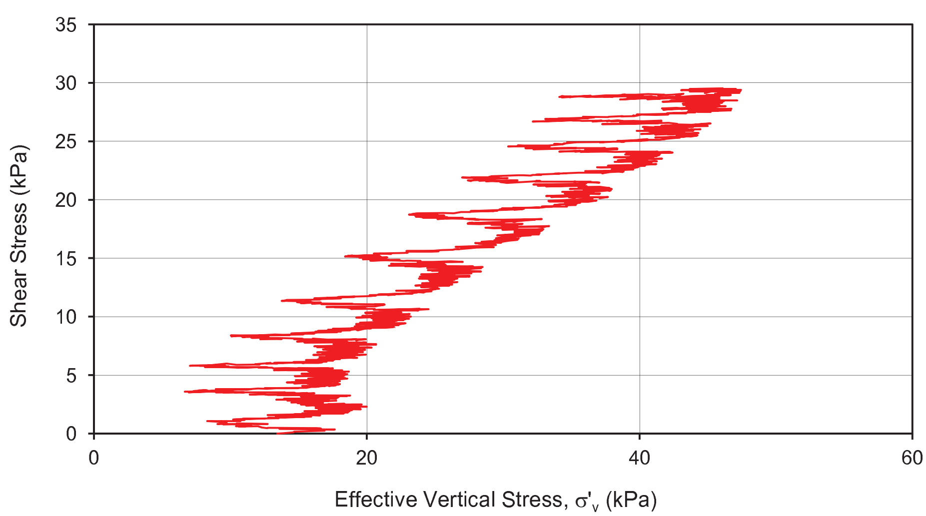
Marine + Earth MEG Geosciences

Project:	AMECFW - Myra Falls			Project No.:	15-MTS-029
Location:	BC	Borehole:	BH15-39	Depth:	6.47 m
Sample:	SH-01	Station:	DSS2	Date:	March 8, 2016

Post-Cyclic Static Direct Simple Shear Test

POST-CYCLIC STATIC SHEAR TEST





Note: Test performed after stress-controlled DSS test at average cyclic stress ratio, CSR = 0.17 with 90% excess pore pressure

Prepared By:	MF	Checked By:	PS	Approved By:	JPS
Date:	March 10, 2016	Date:	March 10, 2016	Date:	March 11, 2016

MEG Technical Services (MTS)

(A Division of MEG Consulting Limited)

Form Nº MTS107

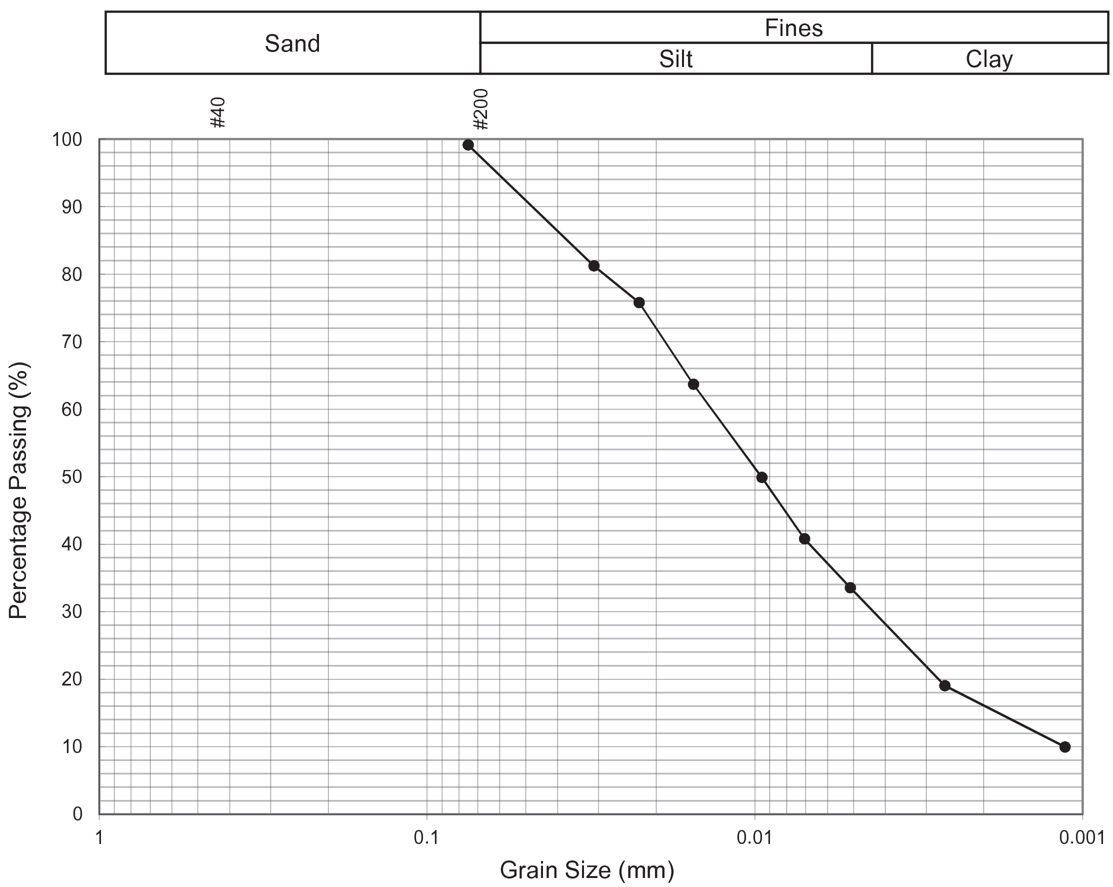
Marine + Earth MEG Geosciences

Project:	AMECFW - Myra Falls			Project No.:	15-MTS-029
Location:	BC			Date:	January 8, 2016
Borehole:	BH15-39	Sample No.:	SH-02	Depth (m):	12.52

Hydrometer Test (ASTM D422)

Unified Soil Classification System (ASTM D 2487)

Description of Material: Very dark gray lean CLAY



Sample No.	Depth (m)	Percentage of Material by Weight (%)			
		Gravel	Sand	Fines	
				Silt	Clay
SH-02	12.52	-	1	69	30

Comments: Clay description based on Atterberg limits result and ASTM flow chart

Prepared by:	PC	Checked by:	MF	Approved by:	PS
Date:	January 8, 2016	Date:	January 8, 2016	Date:	January 11, 2016

OQM | Organizational Quality Management Program

MEG Technical Services (MTS)

(A Division of MEG Consulting Limited)

Marine + Earth MEG Geosciences

Form N° MTS104

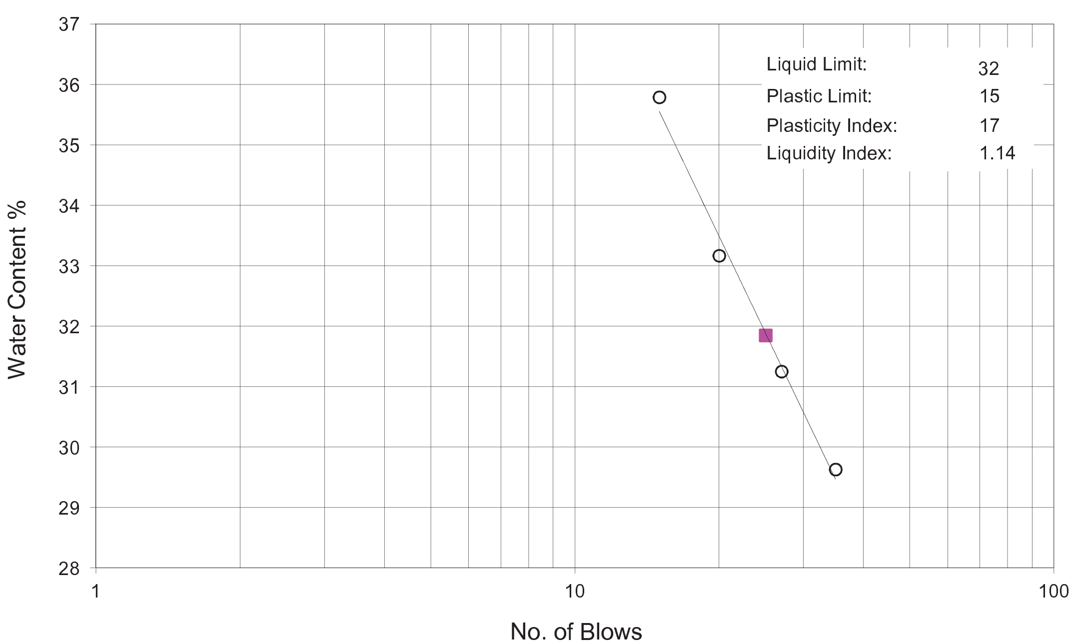
Project:	AMECFW - Myra Falls		Project No.:	15-MTS-029
Location:	BC		Date:	January 6, 2016
Borehole:	BH15-39	Sample No.: SH-02	Depth (m):	12.52

Liquid Limit, Plastic Limit and Plasticity Index of Soils (ASTM D4318)

LIQUID LIMIT								PLASTIC LIMIT						
TIN No.	Tare + Weight of Wet Soil (g)	Tare + Weight of Dry Soil (g)	Weight of Tin (g)	Weight of Water (g)	Weight of Dry Soil (g)	Water Content (%)	No. of Blows	TIN No.	Tare + Weight of Wet Soil (g)	Tare + Weight of Dry Soil (g)	Weight of Tin (g)	Weight of Water (g)	Weight of Dry Soil (g)	Water Content (%)
89A	43.02	40.25	32.51	2.77	7.74	35.8	15	56C	36.41	35.59	30.25	0.82	5.34	15.4
13	32.80	30.81	24.81	1.99	6.00	33.2	20	18E	35.87	35.06	29.78	0.81	5.28	15.3
68	45.98	43.38	35.06	2.60	8.32	31.2	27							
40	40.80	39.12	33.45	1.68	5.67	29.6	35							

Classification of the material : CL

100 % with respect to the total of the material smaller than sieve No. 40



Observations:

Prepared by: PC	Checked by: MF	Approved by: PS
Date: January 6, 2016	Date: January 6, 2016	Date: January 11, 2016

OQM Organizational Quality Management Program

MEG Technical Services (MTS)

(A Division of MEG Consulting Limited)

Form Nº MTS217a

Marine + Earth MEG Geosciences

Project:	AMECFW - Myra Falls	**Borehole:**	BH15-39	**Project No.:**	15-MTS-029
Client:	AMEC	**Sample No.:**	SH02	**Date:**	February 17, 2016
Location:	BC	**Depth (m):**	12.67	**Station:**	DSS1

Bender Element Velocity Measuring

WATER CONTENT & UNIT WEIGHT

	Initial	Final
Tin No.:	58	D1
Wt. of Tin (g):	25.44	116.86
Wet Weight (g):	101.53	332.43
Dry Weight (g):	83.84	289.30
Water Content (%):	30.3	25.0
Total Unit Weight (kN/m^3):	22.27	23.03
Dry Unit Weight (kN/m^3):	17.09	18.42

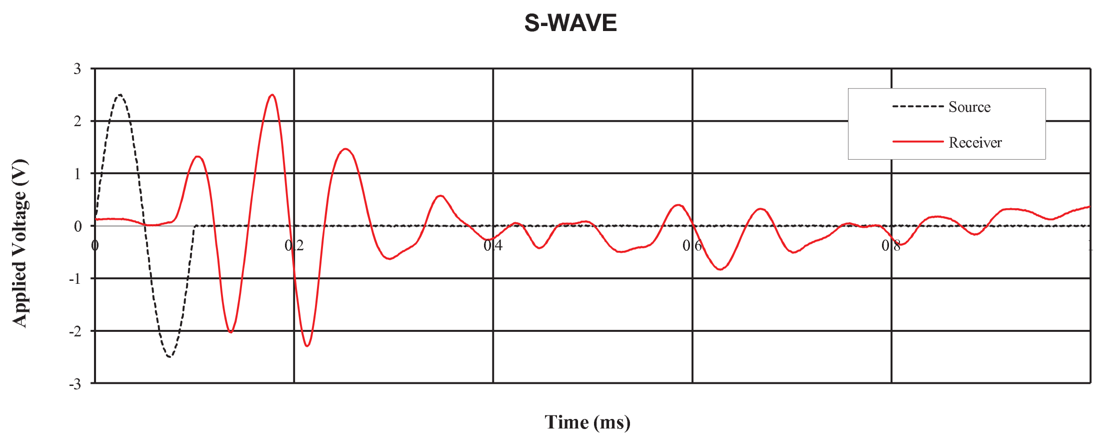
CONSOLIDATION

Specific Gravity, Gs:	3.44
Initial DSS Sample Height (mm):	23.4
Height after Consolidation (mm):	21.7
Initial Void Ratio, e_o	0.98
Initial Degree of Saturation (%)	>100
Final Void Ratio, e_f	0.83
Final Degree of Saturation (%)	>100

BENDER ELEMENTS

	S-Wave
Initial Time, T_o (ms):	0.026
Final Time, T_f (ms):	0.103
Travel Time (ms):	0.077
Wave Velocity (m/s):	282
Shear Modulus, G (MPa)	181.1
Vertical Effective Stress, s'$_v$ (kPa)	260
Maximum to Present Stress Ratio	-

WAVE TRAVEL CURVES



Comments: Test performed on sample consolidated to 260kPa and tested at a CRS of 0.19
*Vs is based on assessment of first shear wave arrival

Prepared By:	MF	Checked By:	PS	Approved By:	JPS
Date:	February 17, 2016	Date:	February 17, 2016	Date:	February 19, 2016

MEG Technical Services (MTS)

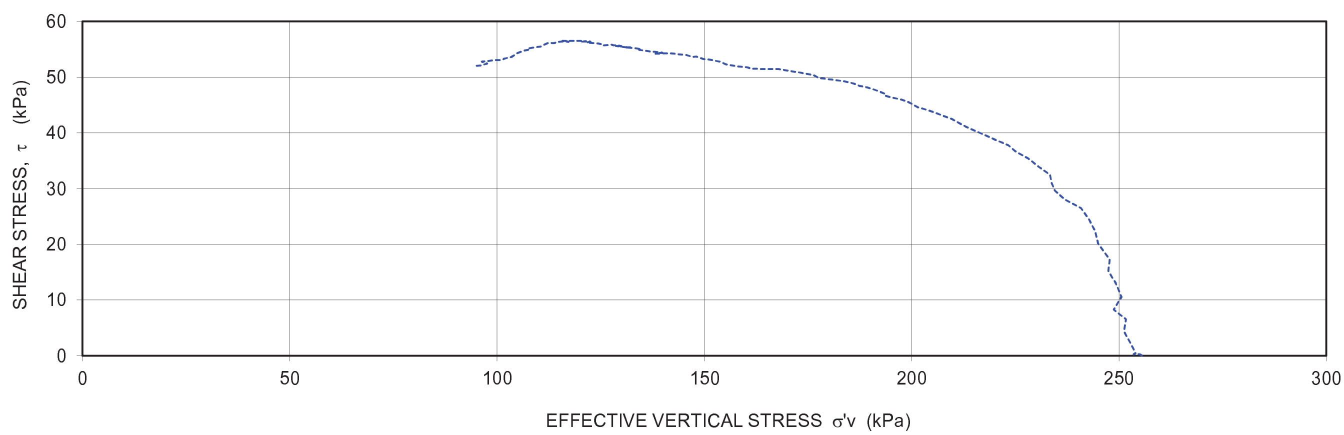
(A Division of MEG Consulting Limited)

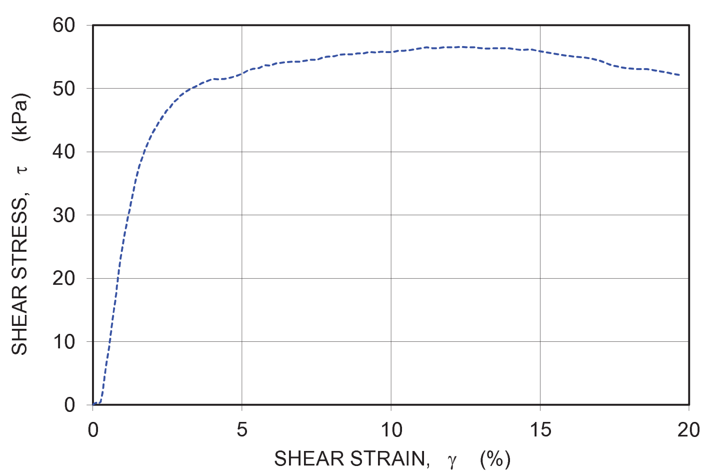
Marine + Earth MEG Geosciences

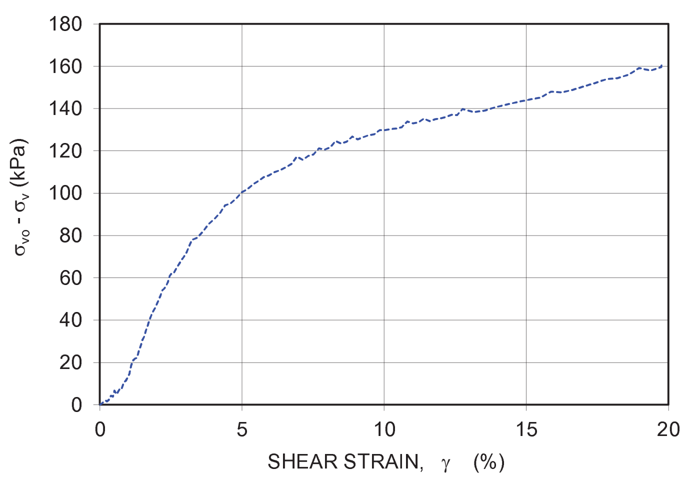
Project:	AMECFW - Myra Falls	Project No.:	15-MTS-029
Location:	BC	Date:	January 12, 2016
Borehole:	BH15-39	Depth (m):	12.52
Sample No.:	SH-02		

Direct Simple Shear (ASTM D6528)

Initial Height (mm):	22.4	**Weight of Specimen (g):**	160.23	**Initial Void Ratio, e_o:**	1.25
Diameter of Ring (mm):	66.5	**Total Unit Weight (kN/m^3):**	20.20	**Final Void Ratio, e_f:**	0.97
Specific Gravity, Gs:	3.44	**Dry Unit Weight (kN/m^3):**	15.04	**Natural Water Content (%):**	34.3
Final Water Content (%):	27.0	**Initial Degree of Saturation, Sr (%):**	94.8	**Final Degree of Saturation, Sr (%):**	95.6







Type of Test: Constant Volume			
Sample No.	Depth (m)	Total Unit Weight (kN/m^3)	Effective Vertical Stress, σ'v (kPa)
SH-02	12.52	20.2	260

Comments: ______________________

Prepared By:	MF	Checked By:	PS	Approved By:	JPS
Date:	January 12, 2016	Date:	January 13, 2016	Date:	January 15, 2016

OQM Organizational Quality Management Program

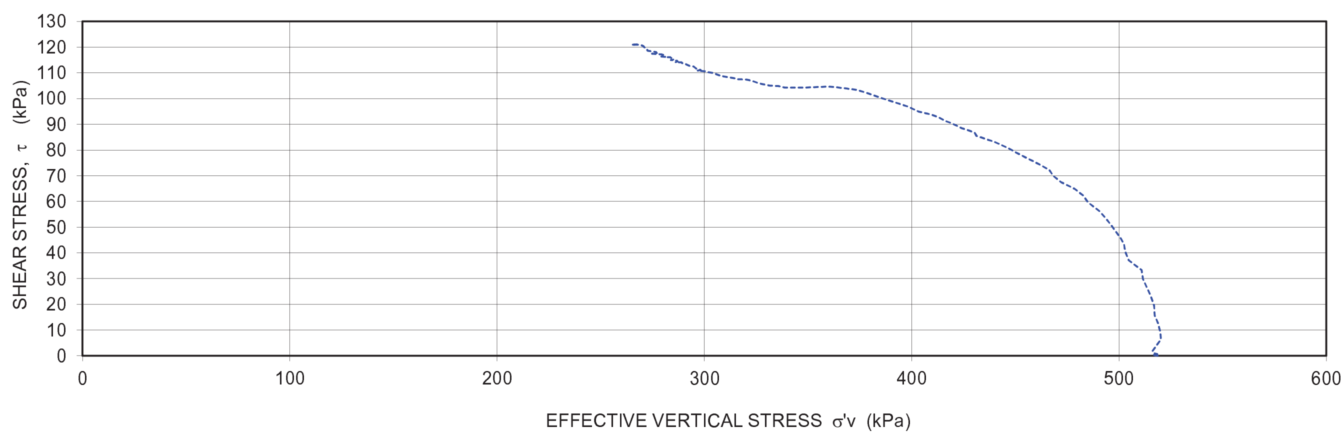
MEG Technical Services (MTS)
(A Division of MEG Consulting Limited)

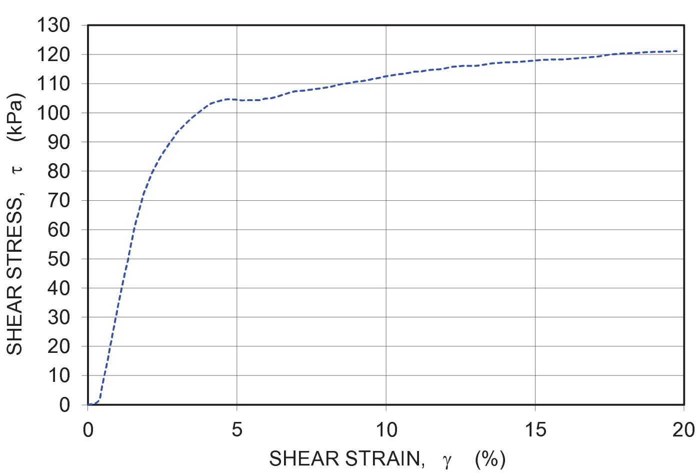
Marine + Earth MEG Geosciences

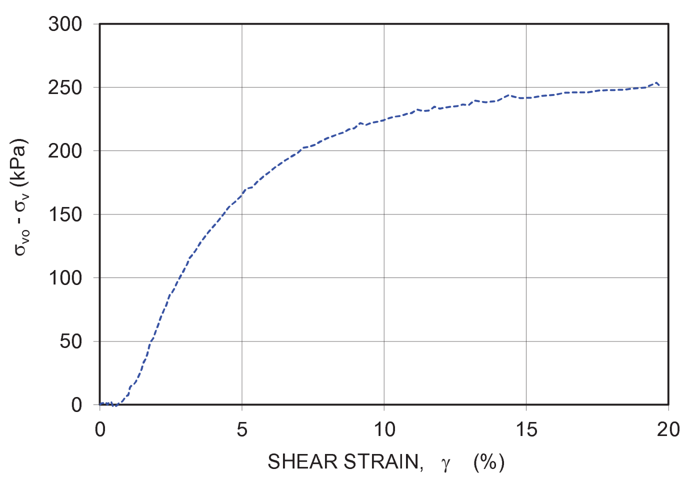
Project:	AMECFW - Myra Falls	Project No.:	15-MTS-029
Location:	BC	Date:	January 12, 2016
Borehole:	BH15-39	Depth (m):	15.48
Sample No.:	SH-02		

Direct Simple Shear (ASTM D6528)

Initial Height (mm):	22.4	**Weight of Specimen (g):**	164.18	**Initial Void Ratio, e_o:**	1.20
Diameter of Ring (mm):	66.5	**Total Unit Weight (kN/m^3):**	20.70	**Final Void Ratio, e_f:**	0.87
Specific Gravity, Gs:	3.44	**Dry Unit Weight (kN/m^3):**	15.36	**Natural Water Content (%):**	34.7
Final Water Content (%):	26.4	**Initial Degree of Saturation, Sr (%):**	99.9	**Final Degree of Saturation, Sr (%):**	>100







Type of Test: Constant Volume			
Sample No.	Depth (m)	Total Unit Weight (kN/m^3)	Effective Vertical Stress, σ'v (kPa)
SH-02	15.48	20.7	520

Comments: ______________________

Prepared By:	MF	Checked By:	PS	Approved By:	JPS
Date:	January 12, 2016	Date:	January 13, 2016	Date:	January 15, 2016

OQM | Organizational Quality Management Program

MEG Technical Services (MTS)

(A Division of MEG Consulting Limited)

Form Nº MTS210

Marine + Earth
MEG
Geosciences

Project:	AMECFW - Myra Falls			Project No.:	15-MTS-029
Location:	BC	Borehole:	BH15-39	Depth:	12.67 m
Sample:	SH-02	Station:	DSS1	Date:	March 10, 2016

Stress Controlled Cyclic Direct Simple Shear Test

0.11 stress ratio (τ_{cyc}/ σ'_{vc}) @ 0.1 Hz for 30 cycles, σ'_{vc}=260kPa

Initial sample Details		Final Sample Details	
Water Content (%):	26.1	Water Content (%):	25.9
Diameter (mm):	73.23	Diameter (mm):	73.23
Height (mm):	23.52	Change in Height, ΔH (mm):	1.42
Specific Gravity, Gs:	3.44	Final Height (mm):	22.10
Weight of Soil (g):	218.79	Weight of Soil (g):	218.49
Total Unit Weight (kN/m^3)	21.67	Total Unit Weight (kN/m^3)	23.03
Dry Unit Weight (kN/m^3)	17.18	Dry Unit Weight (kN/m^3)	18.29
Initial Void Ratio	0.97	Final Void Ratio	0.85

0.19 stress ratio (τ_{cyc}/ σ'_{vc}) @ 0.1 Hz for 3 cycles, σ'_{vc}=260kPa

Initial sample Details		Final Sample Details	
Water Content (%):	30.3	Water Content (%):	25.0
Diameter (mm):	73.29	Diameter (mm):	73.29
Height (mm):	23.44	Change in Height, ΔH (mm):	1.69
Specific Gravity, Gs:		Final Height (mm):	21.75
Weight of Soil (g):	224.51	Weight of Soil (g):	215.41
Total Unit Weight (kN/m^3)	22.27	Total Unit Weight (kN/m^3)	23.03
Dry Unit Weight (kN/m^3)	17.09	Dry Unit Weight (kN/m^3)	18.42
Initial Void Ratio		Final Void Ratio	

0.15 stress ratio (τ_{cyc}/ σ'_{vc}) @ 0.1 Hz for 15 cycles, σ'_{vc}=260kPa

Initial sample Details		Final Sample Details	
Water Content (%):	26.2	Water Content (%):	23.1
Diameter (mm):	73.12	Diameter (mm):	73.12
Height (mm):	23.50	Change in Height, ΔH (mm):	0.85
Specific Gravity, Gs:	3.44	Final Height (mm):	22.65
Weight of Soil (g):	234.63	Weight of Soil (g):	228.81
Total Unit Weight (kN/m^3)	23.32	Total Unit Weight (kN/m^3)	23.60
Dry Unit Weight (kN/m^3)	18.48	Dry Unit Weight (kN/m^3)	19.18
Initial Void Ratio	0.83	Final Void Ratio	0.76

Sample Description:

Prepared By:	MF	Checked By:	PS	Approved By:	JPS
Date:	March 10, 2016	Date:	March 10, 2016	Date:	March 11, 2016

MEG TECHNICAL SERVICES
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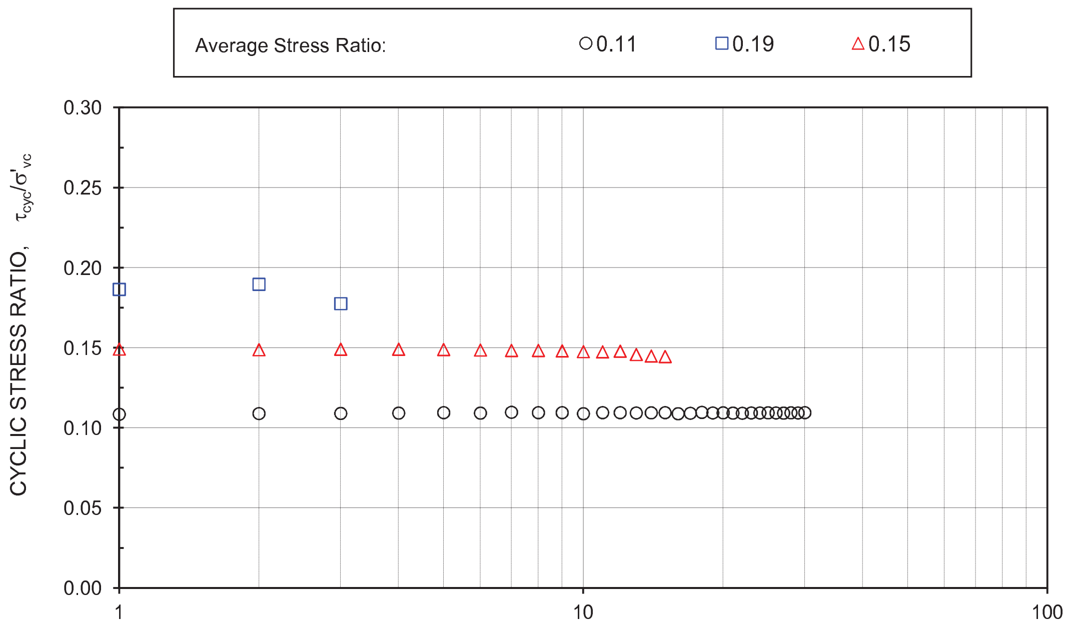
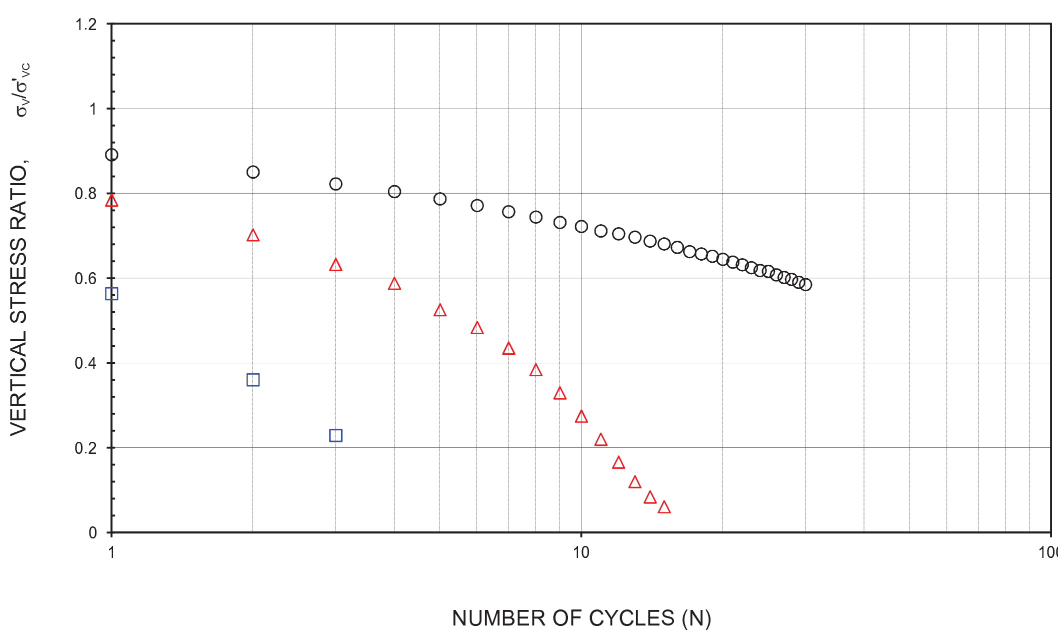
Marine + Earth MEG Geosciences

Project:	AMECFW - Myra Falls			Project No.:	15-MTS-029
Location:	BC	Borehole:	BH15-39	Depth:	12.67 m
Sample:	SH-02	Station:	DSS1	Date:	March 10, 2016

STRESS CONTROLLED CYCLIC DIRECT SIMPLE SHEAR TEST





Prepared By:	MF	Checked By:	PS	Approved By:	JPS
Date:	March 10, 2016	Date:	March 10, 2016	Date:	March 11, 2016

OQM | Organizational Quality Management Program

MEG TECHNICAL SERVICES

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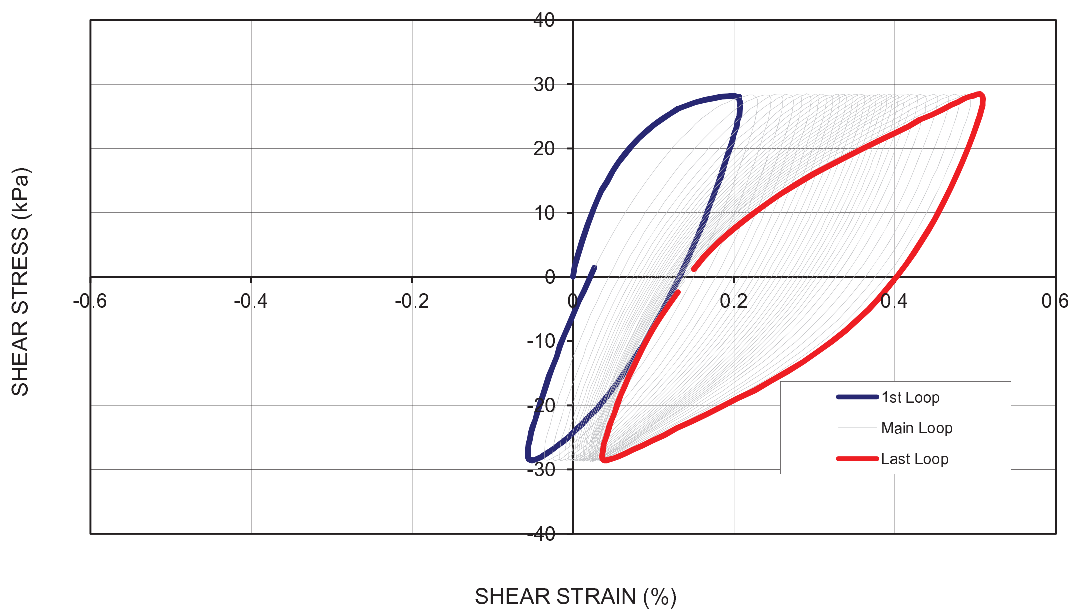
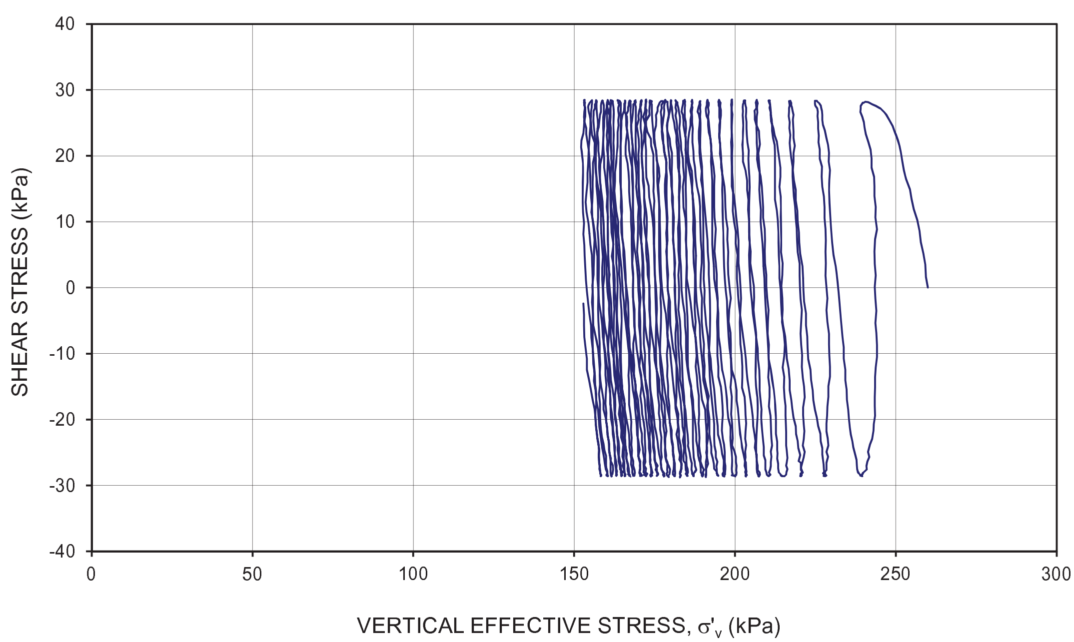
Marine + Earth Geosciences

Project:	AMECFW - Myra Falls			Project No.:	15-MTS-029
Location:	BC	Borehole:	BH15-39	Depth:	12.67 m
Sample:	SH-02	Station:	DSS1	Date:	March 10, 2016

STRESS CONTROLLED CYCLIC DIRECT SIMPLE SHEAR TEST

0.11 stress ratio (τ_{cyc}/ σ'_{vc}) @ 0.1 Hz for 30 cycles, σ'_{vc}=260kPa





Prepared By:	MF	Checked By:	PS	Approved By:	JPS
Date:	March 10, 2016	Date:	March 10, 2016	Date:	March 11, 2016

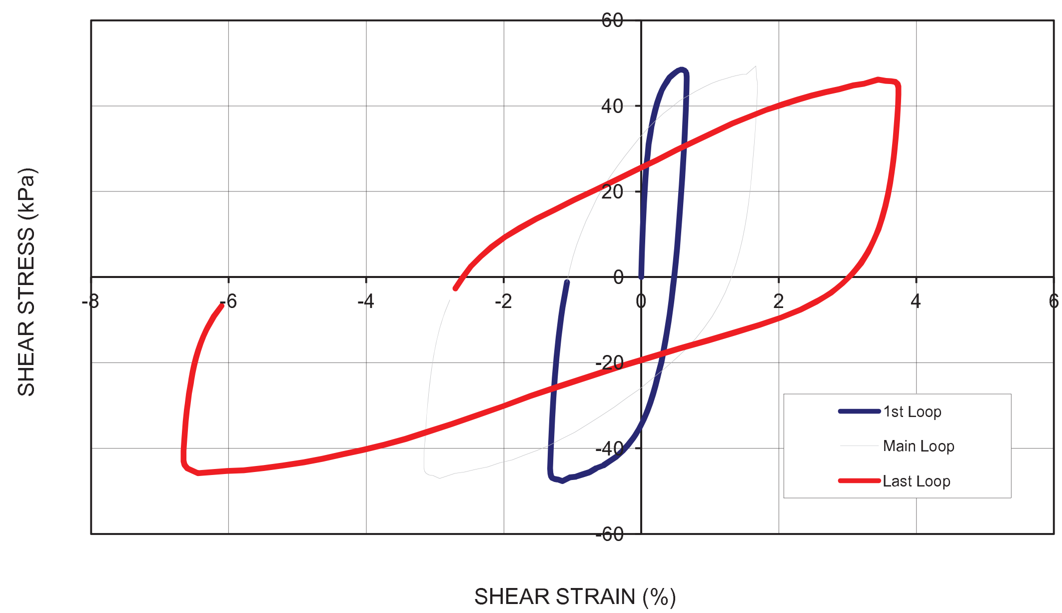
MEG TECHNICAL SERVICES
(A Division of MEG Consulting Limited)

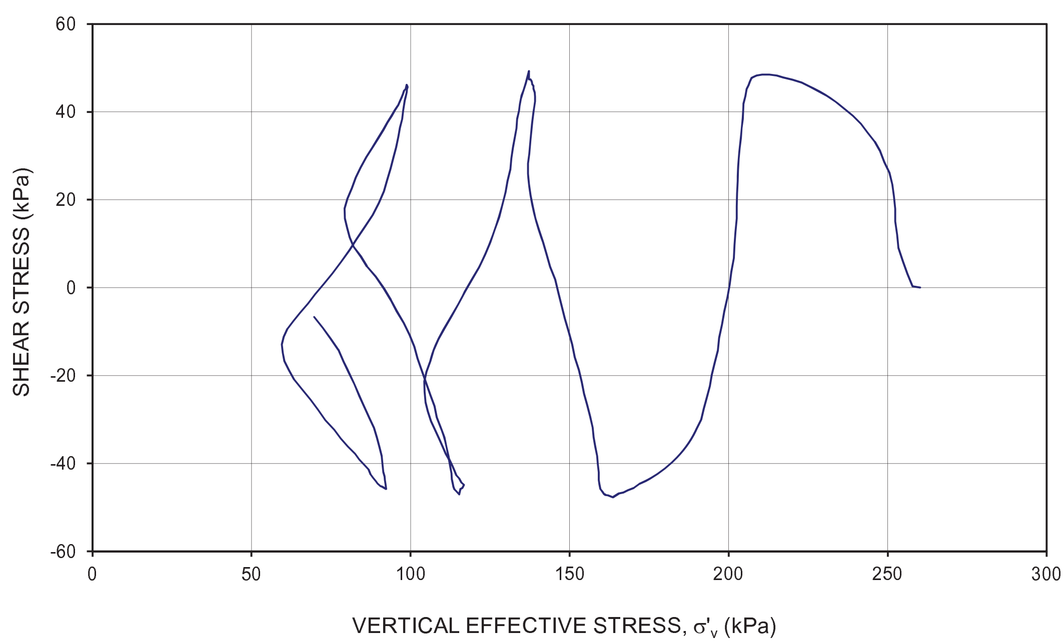
Marine + Earth MEG Geosciences

Project:	AMECFW - Myra Falls			Project No.:	15-MTS-029
Location:	BC	Borehole:	BH15-39	Depth:	12.67 m
Sample:	SH-02	Station:	DSS1	Date:	March 10, 2016

STRESS CONTROLLED CYCLIC DIRECT SIMPLE SHEAR TEST

0.19 stress ratio (τ_{cyc}/ σ'_{vc}) @ 0.1 Hz for 3 cycles, σ'_{vc}=260kPa





Prepared By:	MF	Checked By:	PS	Approved By:	JPS
Date:	March 10, 2016	Date:	March 10, 2016	Date:	March 11, 2016

MEG TECHNICAL SERVICES
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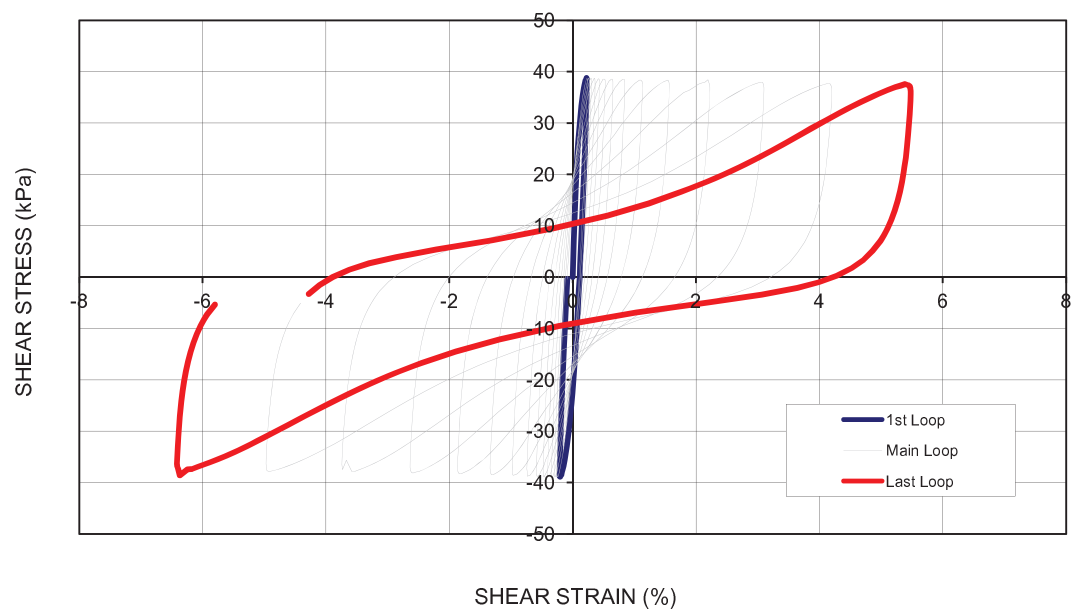
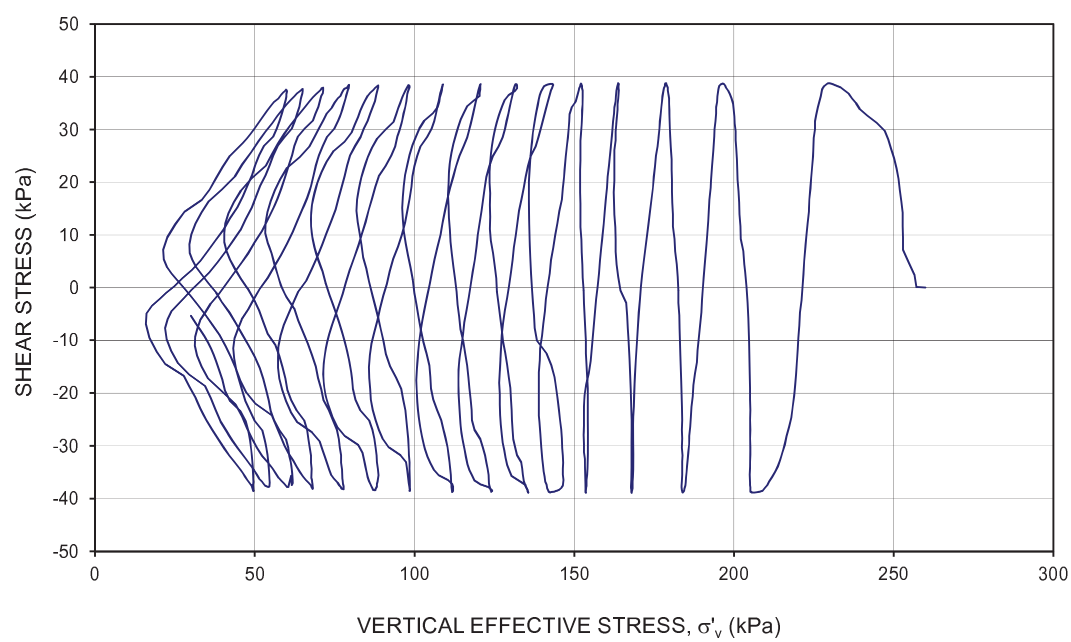
Marine + Earth
MEG
Geosciences

Project:	AMECFW - Myra Falls			Project No.:	15-MTS-029
Location:	BC	Borehole:	BH15-39	Depth:	12.67 m
Sample:	SH-02	Station:	DSS1	Date:	March 10, 2016

STRESS CONTROLLED CYCLIC DIRECT SIMPLE SHEAR TEST

0.15 stress ratio (τ_{cyc}/ σ'_{vc}) @ 0.1 Hz for 15 cycles, σ'_{vc}=400kPa





Prepared By:	MF	Checked By:	PS	Approved By:	JPS
Date:	March 10, 2016	Date:	March 10, 2016	Date:	March 11, 2016

OQM | Organizational Quality Management Program

MEG Technical Services (MTS)

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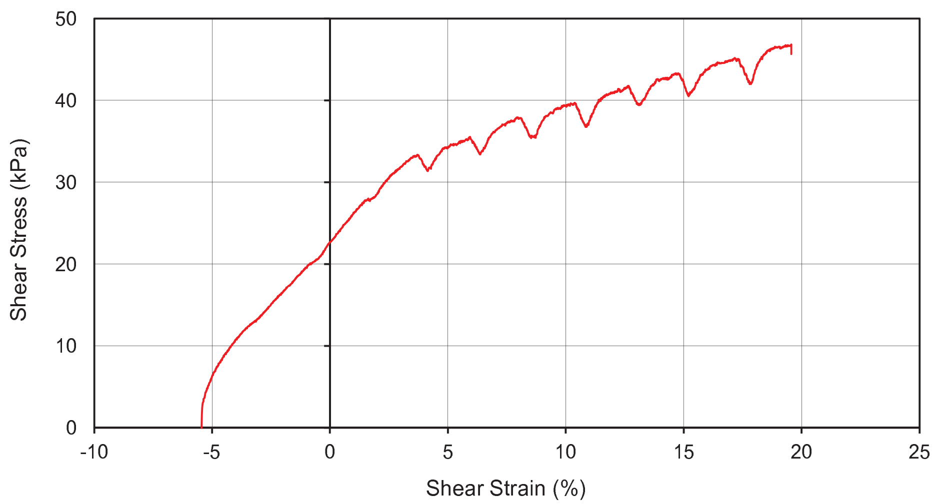
Form N° MTS214

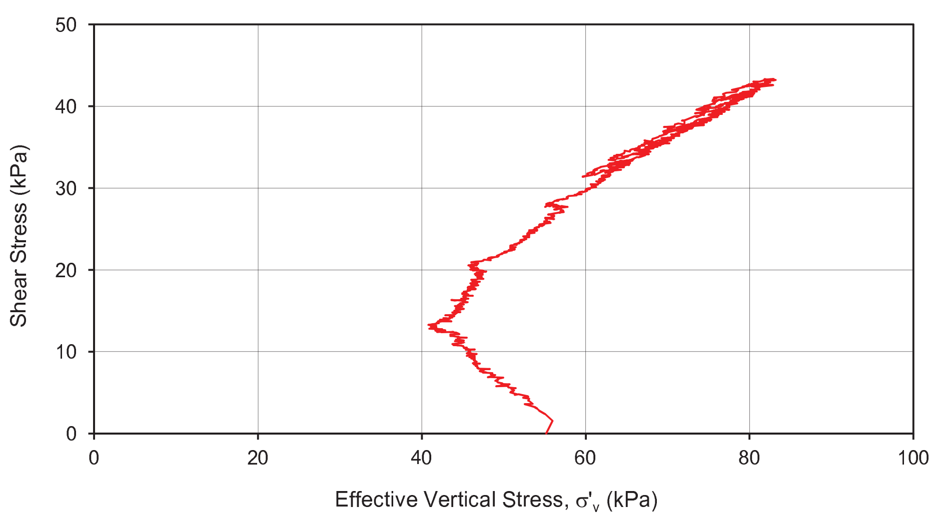
Marine + Earth MEG Geosciences

Project:	AMECFW - Myra Falls			Project No.:	15-MTS-029
Location:	BC	Borehole:	BH15-39	Depth:	12.67 m
Sample:	SH02	Station:	DSS1	Date:	March 10, 2016

Post-Cyclic Static Direct Simple Shear Test

POST-CYCLIC STATIC SHEAR TEST





Note: Test performed after stress-controlled DSS test at average cyclic stress ratio, CSR = 0.19 with 77% excess pore pressure

Prepared By:	MF	Checked By:	PS	Approved By:	JPS
Date:	March 10, 2016	Date:	March 10, 2016	Date:	March 11, 2016

MEG Technical Services (MTS)
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Marine + Earth
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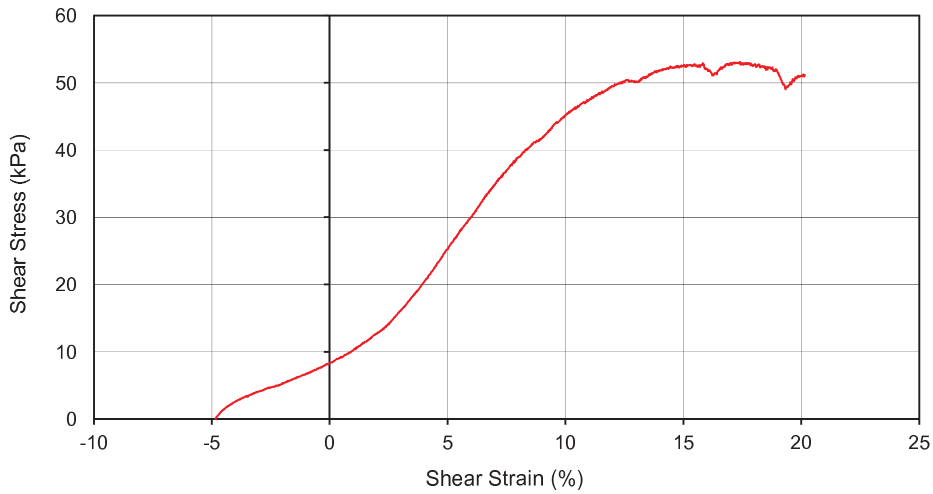
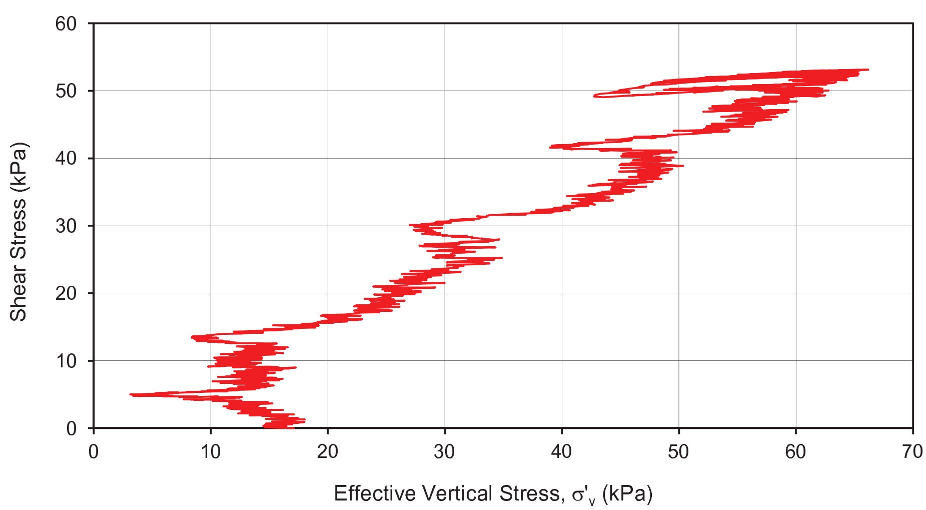
Geosciences

Project:	AMECFW - Myra Falls			Project No.:	15-MTS-029
Location:	BC	Borehole:	BH15-39	Depth:	12.67 m
Sample:	SH02	Station:	DSS1	Date:	March 10, 2016

Post-Cyclic Static Direct Simple Shear Test

POST-CYCLIC STATIC SHEAR TEST





Note: Test performed after stress-controlled DSS test at average cyclic stress ratio, CSR = 0.22 with 87% excess pore pressure

Prepared By:	MF	Checked By:	PS	Approved By:	JPS
Date:	March 10, 2016	Date:	March 10, 2016	Date:	March 11, 2016

MEG Technical Services (MTS)

(A Division of MEG Consulting Limited)

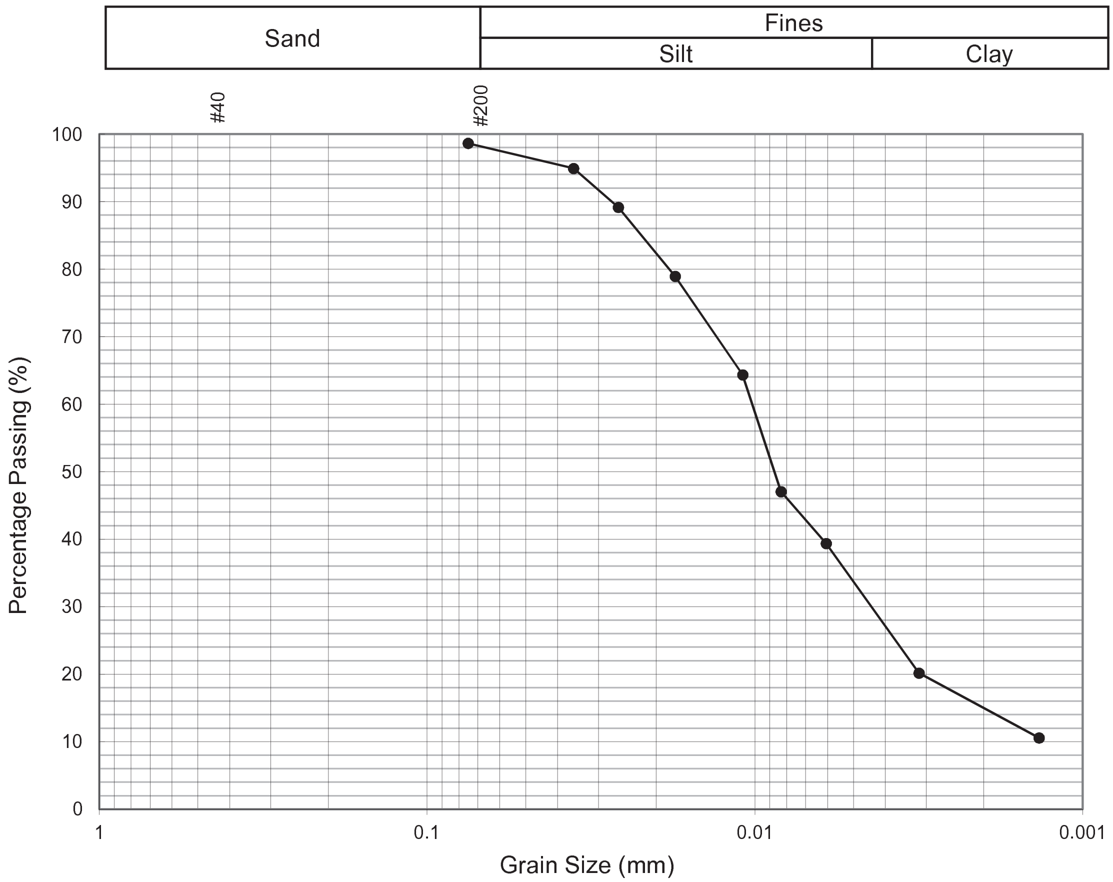
Form Nº MTS107

Project:	AMECFW - Myra Falls	Project No.:	15-MTS-029
Location:	BC	Date:	January 11, 2016
Borehole:	BH15-39 Sample No.: SH-03	Depth (m):	19.33

Hydrometer Test (ASTM D422)

Unified Soil Classification System (ASTM D 2487)

Description of Material: Very dark gray lean CLAY



Sample No.	Depth (m)	Percentage of Material by Weight (%)			
		Gravel	Sand	Fines	
				Silt	Clay
SH-03	19.33	-	1	71	28

Comments: Clay description based on Atterberg limits result and ASTM flow chart

Prepared by:	MF	Checked by:	PC	Approved by:	PS
Date:	January 11, 2016	Date:	January 11, 2016	Date:	January 15, 2016

MEG Technical Services (MTS)

(A Division of MEG Consulting Limited)

Form N° MTS104

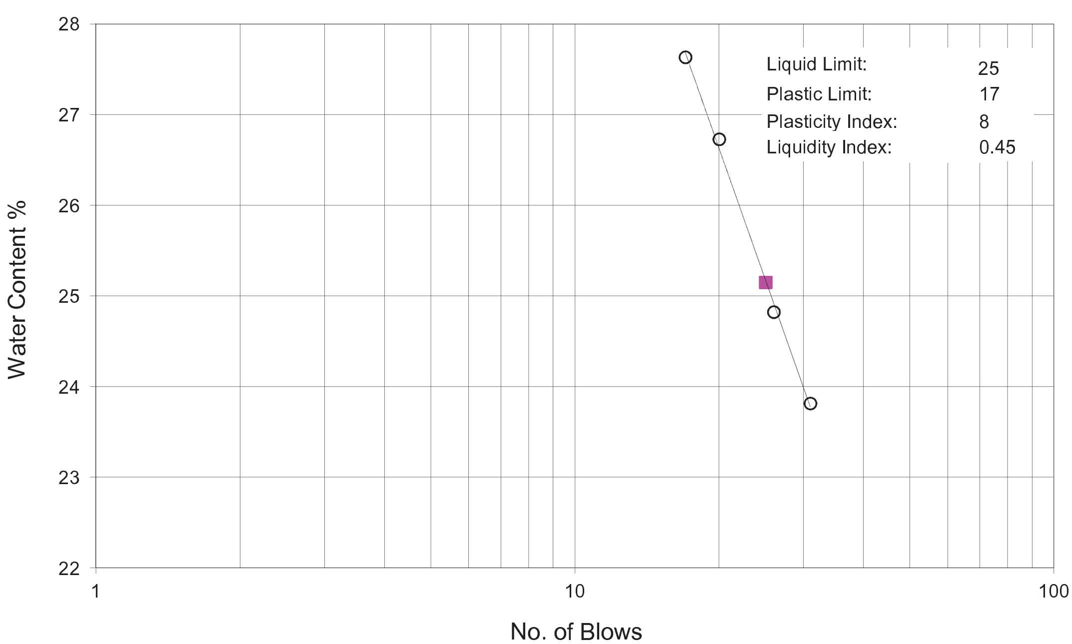
Project: AMECFW - Myra Falls		Project No.:	15-MTS-029
Location: BC		Date:	January 8, 2016
Borehole: BH15-39	Sample No.: SH-03	Depth (m):	19.40

Liquid Limit, Plastic Limit and Plasticity Index of Soils (ASTM D4318)

LIQUID LIMIT								PLASTIC LIMIT						
TIN No.	Tare + Weight of Wet Soil (g)	Tare + Weight of Dry Soil (g)	Weight of Tin (g)	Weight of Water (g)	Weight of Dry Soil (g)	Water Content (%)	No. of Blows	TIN No.	Tare + Weight of Wet Soil (g)	Tare + Weight of Dry Soil (g)	Weight of Tin (g)	Weight of Water (g)	Weight of Dry Soil (g)	Water Content (%)
14A	33.25	31.44	24.89	1.81	6.55	27.6	17	56C	36.56	35.63	30.25	0.93	5.38	17.3
40	45.52	42.97	33.43	2.55	9.54	26.7	20	7E	36.58	35.65	30.20	0.93	5.45	17.1
7	40.40	39.00	33.36	1.40	5.64	24.8	26							
89A	43.48	41.37	32.51	2.11	8.86	23.8	31							

Classification of the material : CL

100 % with respect to the total of the material smaller than sieve No. 40



Observations:

Prepared by:	PC	Checked by:	MF	Approved by:	PS
Date:	January 8, 2016	Date:	January 8, 2016	Date:	January 11, 2016

MEG Technical Services (MTS)

(A Division of MEG Consulting Limited)

Form Nº MTS217a

Marine + Earth MEG Geosciences

Project:	AMECFW - Myra Falls	**Borehole:**	BH15-39	**Project No.:**	15-MTS-029
Client:	AMEC	**Sample No.:**	SH02	**Date:**	February 17, 2016
Location:	BC	**Depth (m):**	12.67	**Station:**	DSS1

Bender Element Velocity Measuring

WATER CONTENT & UNIT WEIGHT

	Initial	Final
Tin No.:	58	D1
Wt. of Tin (g):	25.44	116.86
Wet Weight (g):	101.53	332.43
Dry Weight (g):	83.84	289.30
Water Content (%):	30.3	25.0
Total Unit Weight (kN/m^3):	22.27	23.03
Dry Unit Weight (kN/m^3):	17.09	18.42

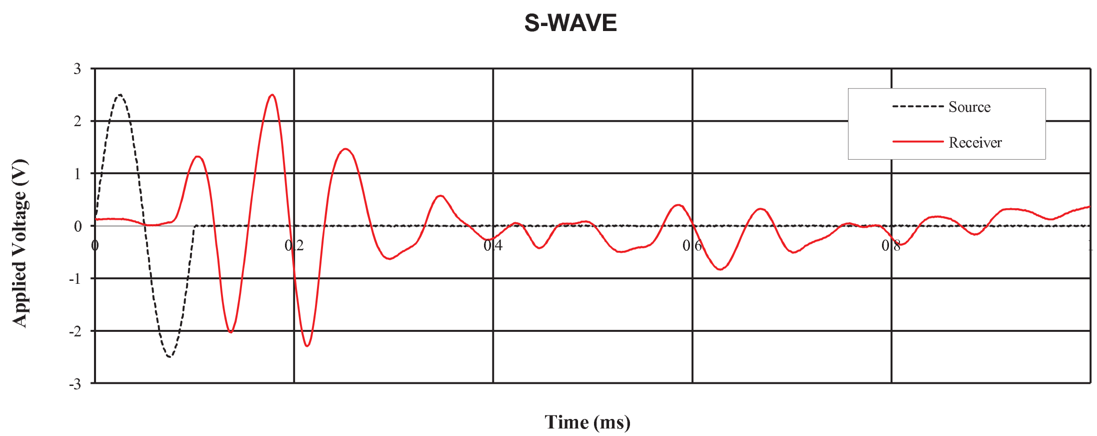
CONSOLIDATION

Specific Gravity, Gs:	3.44
Initial DSS Sample Height (mm):	23.4
Height after Consolidation (mm):	21.7
Initial Void Ratio, e_o	0.98
Initial Degree of Saturation (%)	>100
Final Void Ratio, e_f	0.83
Final Degree of Saturation (%)	>100

BENDER ELEMENTS

	S-Wave
Initial Time, T_o (ms):	0.026
Final Time, T_f (ms):	0.103
Travel Time (ms):	0.077
Wave Velocity (m/s):	282
Shear Modulus, G (MPa)	181.1
Vertical Effective Stress, s'$_v$ (kPa)	260
Maximum to Present Stress Ratio	-

WAVE TRAVEL CURVES



Comments: Test performed on sample consolidated to 260kPa and tested at a CRS of 0.19

*Vs is based on assessment of first shear wave arrival

Prepared By:	MF	Checked By:	PS	Approved By:	JPS
Date:	February 17, 2016	Date:	February 17, 2016	Date:	February 19, 2016

MEG Technical Services (MTS)

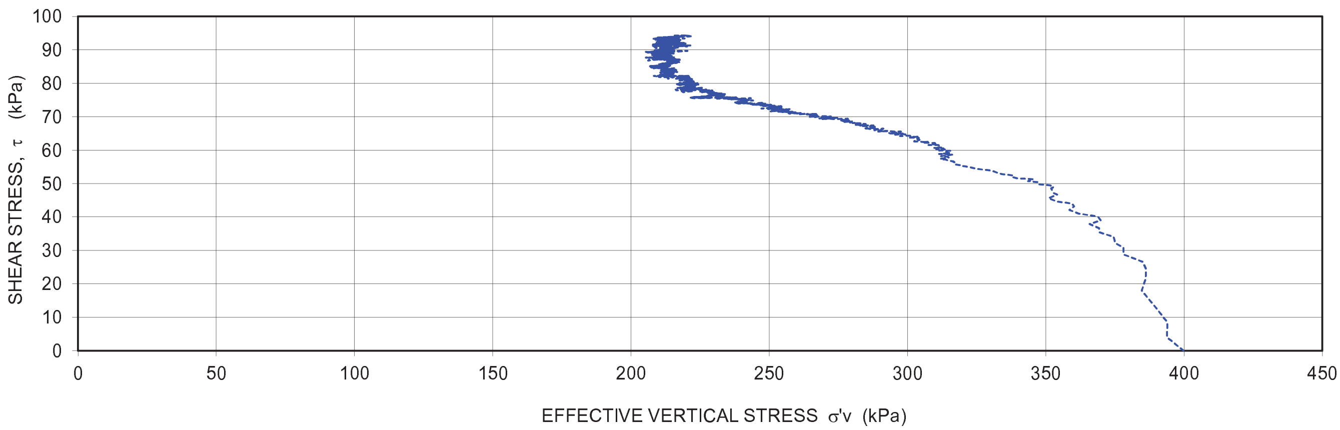
(A Division of MEG Consulting Limited)

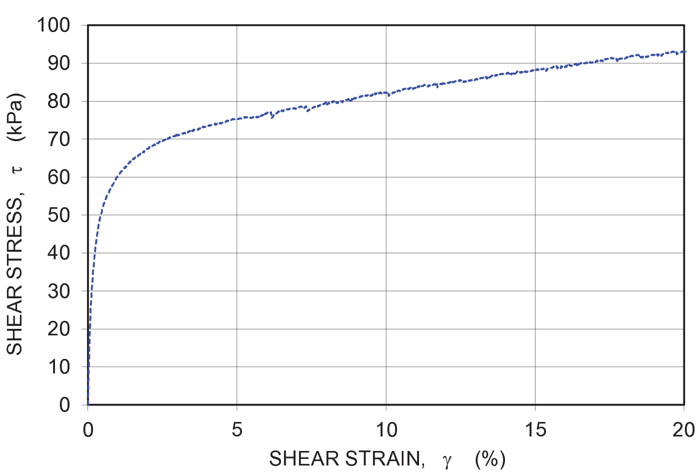
Marine + Earth MEG Geosciences

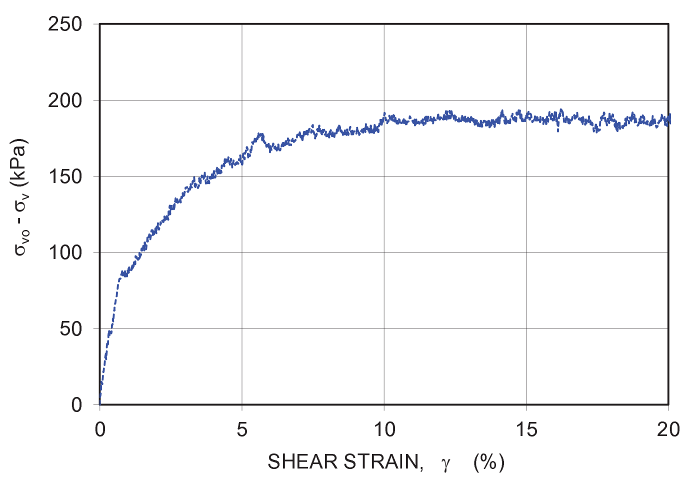
Project:	AMECFW - Myra Falls	Project No.:	15-MTS-029
Location:	BC	Date:	January 13, 2016
Borehole:	BH15-39	Depth (m):	19.32
Sample No.:	SH03		

Direct Simple Shear (ASTM D6528)

Initial Height (mm):	23.5	**Weight of Specimen (g):**	211.32	**Initial Void Ratio, e_o:**	1.26
Diameter of Ring (mm):	73.2	**Total Unit Weight (kN/m^3):**	20.97	**Final Void Ratio, e_f:**	1.09
Specific Gravity, Gs:	3.62	**Dry Unit Weight (kN/m^3):**	15.69	**Natural Water Content (%):**	33.6
Final Water Content (%):	32.2	**Initial Degree of Saturation, Sr (%):**	96.4	**Final Degree of Saturation, Sr (%):**	>100







Type of Test: Constant Volume					
Sample No.	Depth (m)	Total Unit Weight (kN/m^3)	Effective Vertical Stress, σ'v (kPa)	Strain Rate (%/hour)	Test OCR
SH03	19.32	21.0	400	5	-

Comments:

Prepared By:	MF	Checked By:	PS	Approved By:	JPS
Date:	January 13, 2016	Date:	January 13, 2016	Date:	January 15, 2016

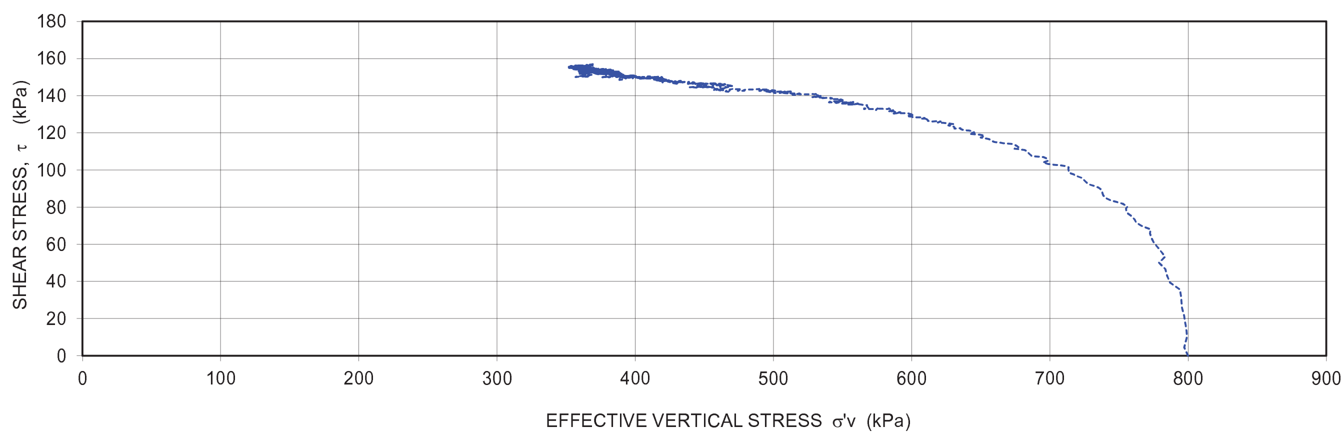
MEG Technical Services (MTS)
(A Division of MEG Consulting Limited)

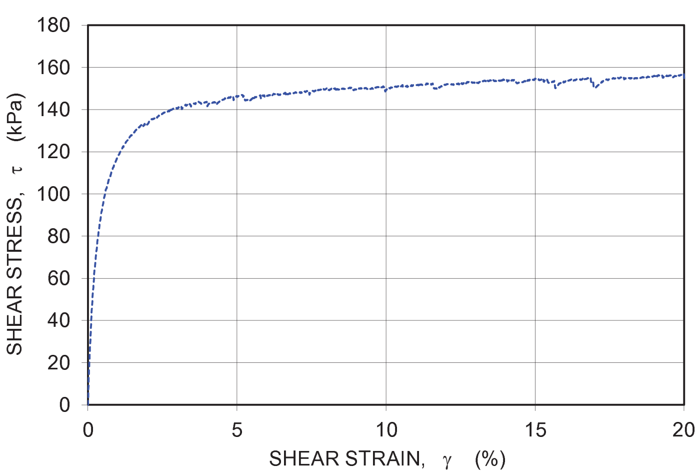
Marine + Earth MEG Geosciences

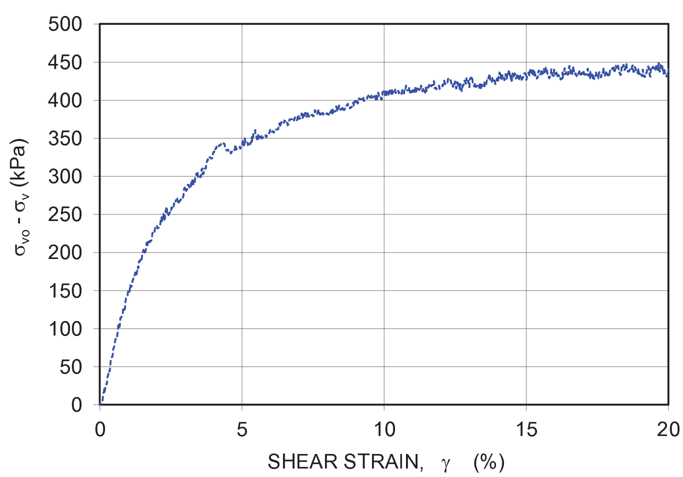
Project:	AMECFW - Myra Falls	Project No.:	15-MTS-029
Location:	BC	Date:	January 13, 2016
Borehole:	BH15-39	Depth (m):	19.37
Sample No.:	SH03		

Direct Simple Shear (ASTM D6528)

Initial Height (mm):	23.4	**Weight of Specimen (g):**	214.84	**Initial Void Ratio, e_o:**	1.20
Diameter of Ring (mm):	73.1	**Total Unit Weight (kN/m^3):**	21.44	**Final Void Ratio, e_f:**	0.98
Specific Gravity, Gs:	3.62	**Dry Unit Weight (kN/m^3):**	16.16	**Natural Water Content (%):**	32.7
Final Water Content (%):	32.0	**Initial Degree of Saturation, Sr (%):**	98.7	**Final Degree of Saturation, Sr (%):**	>100







Type of Test: Constant Volume					
Sample No.	Depth (m)	Total Unit Weight (kN/m^3)	Effective Vertical Stress, σ'v (kPa)	Strain Rate (%/hour)	Test OCR
SH03	19.37	21.4	800	5	-

Comments:

Prepared By:	MF	Checked By:	PS	Approved By:	JPS
Date:	January 13, 2016	Date:	January 13, 2016	Date:	January 15, 2016

OQM Organizational Quality Management Program

MEG Technical Services (MTS)

(A Division of MEG Consulting Limited)

Form Nº MTS210

Marine + Earth
MEG
Geosciences

Project:	AMECFW - Myra Falls			Project No.:	15-MTS-029
Location:	BC	Borehole:	BH15-39	Depth:	18.76 m
Sample:	SH03	Station:	DSS1	Date:	March 10, 2016

Stress Controlled Cyclic Direct Simple Shear Test

0.095 stress ratio (τ_{cyc}/ σ'_{vc}) @ 0.1 Hz for 30 cycles, σ'_{vc}=400kPa

Initial sample Details		Final Sample Details	
Water Content (%):	27.4	Water Content (%):	21.3
Diameter (mm):	73.24	Diameter (mm):	73.24
Height (mm):	23.51	Change in Height, ΔH (mm):	1.27
Specific Gravity, Gs:	3.62	Final Height (mm):	22.24
Weight of Soil (g):	217.09	Weight of Soil (g):	206.69
Total Unit Weight (kN/m^3)	21.50	Total Unit Weight (kN/m^3)	21.64
Dry Unit Weight (kN/m^3)	16.88	Dry Unit Weight (kN/m^3)	17.83
Initial Void Ratio	1.11	Final Void Ratio	0.99

0.13 stress ratio (τ_{cyc}/ σ'_{vc}) @ 0.1 Hz for 30 cycles, σ'_{vc}=400kPa

Initial sample Details		Final Sample Details	
Water Content (%):	24.4	Water Content (%):	23.6
Diameter (mm):	73.19	Diameter (mm):	73.19
Height (mm):	23.47	Change in Height, ΔH (mm):	1.79
Specific Gravity, Gs:	3.62	Final Height (mm):	21.68
Weight of Soil (g):	222.15	Weight of Soil (g):	220.83
Total Unit Weight (kN/m^3)	22.07	Total Unit Weight (kN/m^3)	23.75
Dry Unit Weight (kN/m^3)	17.74	Dry Unit Weight (kN/m^3)	19.21
Initial Void Ratio	1.00	Final Void Ratio	0.85

0.17 stress ratio (τ_{cyc}/ σ'_{vc}) @ 0.1 Hz for 4 cycles, σ'_{vc}=400kPa

Initial sample Details		Final Sample Details	
Water Content (%):	27.5	Water Content (%):	25.7
Diameter (mm):	73.10	Diameter (mm):	73.10
Height (mm):	23.49	Change in Height, ΔH (mm):	1.73
Specific Gravity, Gs:	3.62	Final Height (mm):	21.76
Weight of Soil (g):	217.50	Weight of Soil (g):	214.45
Total Unit Weight (kN/m^3)	21.64	Total Unit Weight (kN/m^3)	23.03
Dry Unit Weight (kN/m^3)	16.98	Dry Unit Weight (kN/m^3)	18.33
Initial Void Ratio	1.09	Final Void Ratio	0.94

Sample Description:

Prepared By:	MF	Checked By:	PS	Approved By:	JPS
Date:	March 10, 2016	Date:	March 10, 2016	Date:	March 11, 2016

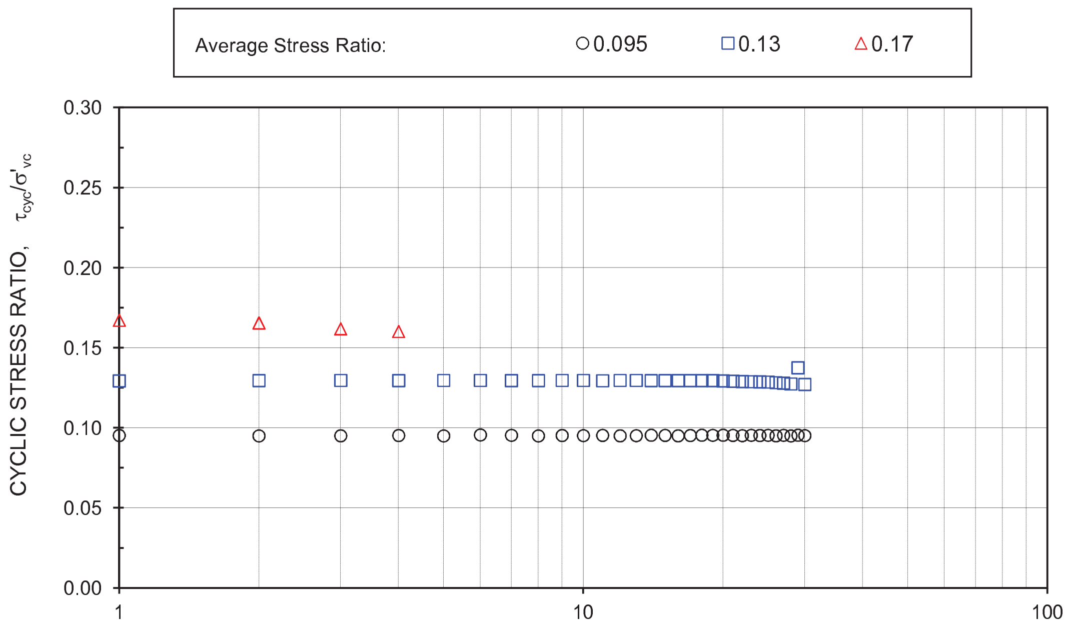
MEG TECHNICAL SERVICES

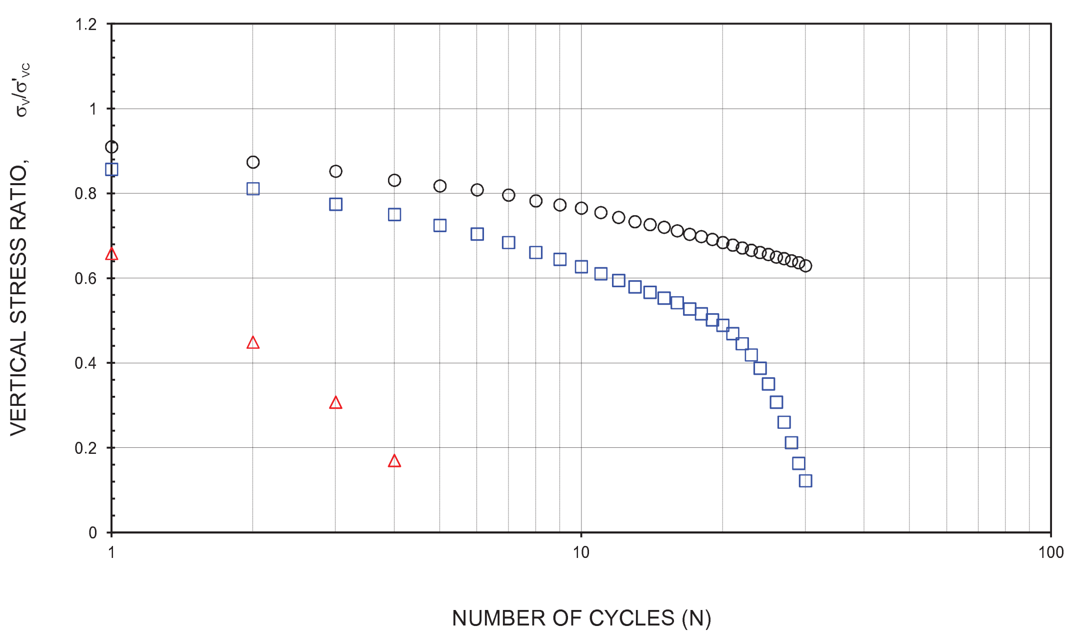
(A Division of MEG Consulting Limited)

Marine + Earth
MEG
Geosciences

Project:	AMECFW - Myra Falls			Project No.:	15-MTS-029
Location:	BC	Borehole:	BH15-39	Depth:	18.76 m
Sample:	SH03	Station:	DSS1	Date:	March 10, 2016

STRESS CONTROLLED CYCLIC DIRECT SIMPLE SHEAR TEST





Prepared By:	MF	Checked By:	PS	Approved By:	JPS
Date:	March 10, 2016	Date:	March 10, 2016	Date:	March 11, 2016

MEG TECHNICAL SERVICES

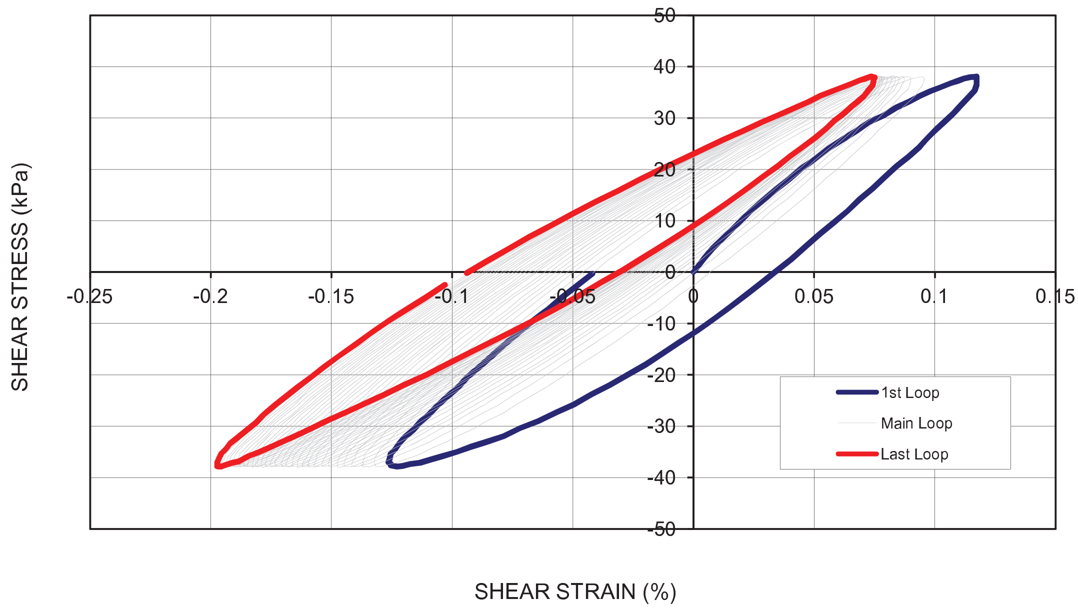
(A Division of MEG Consulting Limited)

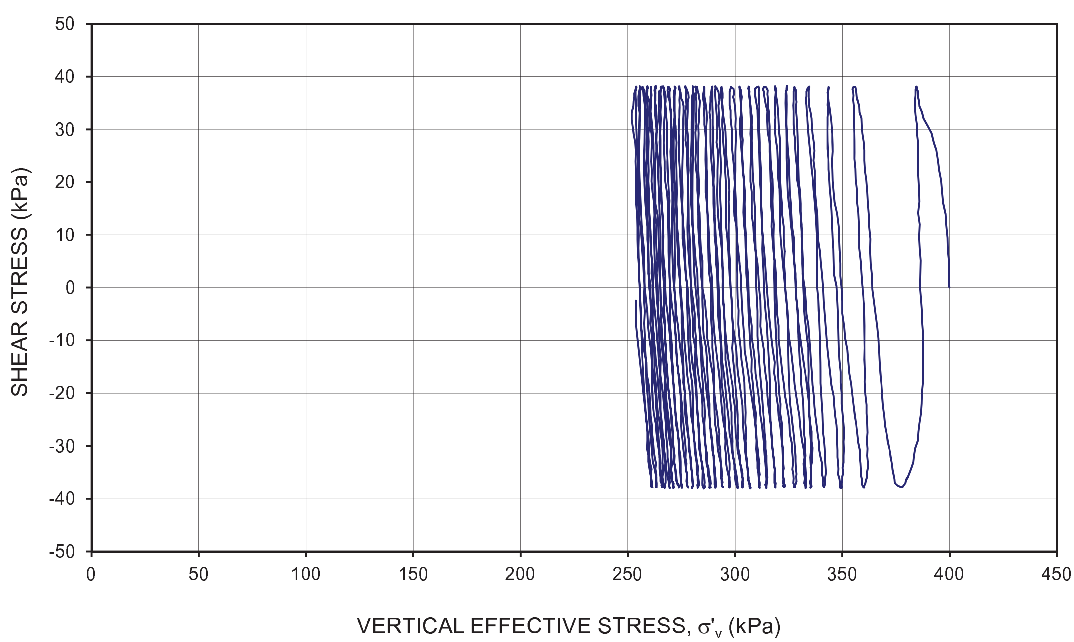
Marine + Earth
MEG
Geosciences

Project:	AMECFW - Myra Falls			Project No.:	15-MTS-029
Location:	BC	Borehole:	BH15-39	Depth:	18.76 m
Sample:	SH03	Station:	DSS1	Date:	March 10, 2016

STRESS CONTROLLED CYCLIC DIRECT SIMPLE SHEAR TEST

0.095 stress ratio (τ_{cyc}/ σ'_{vc}) @ 0.1 Hz for 30 cycles, σ'_{vc}=400kPa





Prepared By:	MF	Checked By:	PS	Approved By:	JPS
Date:	March 10, 2016	Date:	March 10, 2016	Date:	March 11, 2016

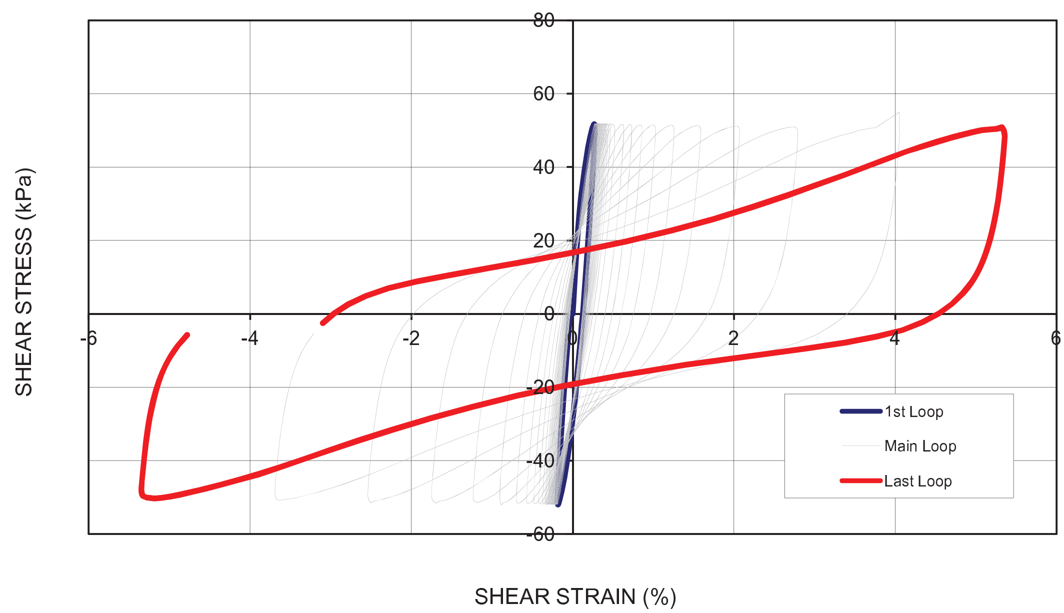
MEG TECHNICAL SERVICES
(A Division of MEG Consulting Limited)

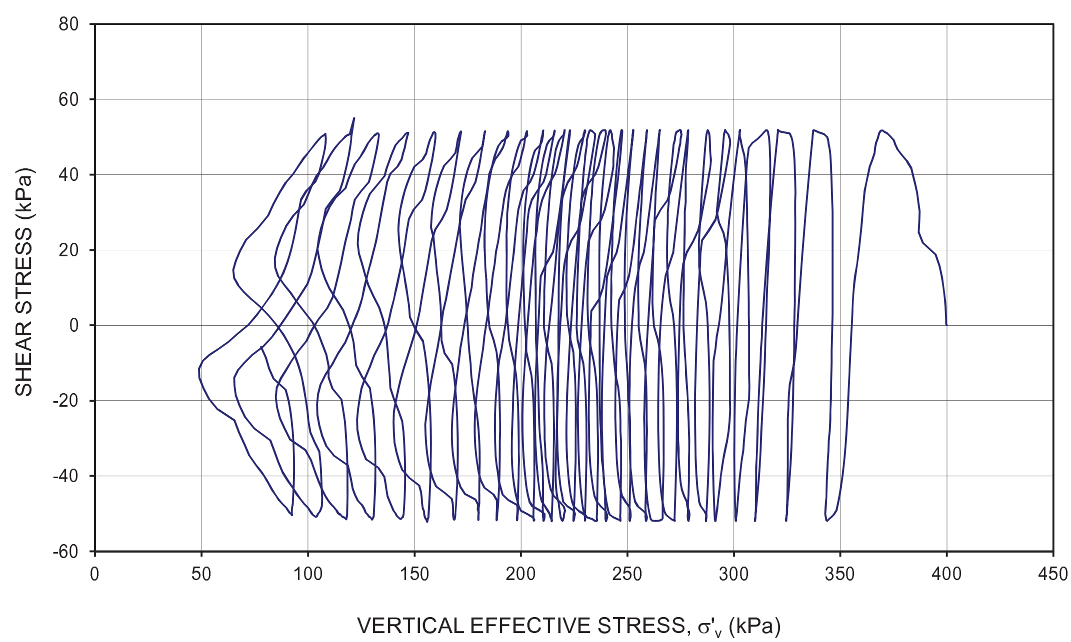
Marine + Earth MEG Geosciences

Project:	AMECFW - Myra Falls			Project No.:	15-MTS-029
Location:	BC	Borehole:	BH15-39	Depth:	18.76 m
Sample:	SH03	Station:	DSS1	Date:	March 10, 2016

STRESS CONTROLLED CYCLIC DIRECT SIMPLE SHEAR TEST

0.13 stress ratio (τ_{cyc}/ σ'_{vc}) @ 0.1 Hz for 30 cycles, σ'_{vc}=400kPa





Prepared By:	MF	Checked By:	PS	Approved By:	JPS
Date:	March 10, 2016	Date:	March 10, 2016	Date:	March 11, 2016

OQM Organizational Quality Management Program

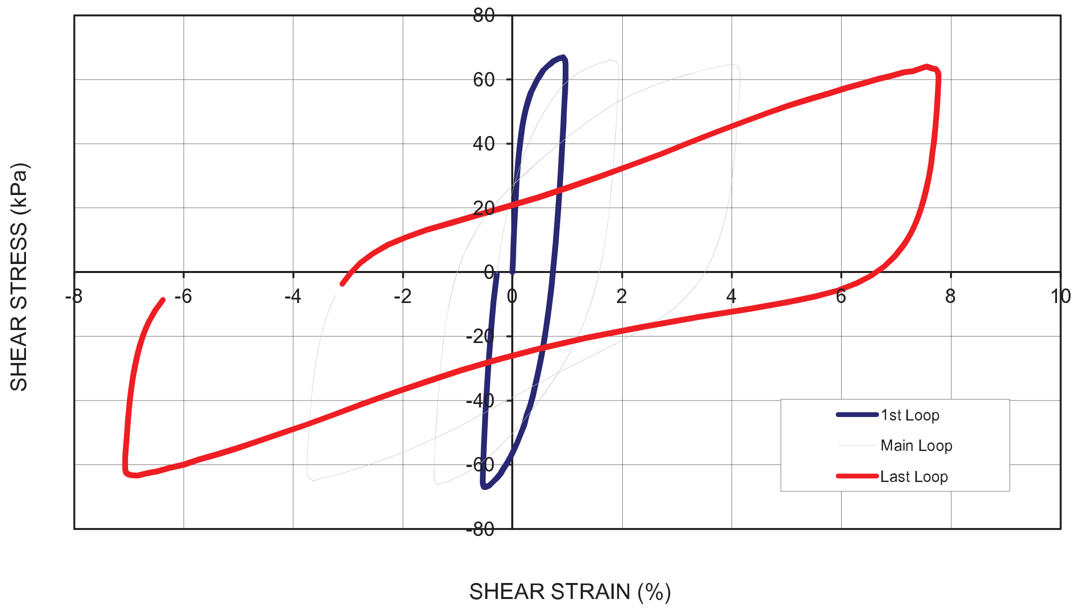
MEG TECHNICAL SERVICES
(A Division of MEG Consulting Limited)

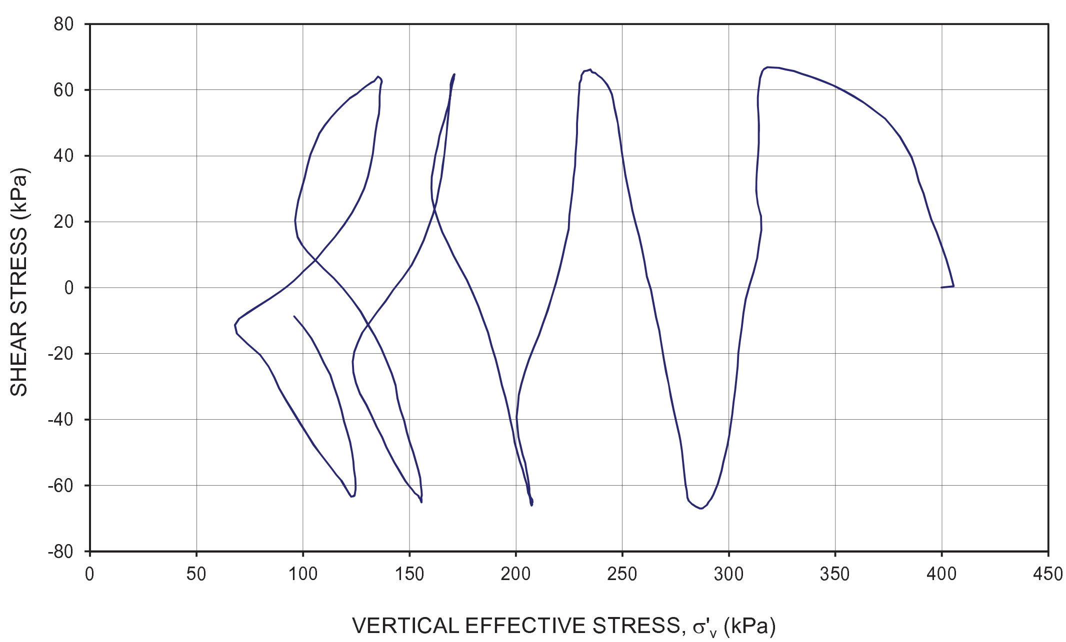
Marine + Earth
MEG
Geosciences

Project:	AMECFW - Myra Falls			Project No.:	15-MTS-029
Location:	BC	Borehole:	BH15-39	Depth:	18.76 m
Sample:	SH03	Station:	DSS1	Date:	March 10, 2016

STRESS CONTROLLED CYCLIC DIRECT SIMPLE SHEAR TEST

0.17 stress ratio (τ_{cyc}/ σ'_{vc}) @ 0.1 Hz for 4 cycles, σ'_{vc}=400kPa





Prepared By:	MF	Checked By:	PS	Approved By:	JPS
Date:	March 10, 2016	Date:	March 10, 2016	Date:	March 11, 2016

MEG Technical Services (MTS)

(A Division of MEG Consulting Limited)

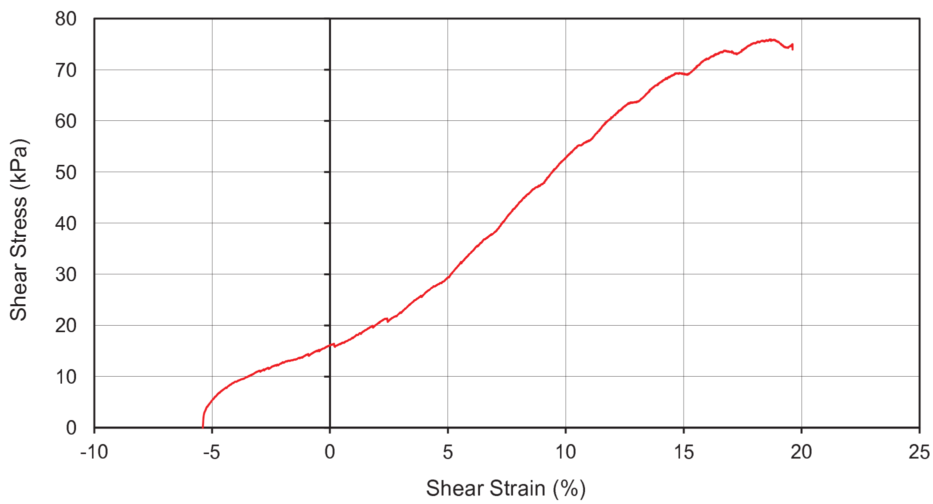
Form N° MTS214

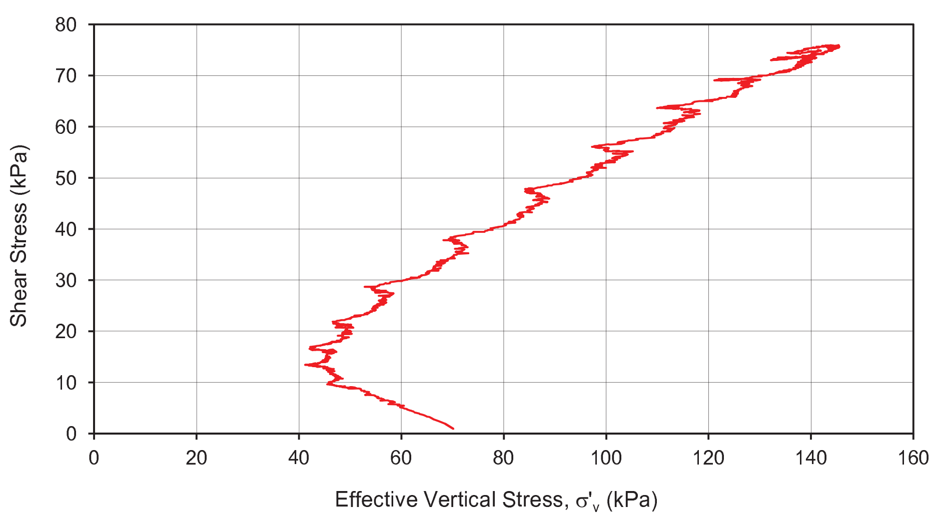
Marine + Earth MEG Geosciences

Project:	AMECFW - Myra Falls			Project No.:	15-MTS-029
Location:	BC	Borehole:	BH15-39	Depth:	18.86 m
Sample:	SH03	Station:	DSS1	Date:	March 9, 2016

Post-Cyclic Static Direct Simple Shear Test

POST-CYCLIC STATIC SHEAR TEST





Note: Test performed after stress-controlled DSS test at average cyclic stress ratio, CSR = 0.17 with 83% excess pore pressure

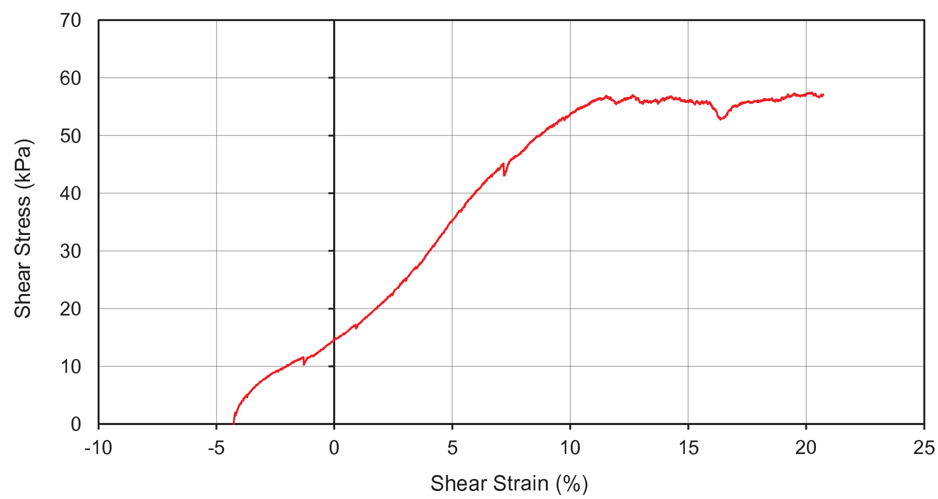
Prepared By:	MF	Checked By:	PS	Approved By:	JPS
Date:	March 9, 2016	Date:	March 9, 2016	Date:	March 11, 2016

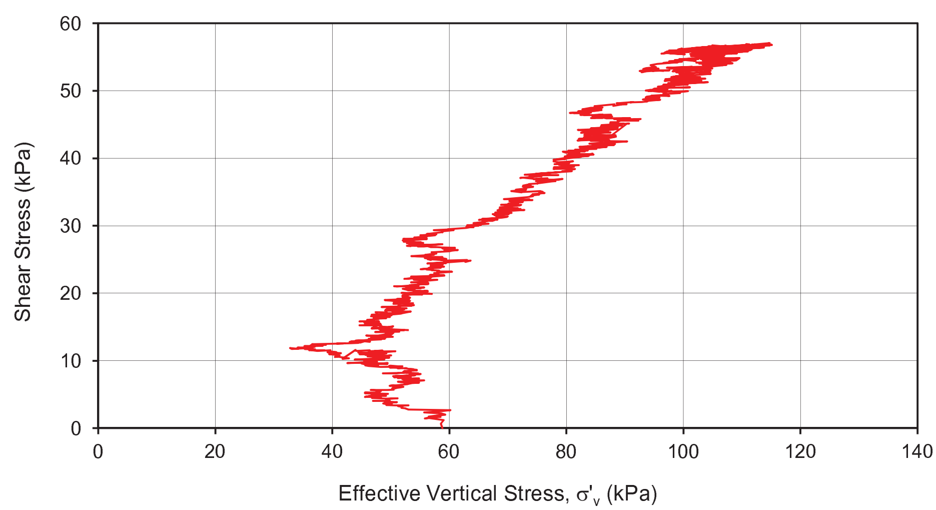
MEG Technical Services (MTS)
(A Division of MEG Consulting Limited)

Project:	AMECFW - Myra Falls			Project No.:	15-MTS-029
Location:	BC	Borehole:	BH15-39	Depth:	18.86 m
Sample:	SH03	Station:	DSS1	Date:	March 9, 2016

Post-Cyclic Static Direct Simple Shear Test

POST-CYCLIC STATIC SHEAR TEST





Note: Test performed after stress-controlled DSS test at average cyclic stress ratio, CSR = 0.13
with 88% excess pore pressure

Prepared By:	MF	Checked By:	PS	Approved By:	JPS
Date:	March 9, 2016	Date:	March 9, 2016	Date:	March 11, 2016

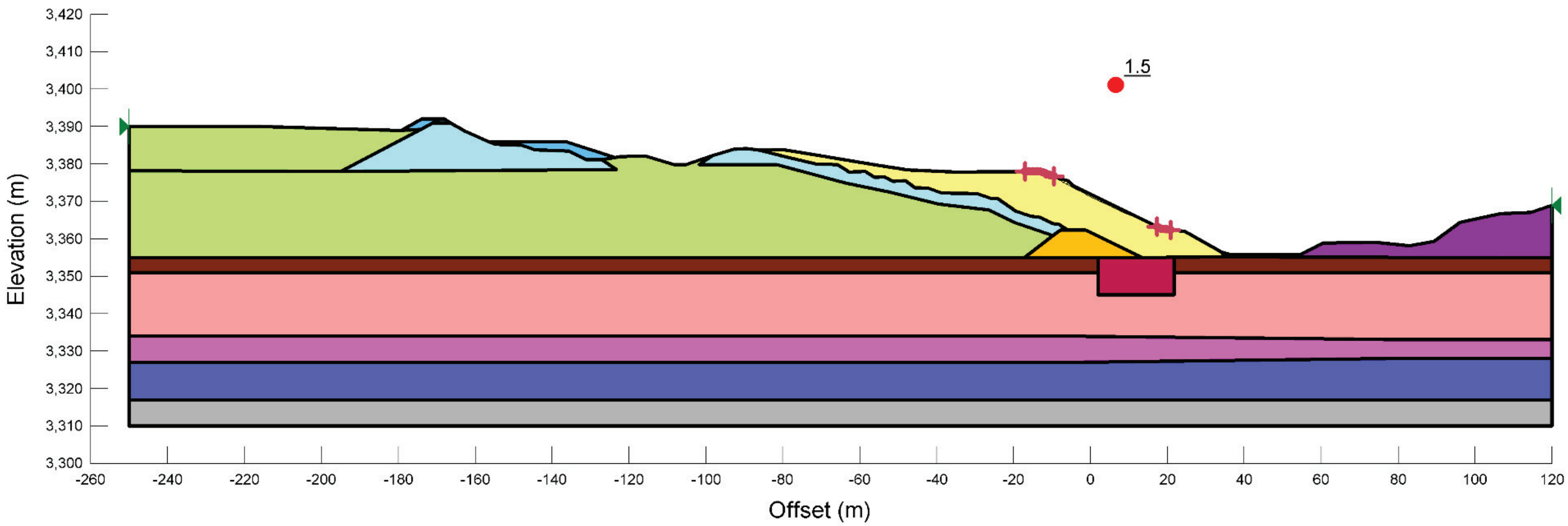
APPENDIX E

Limit Equilibrium Slope Stability Results

APA Berm Limit Equilibrium Slope Stability Analysis

Name: APA Plane 2 - Post-Seismic Case - Mode A

Date: 11/08/2016

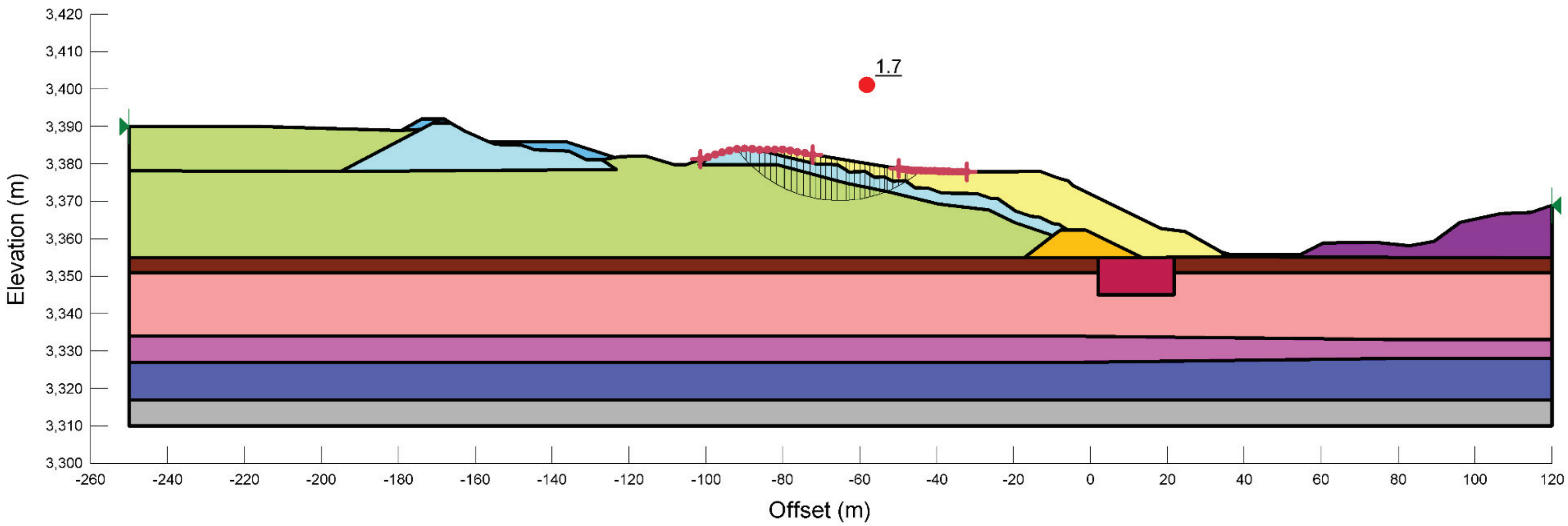


	Material Type	Unit Weight (kN/m^3)	Friction Angle (°)	Tau/Sigma
	Seismic Berm Fill	22	38	--
	Tailings - Post-Seismic	22.5	--	0.14
	Embankment Raises	22	34	--
	Embankment Raises - Addition	22	34	--
	Starter Embankment	22	36	--
	Mine Road Fill	24	30	--
	Densified Sand & Gravel	24	36	--
	Unit 2a Glaciofluvial	24	34	--
	Unit 2b Colluvium - Post-Seismic	24	--	0.28
	Unit 3 Transitional GF - Post-Seismic	22	--	0.35
	Unit 4 GLU - Post-Seismic	20	--	0.13
	Glacial Till	Impenetrable Bedrock		

APA Berm Limit Equilibrium Slope Stability Analysis

Name: APA Plane 2 - Post-Seismic Case - Mode B

Date: 11/08/2016

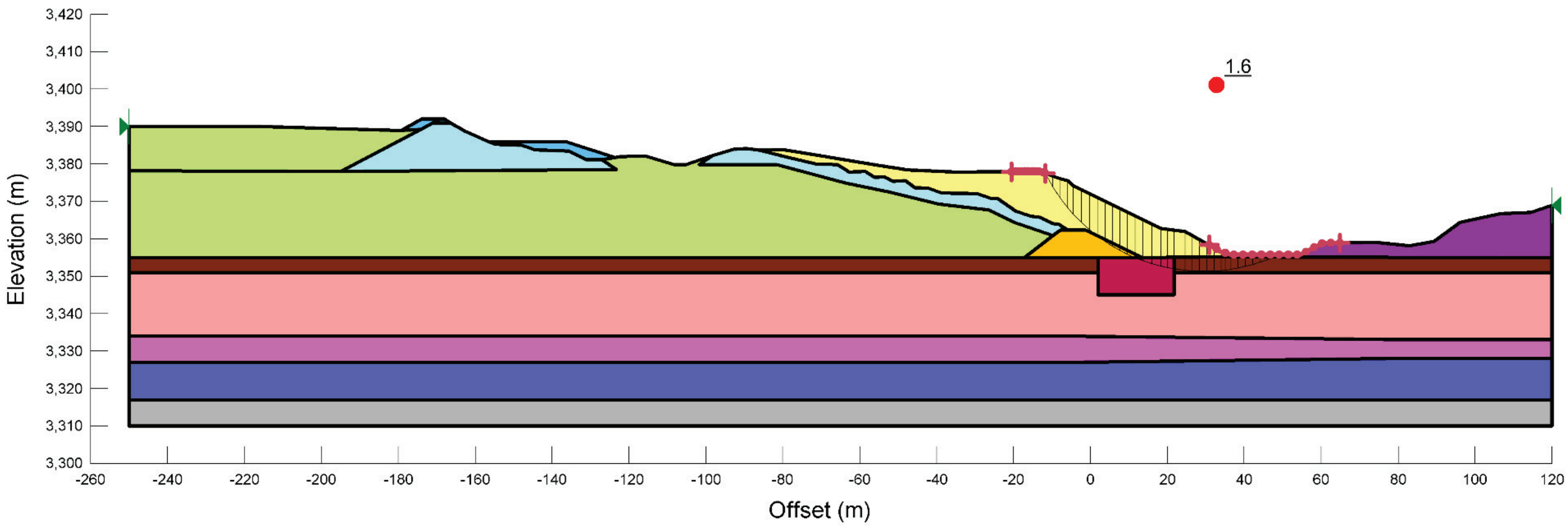


Material Type	Unit Weight (kN/m^3)	Friction Angle (°)	Tau/Sigma
Seismic Berm Fill	22	38	--
Tailings - Post-Seismic	22.5	--	0.14
Embankment Raises	22	34	--
Embankment Raises - Addition	22	34	--
Starter Embankment	22	36	--
Mine Road Fill	24	30	--
Densified Sand & Gravel	24	36	--
Unit 2a Glaciofluvial	24	34	--
Unit 2b Colluvium - Post-Seismic	24	--	0.28
Unit 3 Transitional GF - Post-Seismic	22	--	0.35
Unit 4 GLU - Post-Seismic	20	--	0.13
Glacial Till	Impenetrable Bedrock		

APA Berm Limit Equilibrium Slope Stability Analysis

Name: APA Plane 2 - Post-Seismic Case - Mode C

Date: 11/08/2016

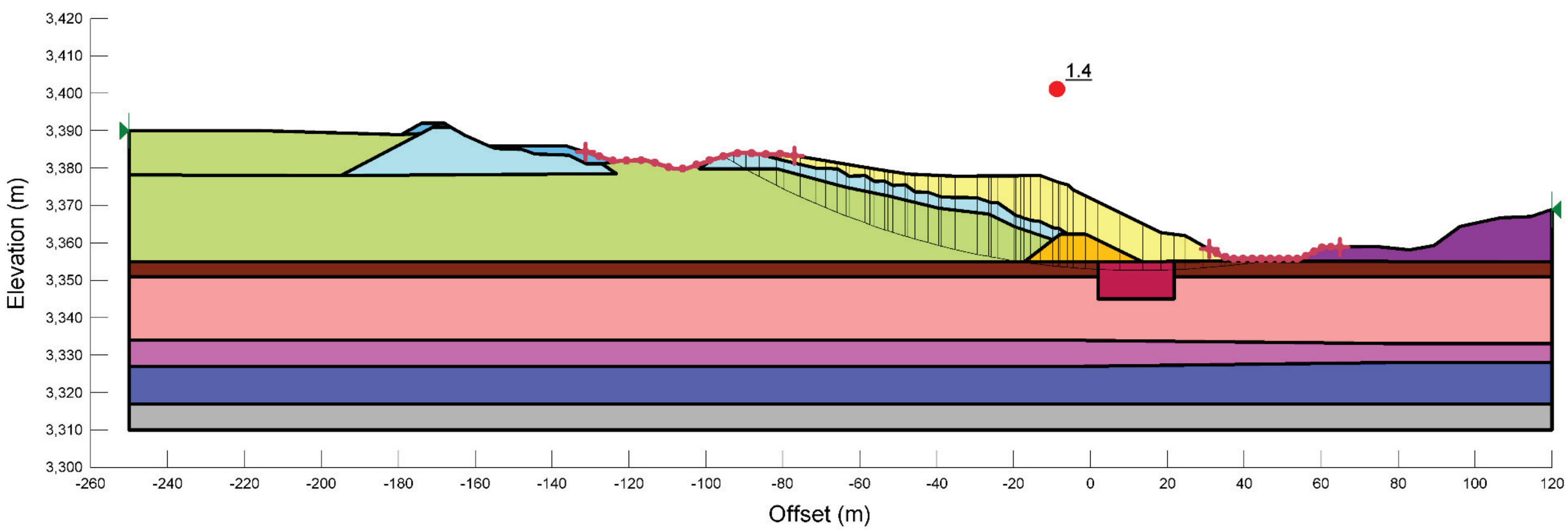


Material Type	Unit Weight (kN/m^3)	Friction Angle (°)	Tau/Sigma
Seismic Berm Fill	22	38	--
Tailings - Post-Seismic	22.5	--	0.14
Embankment Raises	22	34	--
Embankment Raises - Addition	22	34	--
Starter Embankment	22	36	--
Mine Road Fill	24	30	--
Densified Sand & Gravel	24	36	--
Unit 2a Glaciofluvial	24	34	--
Unit 2b Colluvium - Post-Seismic	24	--	0.28
Unit 3 Transitional GF - Post-Seismic	22	--	0.35
Unit 4 GLU - Post-Seismic	20	--	0.13
Glacial Till	Impenetrable Bedrock		

APA Berm Limit Equilibrium Slope Stability Analysis

Name: APA Plane 2 - Post-Seismic Case - Mode D

Date: 11/08/2016

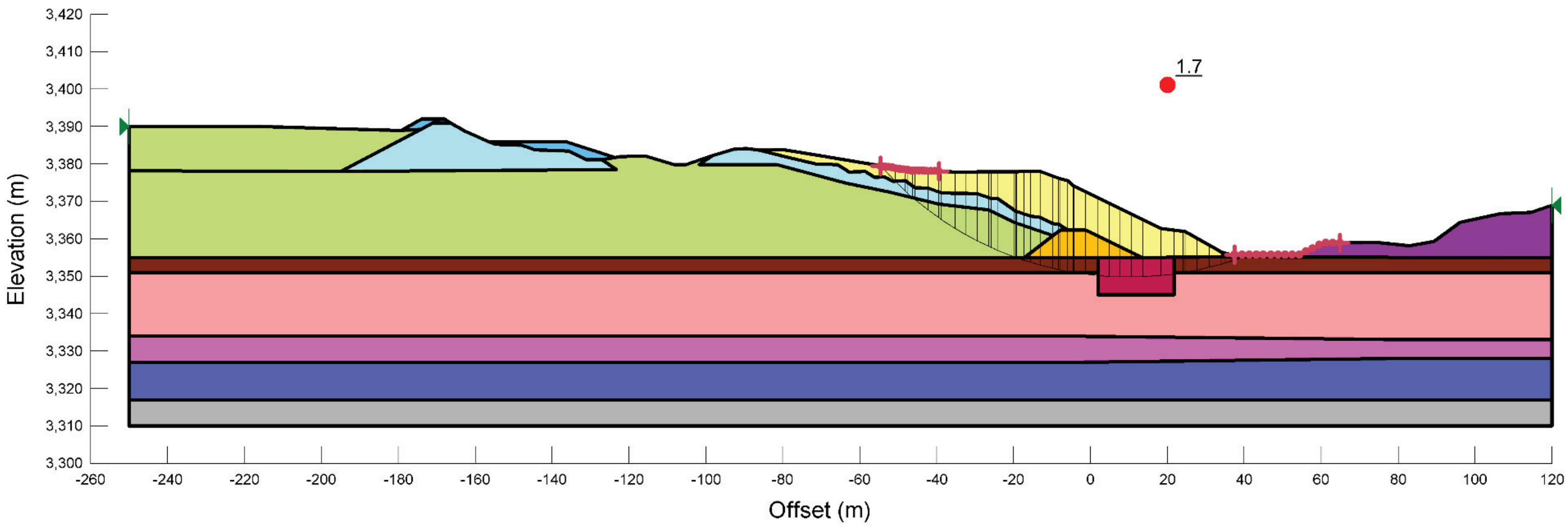


Material Type	Unit Weight (kN/m^3)	Friction Angle (°)	Tau/Sigma
Seismic Berm Fill	22	38	--
Tailings - Post-Seismic	22.5	--	0.14
Embankment Raises	22	34	--
Embankment Raises - Addition	22	34	--
Starter Embankment	22	36	--
Mine Road Fill	24	30	--
Densified Sand & Gravel	24	36	--
Unit 2a Glaciofluvial	24	34	--
Unit 2b Colluvium - Post-Seismic	24	--	0.28
Unit 3 Transitional GF - Post-Seismic	22	--	0.35
Unit 4 GLU - Post-Seismic	20	--	0.13
Glacial Till	Impenetrable Bedrock		

APA Berm Limit Equilibrium Slope Stability Analysis

Name: APA Plane 2 - Post-Seismic Case - Mode E

Date: 11/08/2016

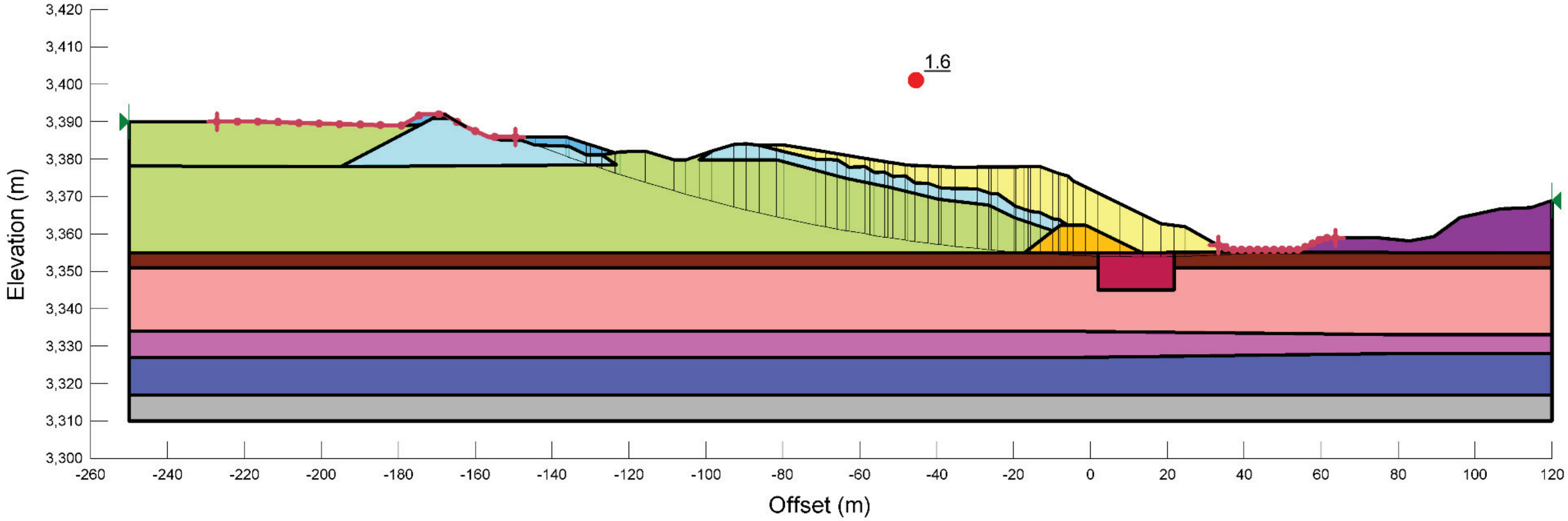


Material Type	Unit Weight (kN/m^3)	Friction Angle (°)	Tau/Sigma
Seismic Berm Fill	22	38	--
Tailings - Post-Seismic	22.5	--	0.14
Embankment Raises	22	34	--
Embankment Raises - Addition	22	34	--
Starter Embankment	22	36	--
Mine Road Fill	24	30	--
Densified Sand & Gravel	24	36	--
Unit 2a Glaciofluvial	24	34	--
Unit 2b Colluvium - Post-Seismic	24	--	0.28
Unit 3 Transitional GF - Post-Seismic	22	--	0.35
Unit 4 GLU - Post-Seismic	20	--	0.13
Glacial Till	Impenetrable Bedrock		

APA Berm Limit Equilibrium Slope Stability Analysis

Name: APA Plane 2 - Post-Seismic Case - Mode F

Date: 11/08/2016

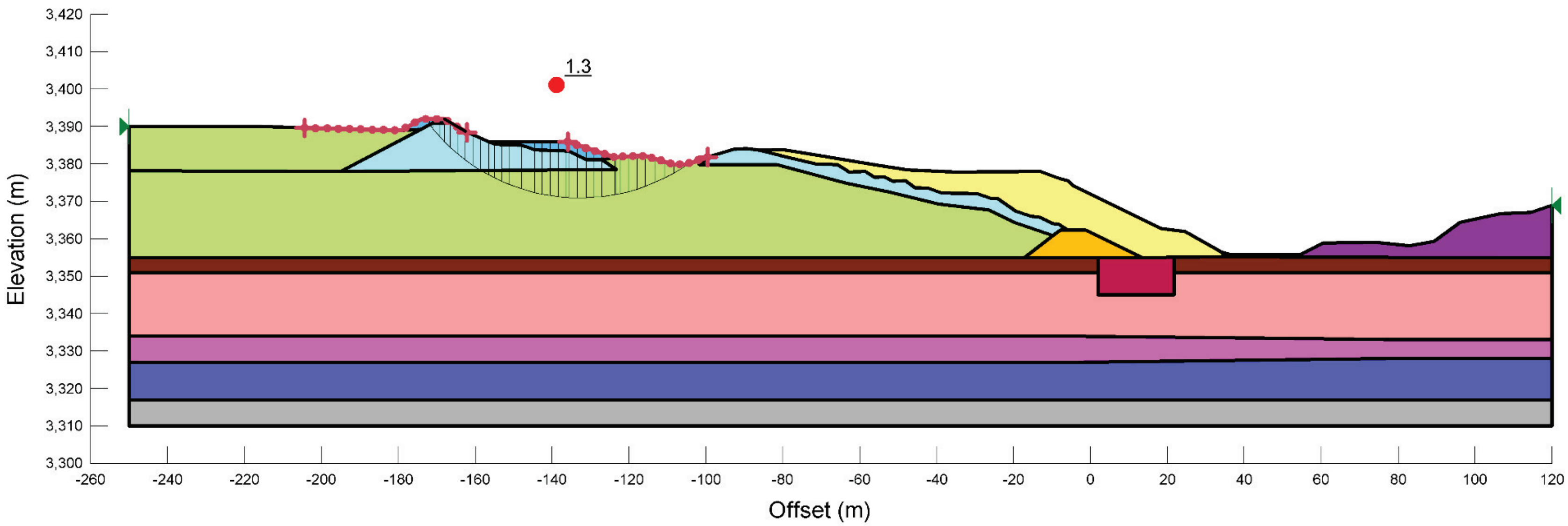


Material Type	Unit Weight (kN/m^3)	Friction Angle (°)	Tau/Sigma
Seismic Berm Fill	22	38	--
Tailings - Post-Seismic	22.5	--	0.14
Embankment Raises	22	34	--
Embankment Raises - Addition	22	34	--
Starter Embankment	22	36	--
Mine Road Fill	24	30	--
Densified Sand & Gravel	24	36	--
Unit 2a Glaciofluvial	24	34	--
Unit 2b Colluvium - Post-Seismic	24	--	0.28
Unit 3 Transitional GF - Post-Seismic	22	--	0.35
Unit 4 GLU - Post-Seismic	20	--	0.13
Glacial Till	Impenetrable Bedrock		

APA Berm Limit Equilibrium Slope Stability Analysis

Name: APA Plane 2 - Post-Seismic Case - Mode G

Date: 11/08/2016

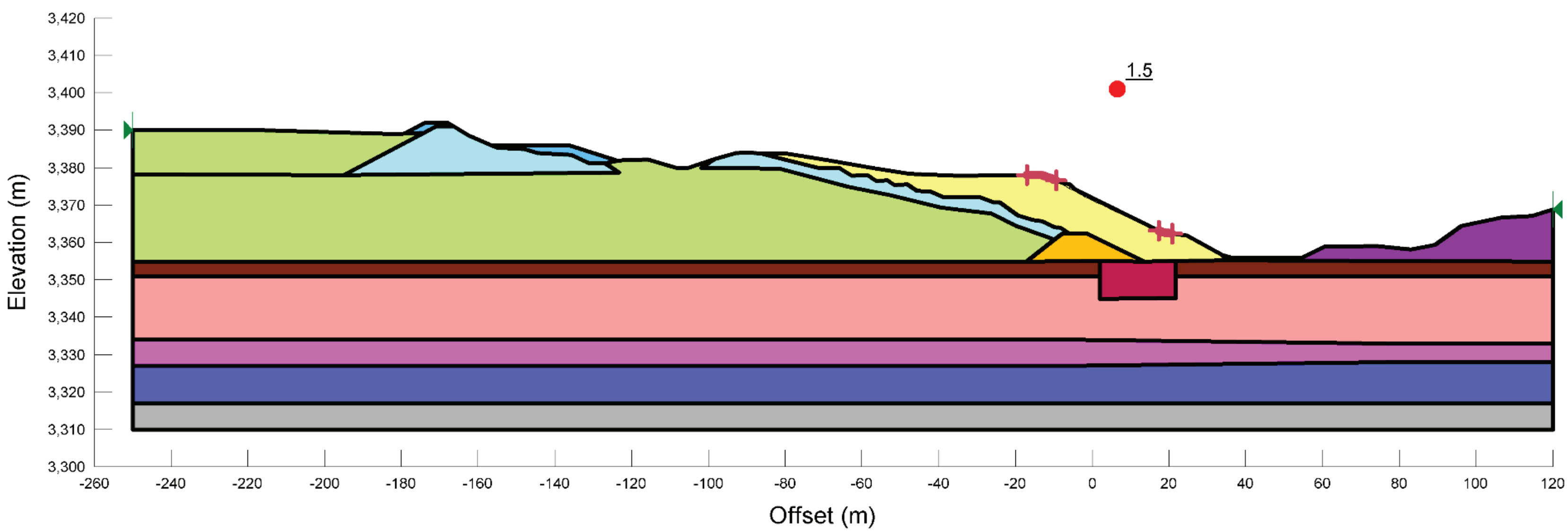


Material Type	Unit Weight (kN/m^3)	Friction Angle (°)	Tau/Sigma
Seismic Berm Fill	22	38	--
Tailings - Post-Seismic	22.5	--	0.14
Embankment Raises	22	34	--
Embankment Raises - Addition	22	34	--
Starter Embankment	22	36	--
Mine Road Fill	24	30	--
Densified Sand & Gravel	24	36	--
Unit 2a Glaciofluvial	24	34	--
Unit 2b Colluvium - Post-Seismic	24	--	0.28
Unit 3 Transitional GF - Post-Seismic	22	--	0.35
Unit 4 GLU - Post-Seismic	20	--	0.13
Glacial Till	Impenetrable Bedrock		

APA Berm Limit Equilibrium Slope Stability Analysis

Name: APA Plane 2 - Static Case - Mode A

Date: 12/08/2016

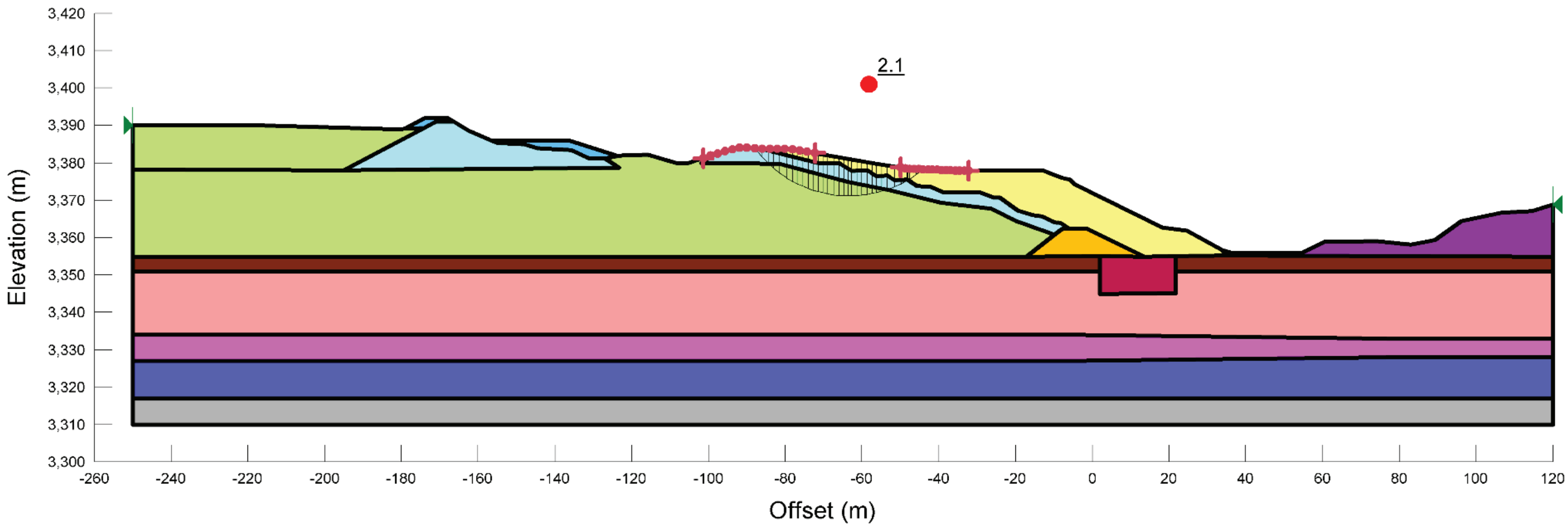


Material Type	Unit Weight (kN/m^3)	Friction Angle (°)	Tau/Sigma
Seismic Berm Fill	22	38	--
Tailings - Static	22.5	--	0.2
Embankment Raises	22	34	--
Embankment Raises - Addition	22	34	--
Starter Embankment	22	36	--
Mine Road Fill	24	30	--
Densified Sand & Gravel	24	36	--
Unit 2a Glaciofluvial	24	34	--
Unit 2b Colluvium - Static	24	34	--
Unit 3 Transitional GF - Static	22	32	--
Unit 4 GLU - Static	20	--	0.22
Glacial Till	Impenetrable Bedrock		

APA Berm Limit Equilibrium Slope Stability Analysis

Name: APA Plane 2 - Static Case - Mode B

Date: 12/08/2016

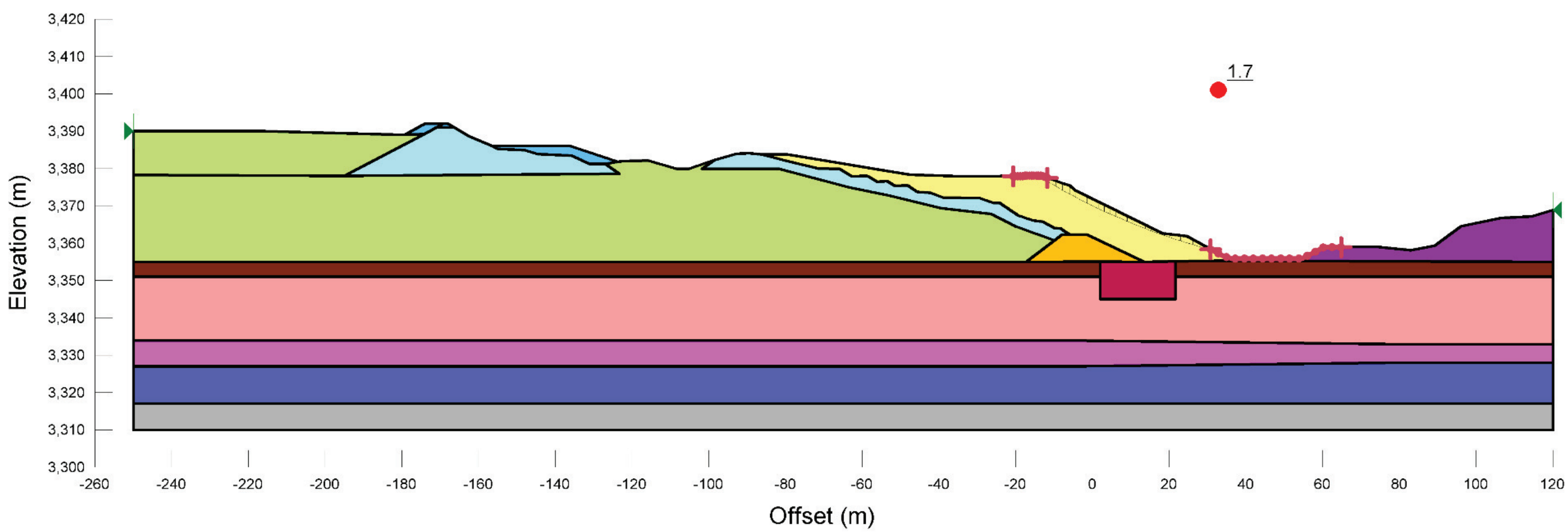


	Material Type	Unit Weight (kN/m^3)	Friction Angle (°)	Tau/Sigma
	Seismic Berm Fill	22	38	--
	Tailings - Static	22.5	--	0.2
	Embankment Raises	22	34	--
	Embankment Raises - Addition	22	34	--
	Starter Embankment	22	36	--
	Mine Road Fill	24	30	--
	Densified Sand & Gravel	24	36	--
	Unit 2a Glaciofluvial	24	34	--
	Unit 2b Colluvium - Static	24	34	--
	Unit 3 Transitional GF - Static	22	32	--
	Unit 4 GLU - Static	20	--	0.22
	Glacial Till	Impenetrable Bedrock		

APA Berm Limit Equilibrium Slope Stability Analysis

Name: APA Plane 2 - Static Case - Mode C

Date: 12/08/2016

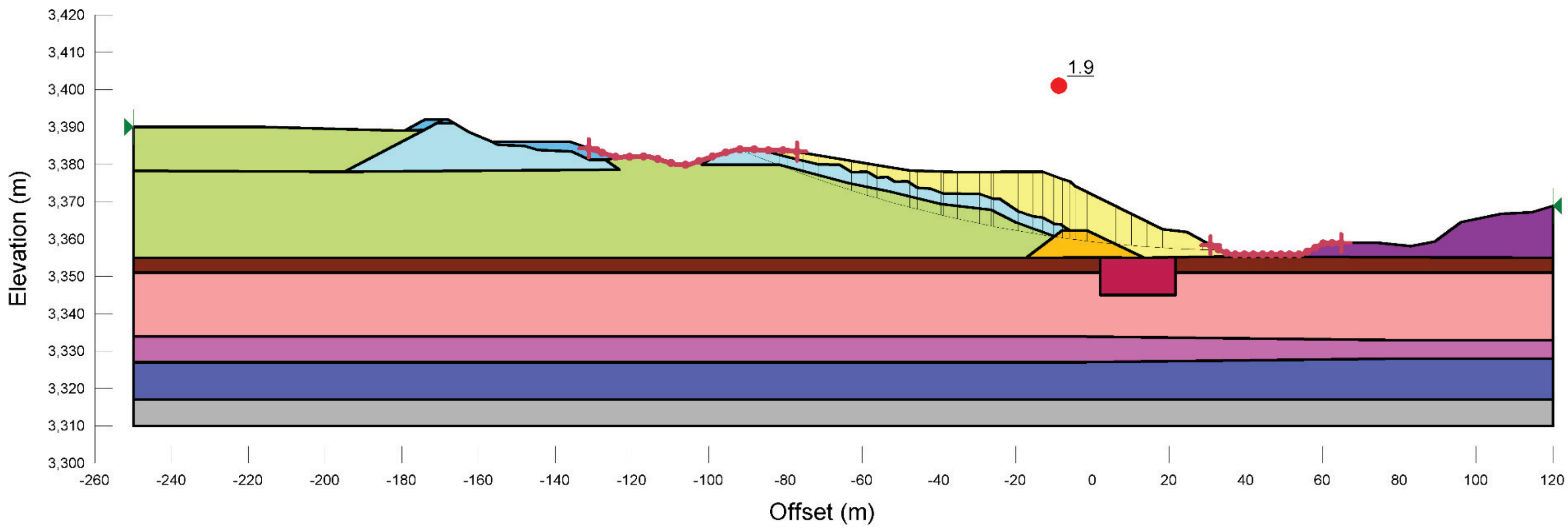


	Material Type	Unit Weight (kN/m^3)	Friction Angle (°)	Tau/Sigma
	Seismic Berm Fill	22	38	--
	Tailings - Static	22.5	--	0.2
	Embankment Raises	22	34	--
	Embankment Raises - Addition	22	34	--
	Starter Embankment	22	36	--
	Mine Road Fill	24	30	--
	Densified Sand & Gravel	24	36	--
	Unit 2a Glaciofluvial	24	34	--
	Unit 2b Colluvium - Static	24	34	--
	Unit 3 Transitional GF - Static	22	32	--
	Unit 4 GLU - Static	20	--	0.22
	Glacial Till	Impenetrable Bedrock		

APA Berm Limit Equilibrium Slope Stability Analysis

Name: APA Plane 2 - Static Case - Mode D

Date: 12/08/2016

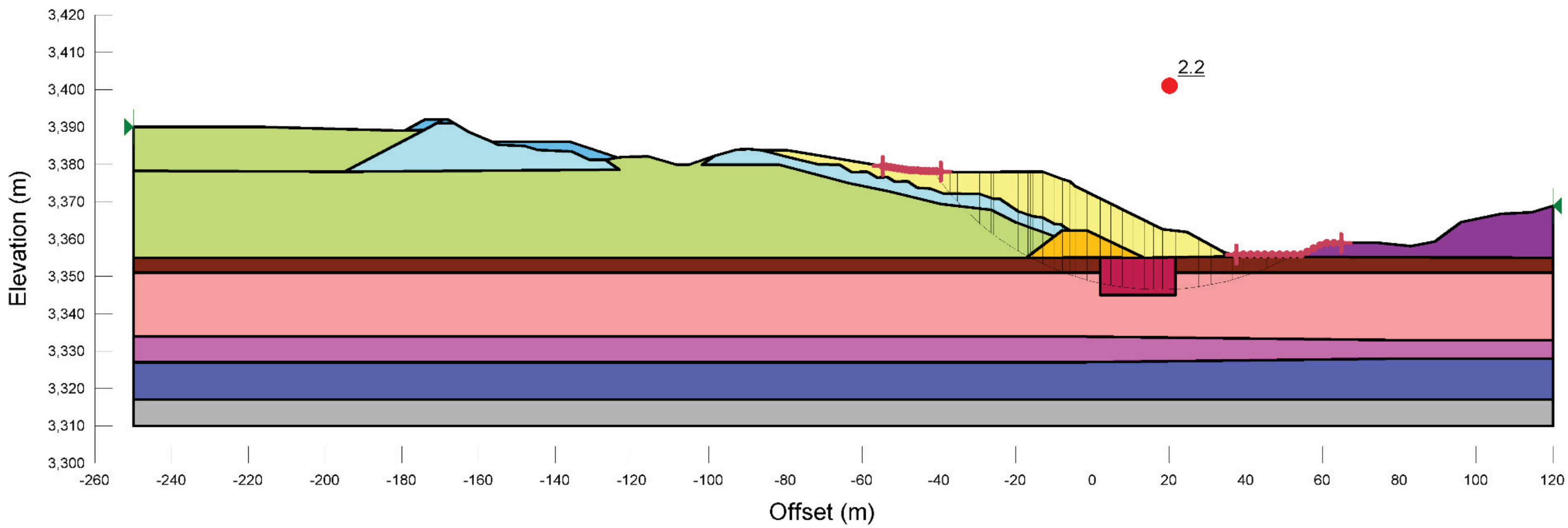


Material Type	Unit Weight (kN/m^3)	Friction Angle (°)	Tau/Sigma
Seismic Berm Fill	22	38	--
Tailings - Static	22.5	--	0.2
Embankment Raises	22	34	--
Embankment Raises - Addition	22	34	--
Starter Embankment	22	36	--
Mine Road Fill	24	30	--
Densified Sand & Gravel	24	36	--
Unit 2a Glaciofluvial	24	34	--
Unit 2b Colluvium - Static	24	34	--
Unit 3 Transitional GF - Static	22	32	--
Unit 4 GLU - Static	20	--	0.22
Glacial Till	Impenetrable Bedrock		

APA Berm Limit Equilibrium Slope Stability Analysis

Name: APA Plane 2 - Static Case - Mode E

Date: 12/08/2016

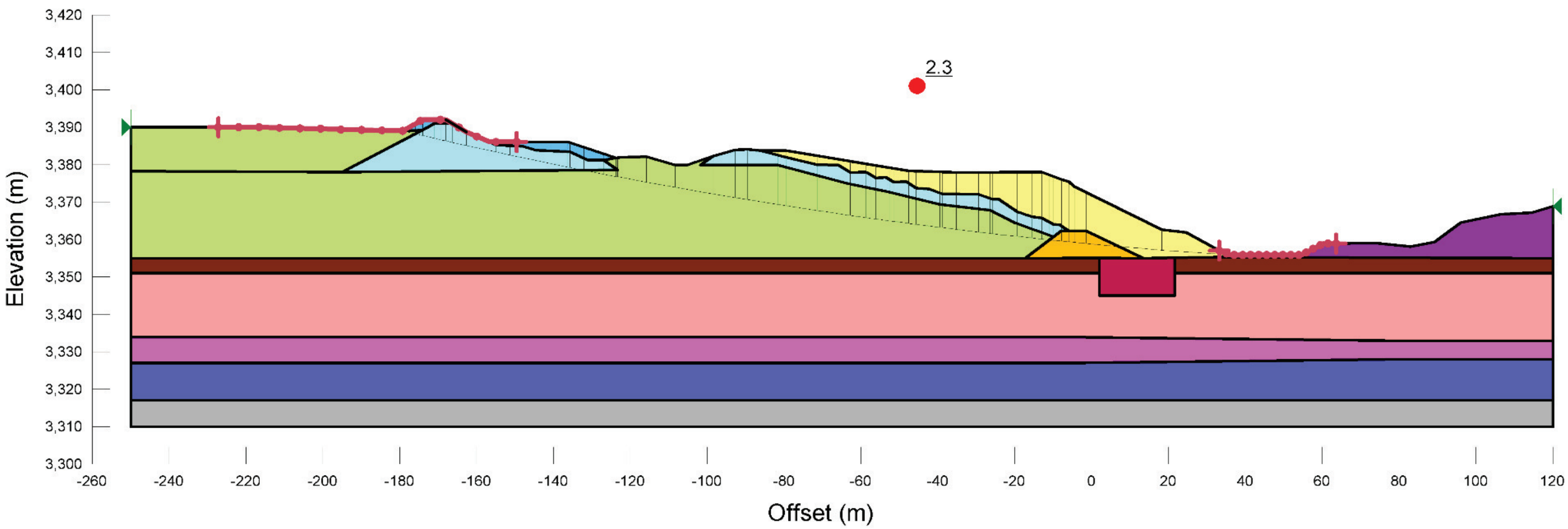


	Material Type	Unit Weight (kN/m^3)	Friction Angle (°)	Tau/Sigma
	Seismic Berm Fill	22	38	--
	Tailings - Static	22.5	--	0.2
	Embankment Raises	22	34	--
	Embankment Raises - Addition	22	34	--
	Starter Embankment	22	36	--
	Mine Road Fill	24	30	--
	Densified Sand & Gravel	24	36	--
	Unit 2a Glaciofluvial	24	34	--
	Unit 2b Colluvium - Static	24	34	--
	Unit 3 Transitional GF - Static	22	32	--
	Unit 4 GLU - Static	20	--	0.22
	Glacial Till	Impenetrable Bedrock		

APA Berm Limit Equilibrium Slope Stability Analysis

Name: APA Plane 2 - Static Case - Mode F

Date: 12/08/2016

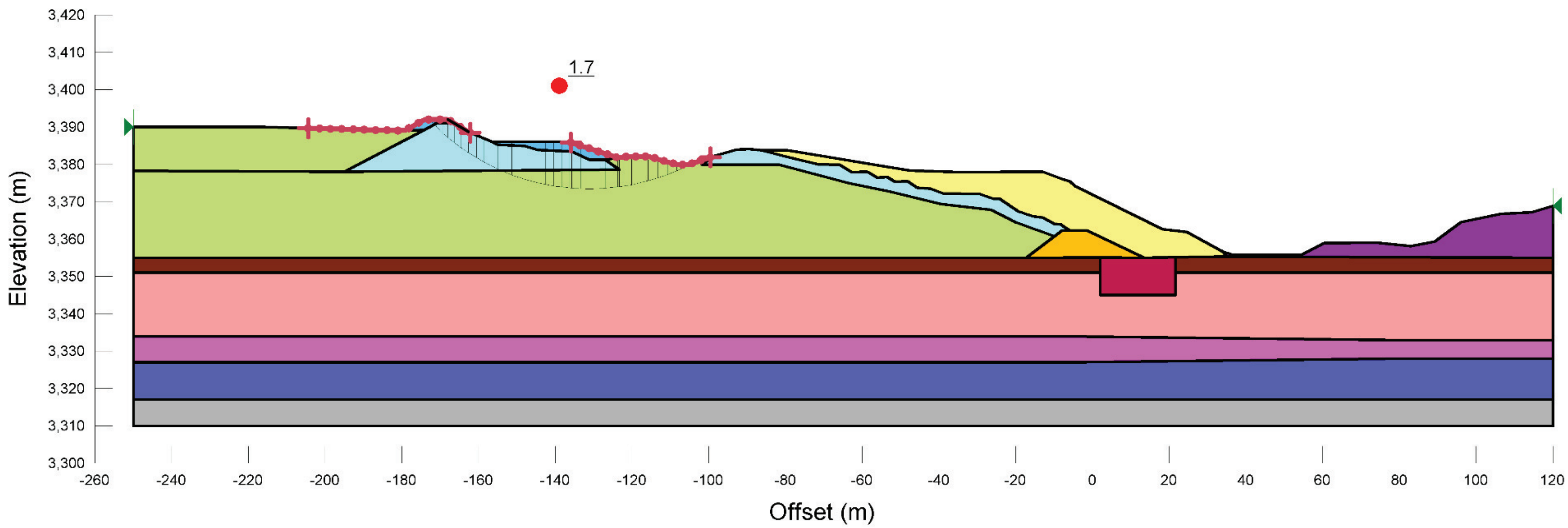


	Material Type	Unit Weight (kN/m³)	Friction Angle (°)	Tau/Sigma
	Seismic Berm Fill	22	38	--
	Tailings - Static	22.5	--	0.2
	Embankment Raises	22	34	--
	Embankment Raises - Addition	22	34	--
	Starter Embankment	22	36	--
	Mine Road Fill	24	30	--
	Densified Sand & Gravel	24	36	--
	Unit 2a Glaciofluvial	24	34	--
	Unit 2b Colluvium - Static	24	34	--
	Unit 3 Transitional GF - Static	22	32	--
	Unit 4 GLU - Static	20	--	0.22
	Glacial Till	Impenetrable Bedrock		

APA Berm Limit Equilibrium Slope Stability Analysis

Name: APA Plane 2 - Static Case - Mode G

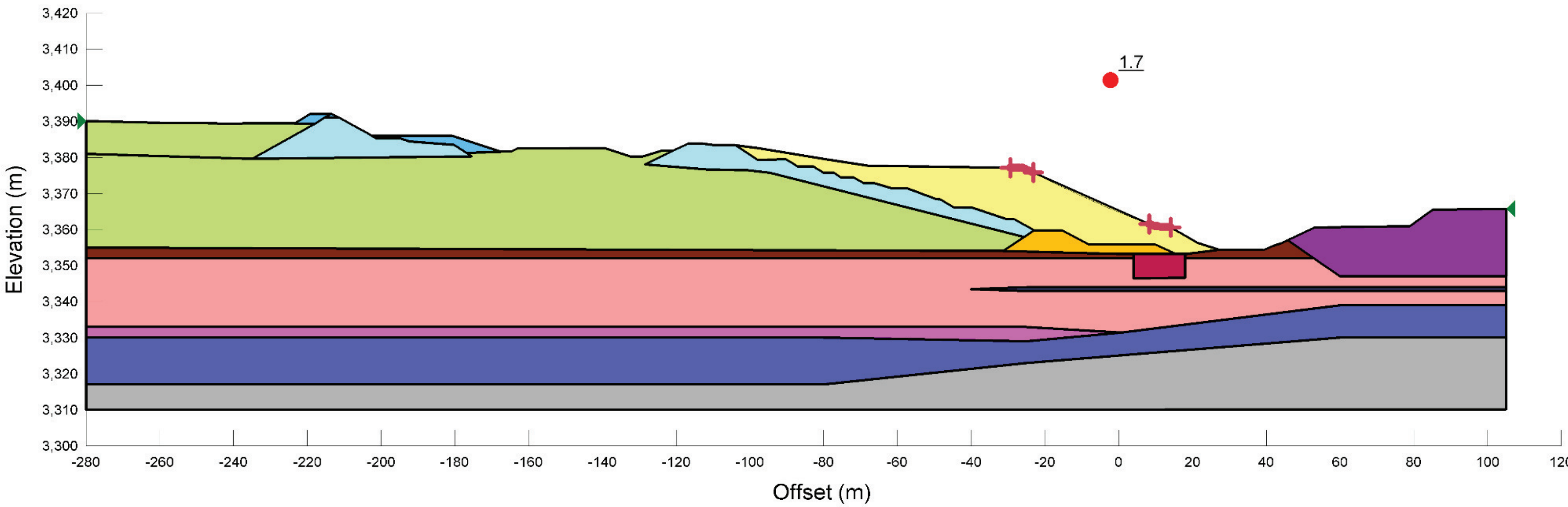
Date: 12/08/2016



	Material Type	Unit Weight (kN/m^3)	Friction Angle (°)	Tau/Sigma
	Seismic Berm Fill	22	38	--
	Tailings - Static	22.5	--	0.2
	Embankment Raises	22	34	--
	Embankment Raises - Addition	22	34	--
	Starter Embankment	22	36	--
	Mine Road Fill	24	30	--
	Densified Sand & Gravel	24	36	--
	Unit 2a Glaciofluvial	24	34	--
	Unit 2b Colluvium - Static	24	34	--
	Unit 3 Transitional GF - Static	22	32	--
	Unit 4 GLU - Static	20	--	0.22
	Glacial Till	Impenetrable Bedrock		

APA Berm Limit Equilibrium Slope Stability Analysis

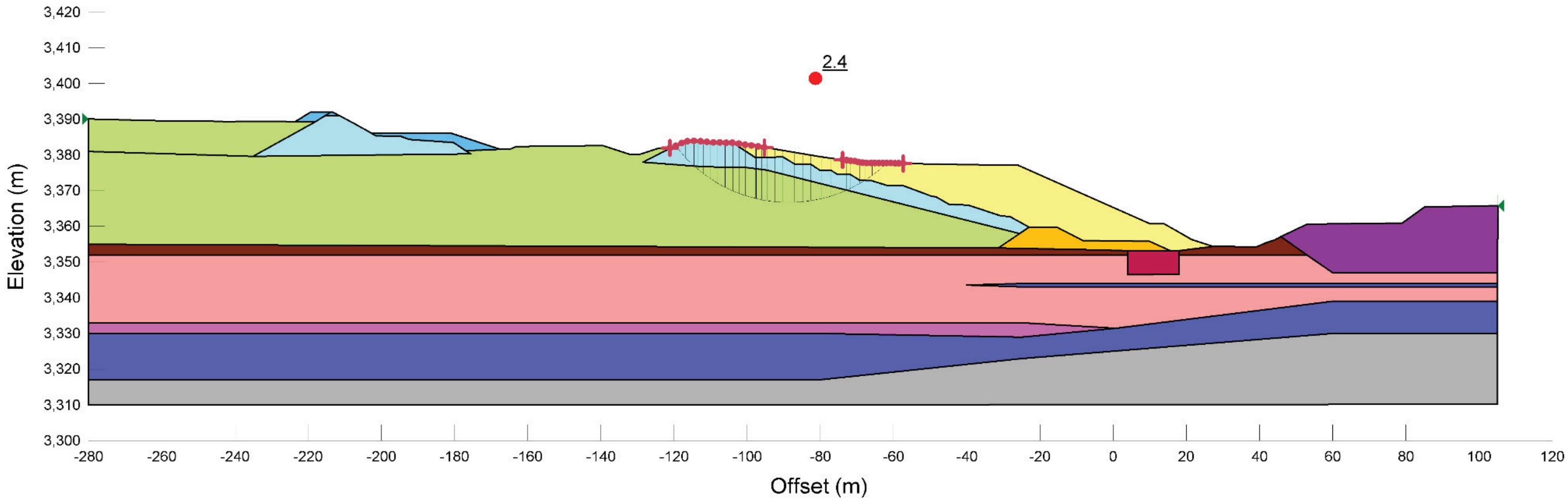
Name: APA Plane 3 - Post-Seismic - Mode A
Date: 11/08/2016



Material Type	Unit Weight (kN/m^3)	Friction Angle (°)	Tau/Sigma
Seismic Berm Fill	22	38	--
Tailings - Post-Seismic	22.5	--	0.14
Embankment Raises	22	34	--
Embankment Raises - Addition	22	34	--
Starter Embankment	22	36	--
Mine Road Fill	24	30	--
Densified Sand & Gravel	24	36	--
Unit 2a Glaciofluvial	24	34	--
Unit 2b Colluvium - Post-Seismic	24	--	0.28
Unit 3 Transitional GF - Post-Seismic	22	--	0.35
Unit 4 GLU - Post-Seismic	20	--	0.13
Glacial Till	Impenetrable Bedrock		

APA Berm Limit Equilibrium Slope Stability Analysis

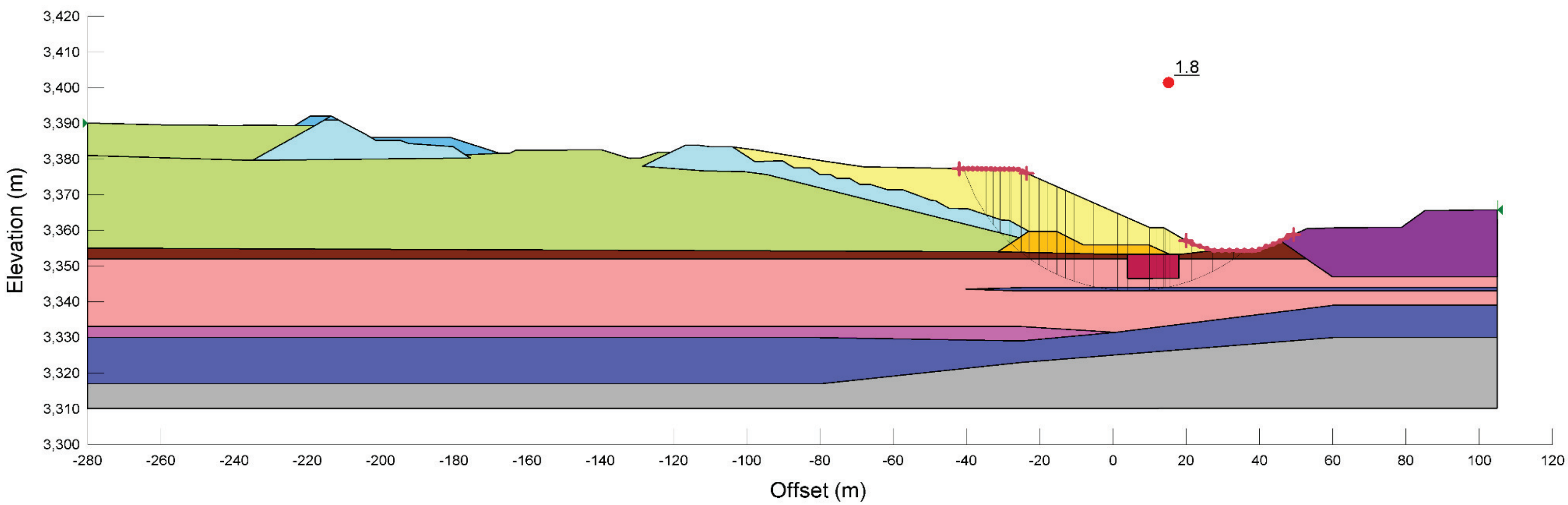
Name: APA Plane 3 - Post-Seismic - Mode B
Date: 11/08/2016



Material Type	Unit Weight (kN/m^3)	Friction Angle (°)	Tau/Sigma
Seismic Berm Fill	22	38	--
Tailings - Post-Seismic	22.5	--	0.14
Embankment Raises	22	34	--
Embankment Raises - Addition	22	34	--
Starter Embankment	22	36	--
Mine Road Fill	24	30	--
Densified Sand & Gravel	24	36	--
Unit 2a Glaciofluvial	24	34	--
Unit 2b Colluvium - Post-Seismic	24	--	0.28
Unit 3 Transitional GF - Post-Seismic	22	--	0.35
Unit 4 GLU - Post-Seismic	20	--	0.13
Glacial Till	Impenetrable Bedrock		

APA Berm Limit Equilibrium Slope Stability Analysis

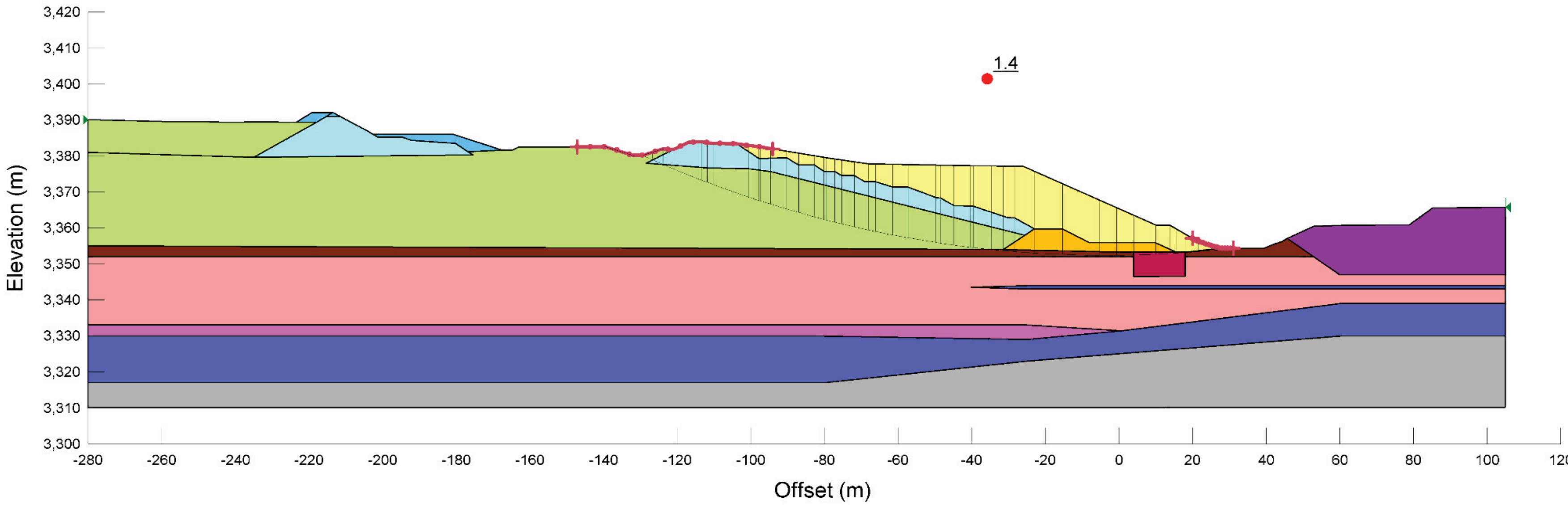
Name: APA Plane 3 - Post-Seismic - Mode C
Date: 11/08/2016



	Material Type	Unit Weight (kN/m^3)	Friction Angle (°)	Tau/Sigma
	Seismic Berm Fill	22	38	--
	Tailings - Post-Seismic	22.5	--	0.14
	Embankment Raises	22	34	--
	Embankment Raises - Addition	22	34	--
	Starter Embankment	22	36	--
	Mine Road Fill	24	30	--
	Densified Sand & Gravel	24	36	--
	Unit 2a Glaciofluvial	24	34	--
	Unit 2b Colluvium - Post-Seismic	24	--	0.28
	Unit 3 Transitional GF - Post-Seismic	22	--	0.35
	Unit 4 GLU - Post-Seismic	20	--	0.13
	Glacial Till	Impenetrable Bedrock		

APA Berm Limit Equilibrium Slope Stability Analysis

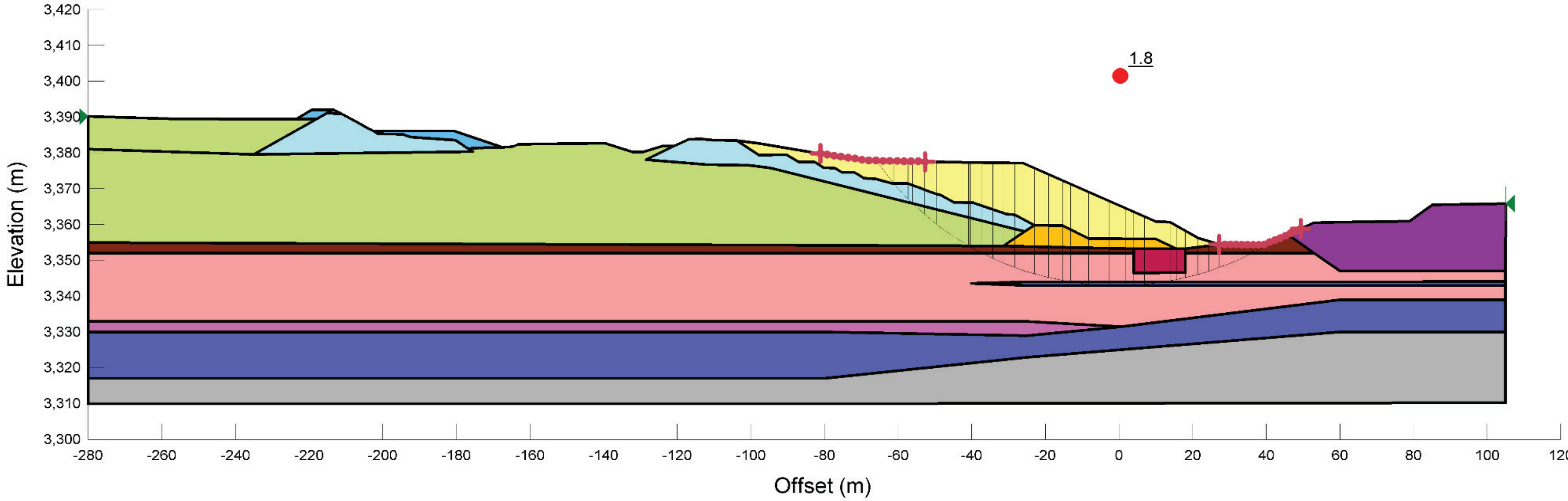
Name: APA Plane 3 - Post-Seismic - Mode D
Date: 11/08/2016



Material Type	Unit Weight (kN/m^3)	Friction Angle (°)	Tau/Sigma
Seismic Berm Fill	22	38	--
Tailings - Post-Seismic	22.5	--	0.14
Embankment Raises	22	34	--
Embankment Raises - Addition	22	34	--
Starter Embankment	22	36	--
Mine Road Fill	24	30	--
Densified Sand & Gravel	24	36	--
Unit 2a Glaciofluvial	24	34	--
Unit 2b Colluvium - Post-Seismic	24	--	0.28
Unit 3 Transitional GF - Post-Seismic	22	--	0.35
Unit 4 GLU - Post-Seismic	20	--	0.13
Glacial Till	Impenetrable Bedrock		

APA Berm Limit Equilibrium Slope Stability Analysis

Name: APA Plane 3 - Post-Seismic - Mode E
Date: 11/08/2016

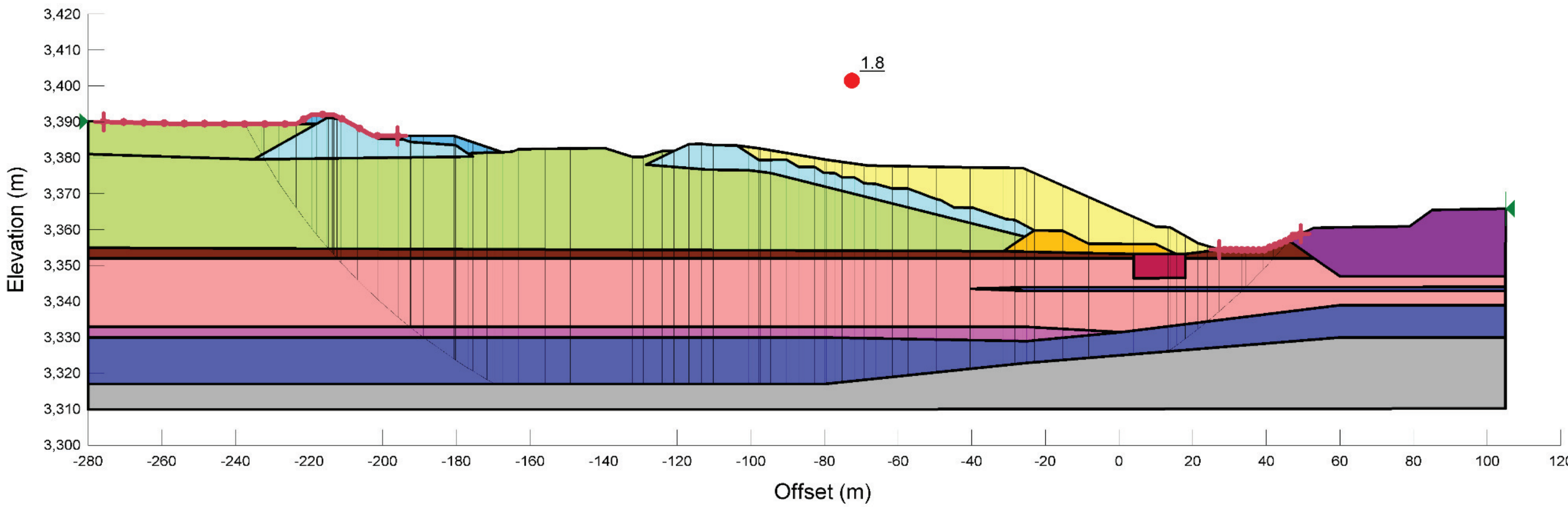


Material Type	Unit Weight (kN/m^3)	Friction Angle (°)	Tau/Sigma
Seismic Berm Fill	22	38	--
Tailings - Post-Seismic	22.5	--	0.14
Embankment Raises	22	34	--
Embankment Raises - Addition	22	34	--
Starter Embankment	22	36	--
Mine Road Fill	24	30	--
Densified Sand & Gravel	24	36	--
Unit 2a Glaciofluvial	24	34	--
Unit 2b Colluvium - Post-Seismic	24	--	0.28
Unit 3 Transitional GF - Post-Seismic	22	--	0.35
Unit 4 GLU - Post-Seismic	20	--	0.13
Glacial Till	Impenetrable Bedrock		

APA Berm Limit Equilibrium Slope Stability Analysis

Name: APA Plane 3 - Post-Seismic - Mode F

Date: 11/08/2016

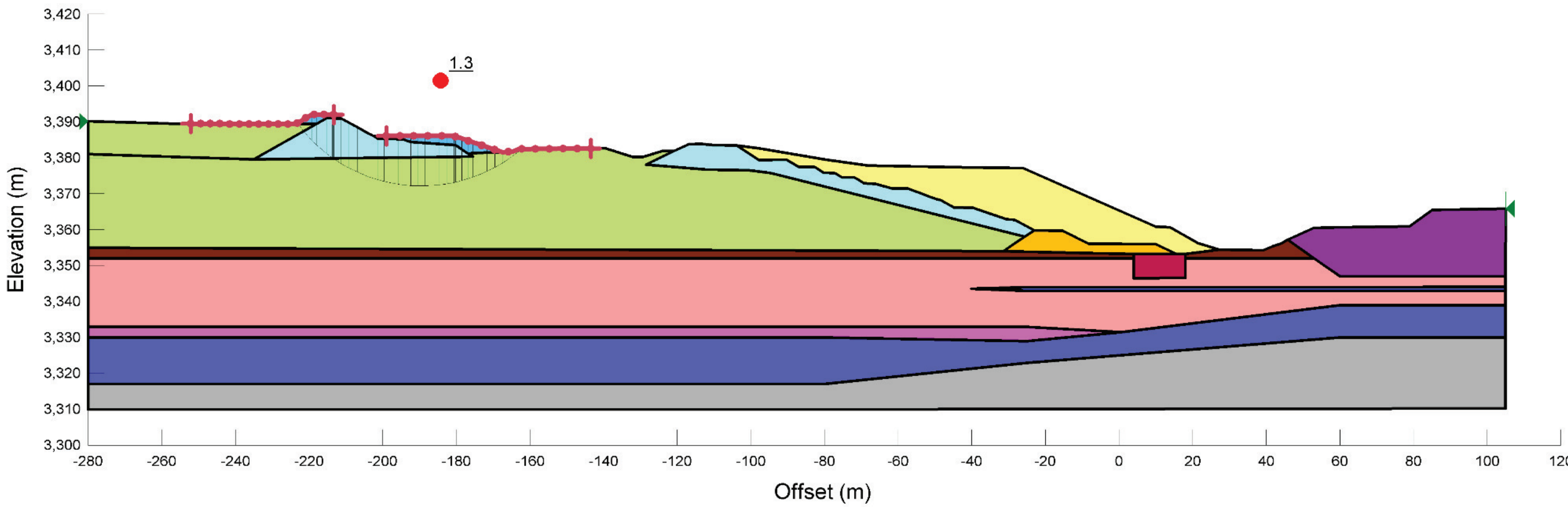


	Material Type	Unit Weight (kN/m^3)	Friction Angle (°)	Tau/Sigma
	Seismic Berm Fill	22	38	--
	Tailings - Post-Seismic	22.5	--	0.14
	Embankment Raises	22	34	--
	Embankment Raises - Addition	22	34	--
	Starter Embankment	22	36	--
	Mine Road Fill	24	30	--
	Densified Sand & Gravel	24	36	--
	Unit 2a Glaciofluvial	24	34	--
	Unit 2b Colluvium - Post-Seismic	24	--	0.28
	Unit 3 Transitional GF - Post-Seismic	22	--	0.35
	Unit 4 GLU - Post-Seismic	20	--	0.13
	Glacial Till	Impenetrable Bedrock		

APA Berm Limit Equilibrium Slope Stability Analysis

Name: APA Plane 3 - Post-Seismic - Mode G

Date: 11/08/2016

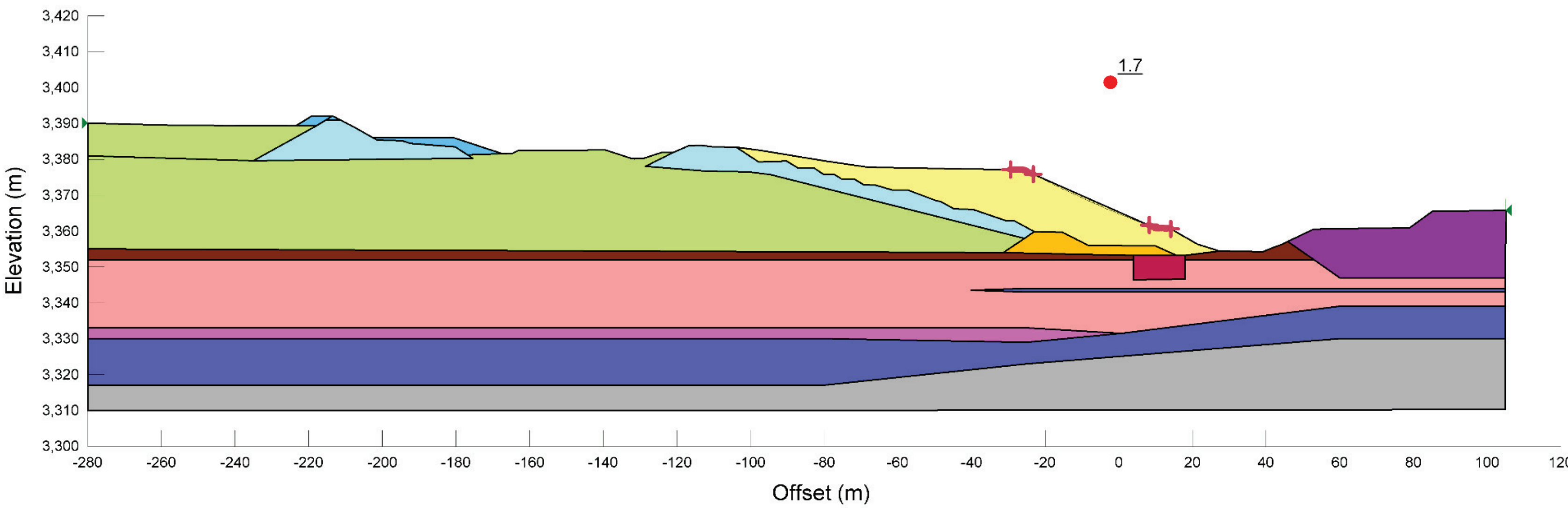


Material Type	Unit Weight (kN/m^3)	Friction Angle (°)	Tau/Sigma
Seismic Berm Fill	22	38	--
Tailings - Post-Seismic	22.5	--	0.14
Embankment Raises	22	34	--
Embankment Raises - Addition	22	34	--
Starter Embankment	22	36	--
Mine Road Fill	24	30	--
Densified Sand & Gravel	24	36	--
Unit 2a Glaciofluvial	24	34	--
Unit 2b Colluvium - Post-Seismic	24	--	0.28
Unit 3 Transitional GF - Post-Seismic	22	--	0.35
Unit 4 GLU - Post-Seismic	20	--	0.13
Glacial Till	Impenetrable Bedrock		

APA Berm Limit Equilibrium Slope Stability Analysis

Name: APA Plane 3 - Static - Mode A

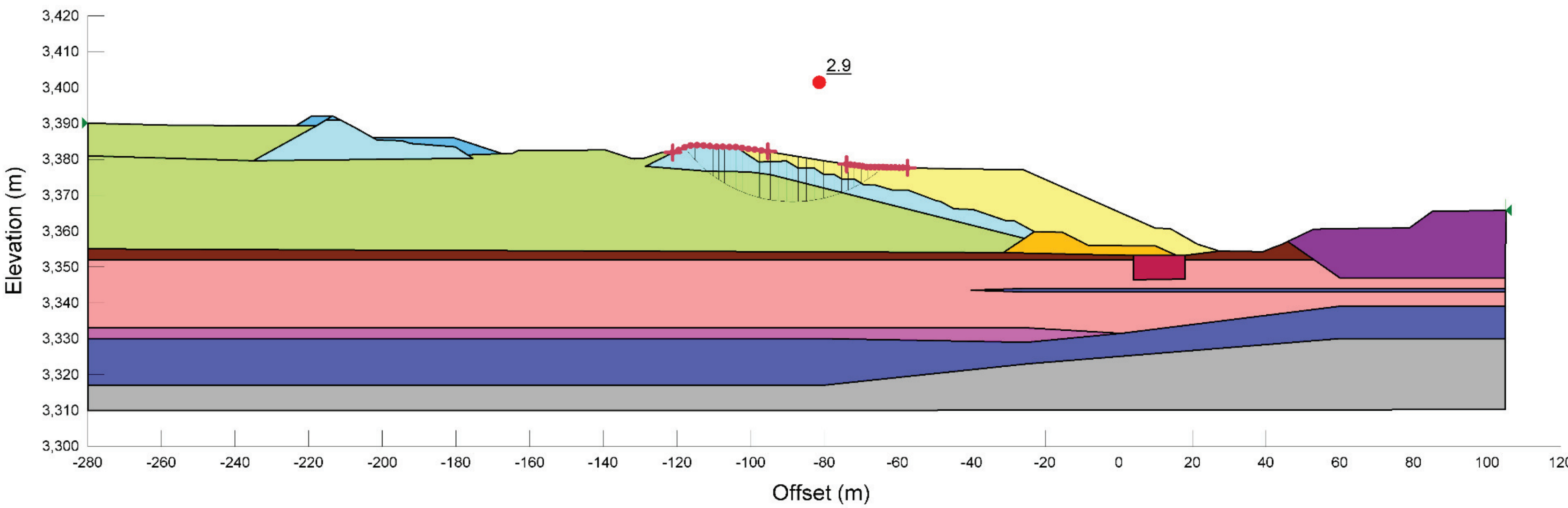
Date: 15/08/2016



	Material Type	Unit Weight (kN/m^3)	Friction Angle (°)	Tau/Sigma
	Seismic Berm Fill	22	38	--
	Tailings - Static	22.5	--	0.2
	Embankment Raises	22	34	--
	Embankment Raises - Addition	22	34	--
	Starter Embankment	22	36	--
	Mine Road Fill	24	30	--
	Densified Sand & Gravel	24	36	--
	Unit 2a Glaciofluvial	24	34	--
	Unit 2b Colluvium - Static	24	34	--
	Unit 3 Transitional GF - Static	22	32	--
	Unit 4 GLU - Static	20	--	0.22
	Glacial Till	Impenetrable Bedrock		

APA Berm Limit Equilibrium Slope Stability Analysis

Name: APA Plane 3 - Static - Mode B
Date: 15/08/2016

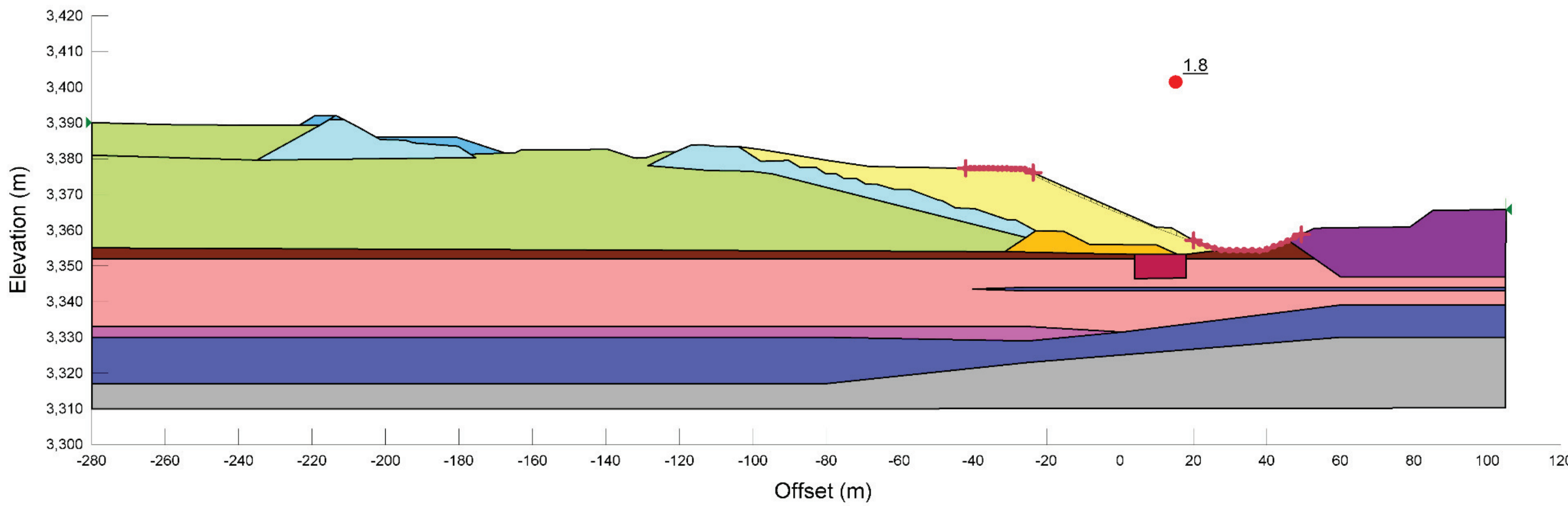


	Material Type	Unit Weight (kN/m^3)	Friction Angle (°)	Tau/Sigma
	Seismic Berm Fill	22	38	--
	Tailings - Static	22.5	--	0.2
	Embankment Raises	22	34	--
	Embankment Raises - Addition	22	34	--
	Starter Embankment	22	36	--
	Mine Road Fill	24	30	--
	Densified Sand & Gravel	24	36	--
	Unit 2a Glaciofluvial	24	34	--
	Unit 2b Colluvium - Static	24	34	--
	Unit 3 Transitional GF - Static	22	32	--
	Unit 4 GLU - Static	20	--	0.22
	Glacial Till	Impenetrable Bedrock		

APA Berm Limit Equilibrium Slope Stability Analysis

Name: APA Plane 3 - Static - Mode C

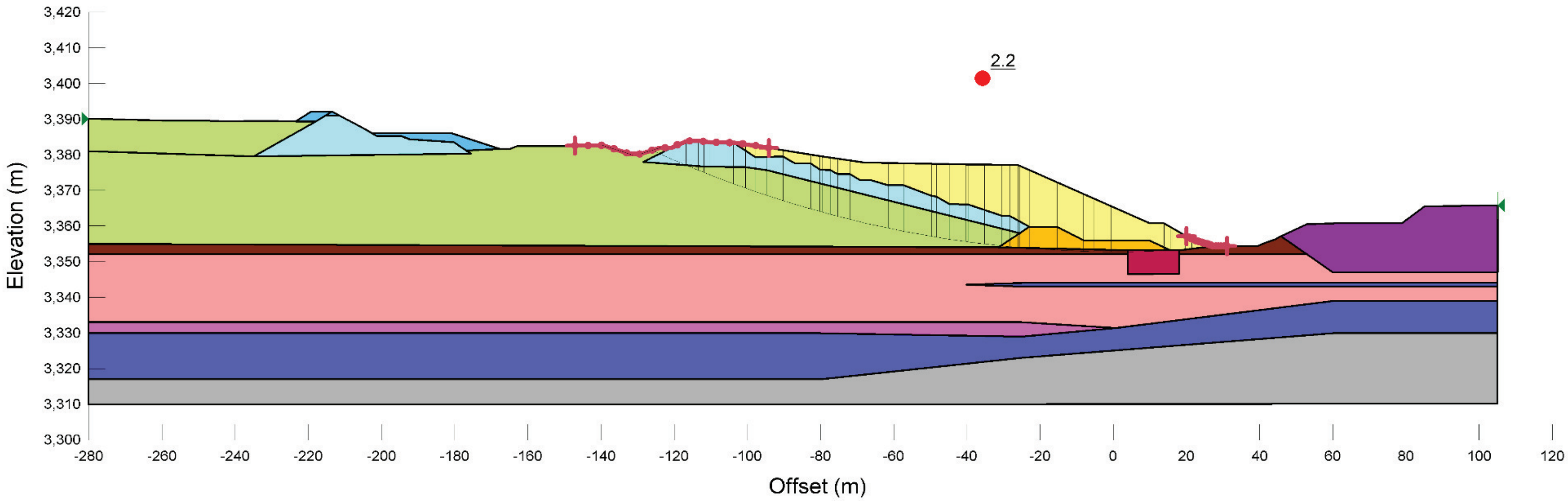
Date: 15/08/2016



Material Type	Unit Weight (kN/m^3)	Friction Angle (°)	Tau/Sigma
Seismic Berm Fill	22	38	--
Tailings - Static	22.5	--	0.2
Embankment Raises	22	34	--
Embankment Raises - Addition	22	34	--
Starter Embankment	22	36	--
Mine Road Fill	24	30	--
Densified Sand & Gravel	24	36	--
Unit 2a Glaciofluvial	24	34	--
Unit 2b Colluvium - Static	24	34	--
Unit 3 Transitional GF - Static	22	32	--
Unit 4 GLU - Static	20	--	0.22
Glacial Till	Impenetrable Bedrock		

APA Berm Limit Equilibrium Slope Stability Analysis

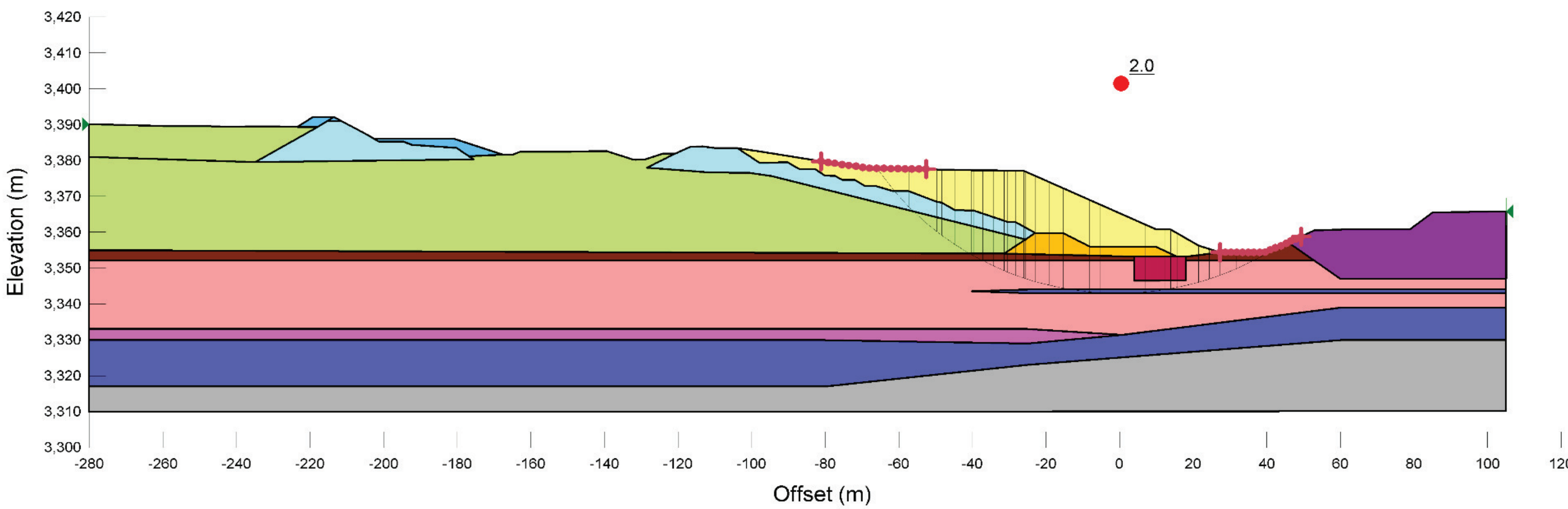
Name: APA Plane 3 - Static - Mode D
Date: 15/08/2016



Material Type	Unit Weight (kN/m^3)	Friction Angle (°)	Tau/Sigma
Seismic Berm Fill	22	38	--
Tailings - Static	22.5	--	0.2
Embankment Raises	22	34	--
Embankment Raises - Addition	22	34	--
Starter Embankment	22	36	--
Mine Road Fill	24	30	--
Densified Sand & Gravel	24	36	--
Unit 2a Glaciofluvial	24	34	--
Unit 2b Colluvium - Static	24	34	--
Unit 3 Transitional GF - Static	22	32	--
Unit 4 GLU - Static	20	--	0.22
Glacial Till	Impenetrable Bedrock		

APA Berm Limit Equilibrium Slope Stability Analysis

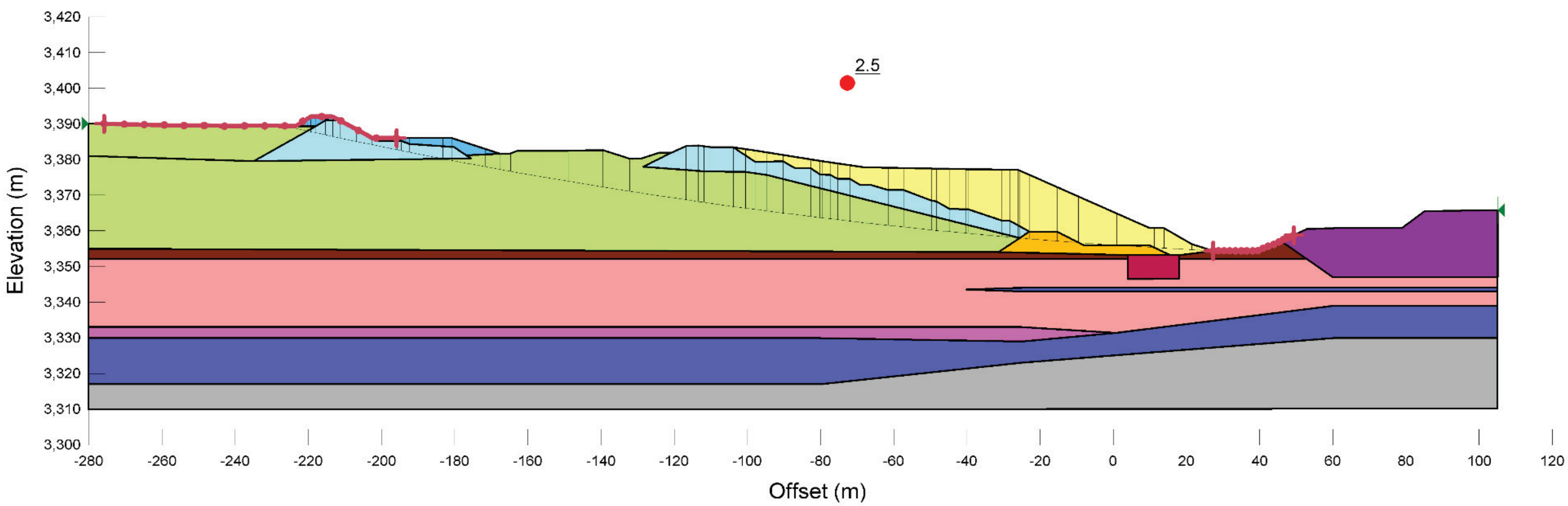
Name: APA Plane 3 - Static - Mode E
Date: 15/08/2016



Material Type	Unit Weight (kN/m^3)	Friction Angle (°)	Tau/Sigma
Seismic Berm Fill	22	38	--
Tailings - Static	22.5	--	0.2
Embankment Raises	22	34	--
Embankment Raises - Addition	22	34	--
Starter Embankment	22	36	--
Mine Road Fill	24	30	--
Densified Sand & Gravel	24	36	--
Unit 2a Glaciofluvial	24	34	--
Unit 2b Colluvium - Static	24	34	--
Unit 3 Transitional GF - Static	22	32	--
Unit 4 GLU - Static	20	--	0.22
Glacial Till	Impenetrable Bedrock		

APA Berm Limit Equilibrium Slope Stability Analysis

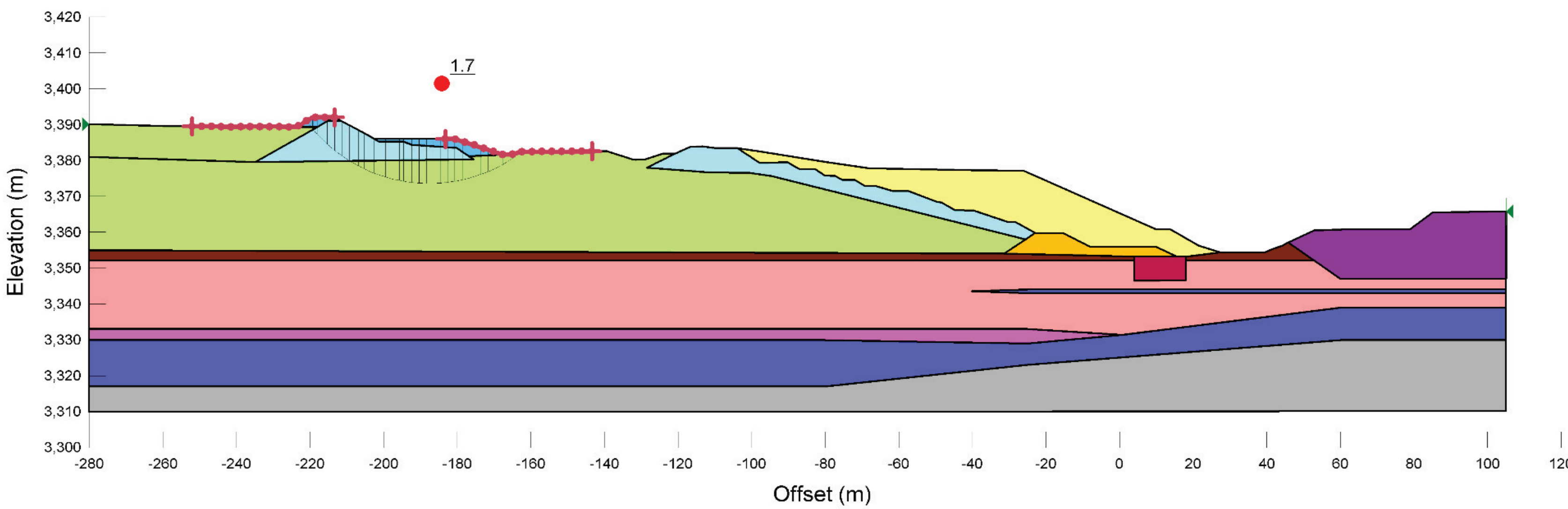
Name: APA Plane 3 - Static - Mode F
Date: 15/08/2016



Material Type	Unit Weight (kN/m^3)	Friction Angle (°)	Tau/Sigma
Seismic Berm Fill	22	38	--
Tailings - Static	22.5	--	0.2
Embankment Raises	22	34	--
Embankment Raises - Addition	22	34	--
Starter Embankment	22	36	--
Mine Road Fill	24	30	--
Densified Sand & Gravel	24	36	--
Unit 2a Glaciofluvial	24	34	--
Unit 2b Colluvium - Static	24	34	--
Unit 3 Transitional GF - Static	22	32	--
Unit 4 GLU - Static	20	--	0.22
Glacial Till	Impenetrable Bedrock		

APA Berm Limit Equilibrium Slope Stability Analysis

Name: APA Plane 3 - Static - Mode G
Date: 15/08/2016

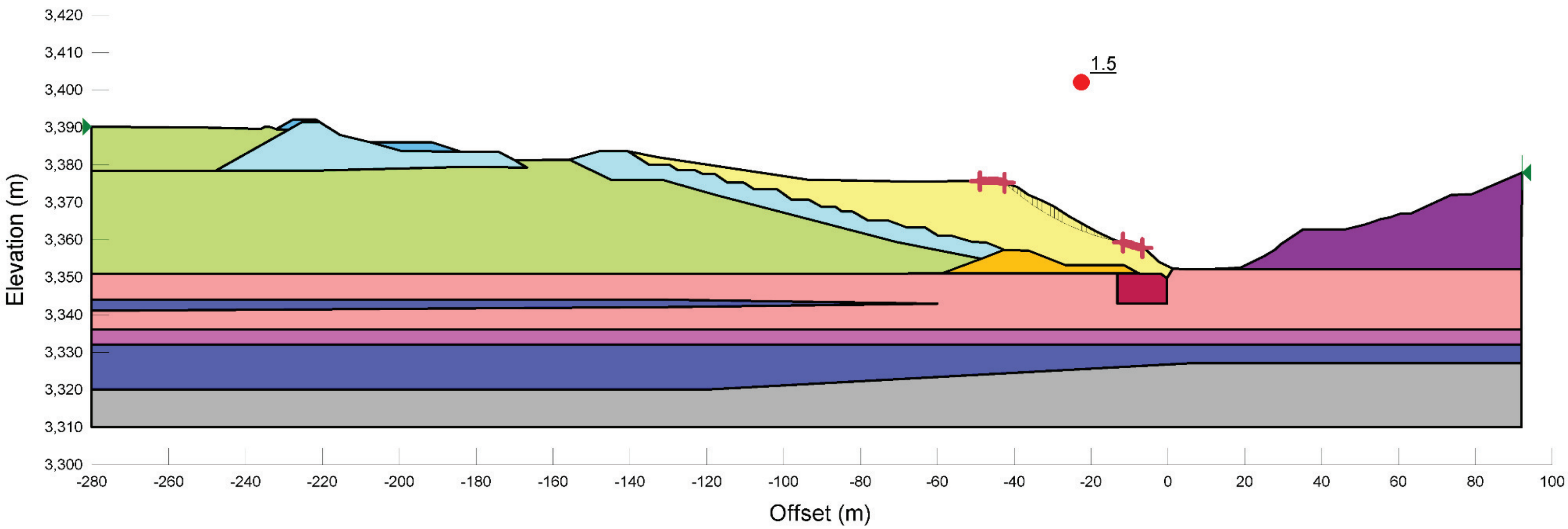


	Material Type	Unit Weight (kN/m^3)	Friction Angle (°)	Tau/Sigma
	Seismic Berm Fill	22	38	--
	Tailings - Static	22.5	--	0.2
	Embankment Raises	22	34	--
	Embankment Raises - Addition	22	34	--
	Starter Embankment	22	36	--
	Mine Road Fill	24	30	--
	Densified Sand & Gravel	24	36	--
	Unit 2a Glaciofluvial	24	34	--
	Unit 2b Colluvium - Static	24	34	--
	Unit 3 Transitional GF - Static	22	32	--
	Unit 4 GLU - Static	20	--	0.22
	Glacial Till	Impenetrable Bedrock		

APA Berm Limit Equilibrium Slope Stability Analysis

Name: APA Plane 4 - Post-Seismic - Mode A

Date: 18/08/2016

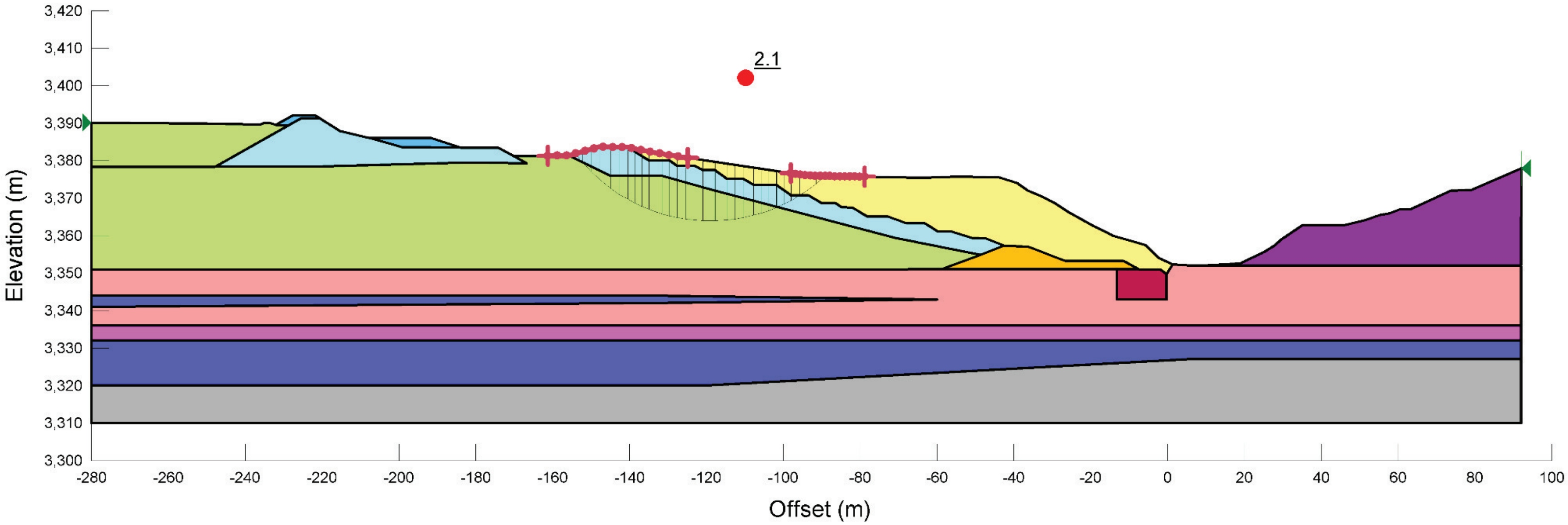


Material Type	Unit Weight (kN/m^3)	Friction Angle (°)	Tau/Sigma
Seismic Berm Fill	22	38	--
Tailings - Post-Seismic	22.5	--	0.14
Embankment Raises	22	34	--
Embankment Raises - Addition	22	34	--
Starter Embankment	22	36	--
Mine Road Fill	24	30	--
Densified Sand & Gravel	24	36	--
Unit 2a Glaciofluvial	24	34	--
Unit 2b Colluvium - Post-Seismic	24	--	0.28
Unit 3 Transitional GF - Post-Seismic	22	--	0.35
Unit 4 GLU - Post-Seismic	20	--	0.13
Glacial Till	Impenetrable Bedrock		

APA Berm Limit Equilibrium Slope Stability Analysis

Name: APA Plane 4 - Post-Seismic - Mode B

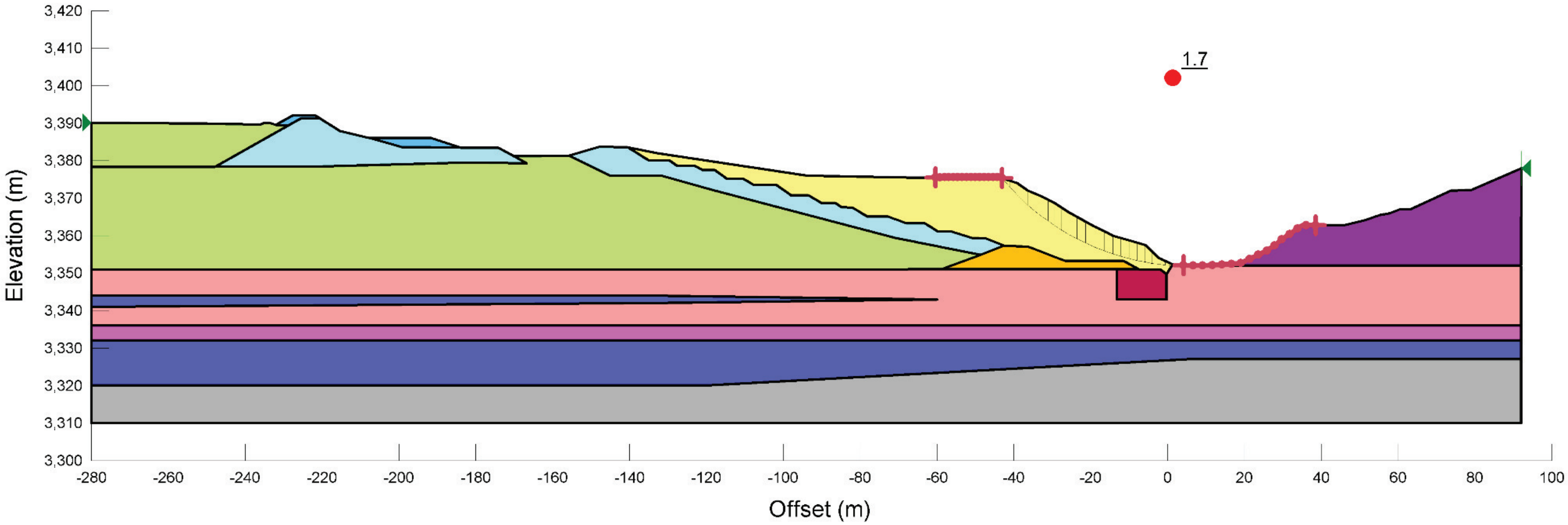
Date: 11/08/2016



Material Type	Unit Weight (kN/m^3)	Friction Angle (°)	Tau/Sigma
Seismic Berm Fill	22	38	--
Tailings - Post-Seismic	22.5	--	0.14
Embankment Raises	22	34	--
Embankment Raises - Addition	22	34	--
Starter Embankment	22	36	--
Mine Road Fill	24	30	--
Densified Sand & Gravel	24	36	--
Unit 2a Glaciofluvial	24	34	--
Unit 2b Colluvium - Post-Seismic	24	--	0.28
Unit 3 Transitional GF - Post-Seismic	22	--	0.35
Unit 4 GLU - Post-Seismic	20	--	0.13
Glacial Till	Impenetrable Bedrock		

APA Berm Limit Equilibrium Slope Stability Analysis

Name: APA Plane 4 - Post-Seismic - Mode C
Date: 11/08/2016

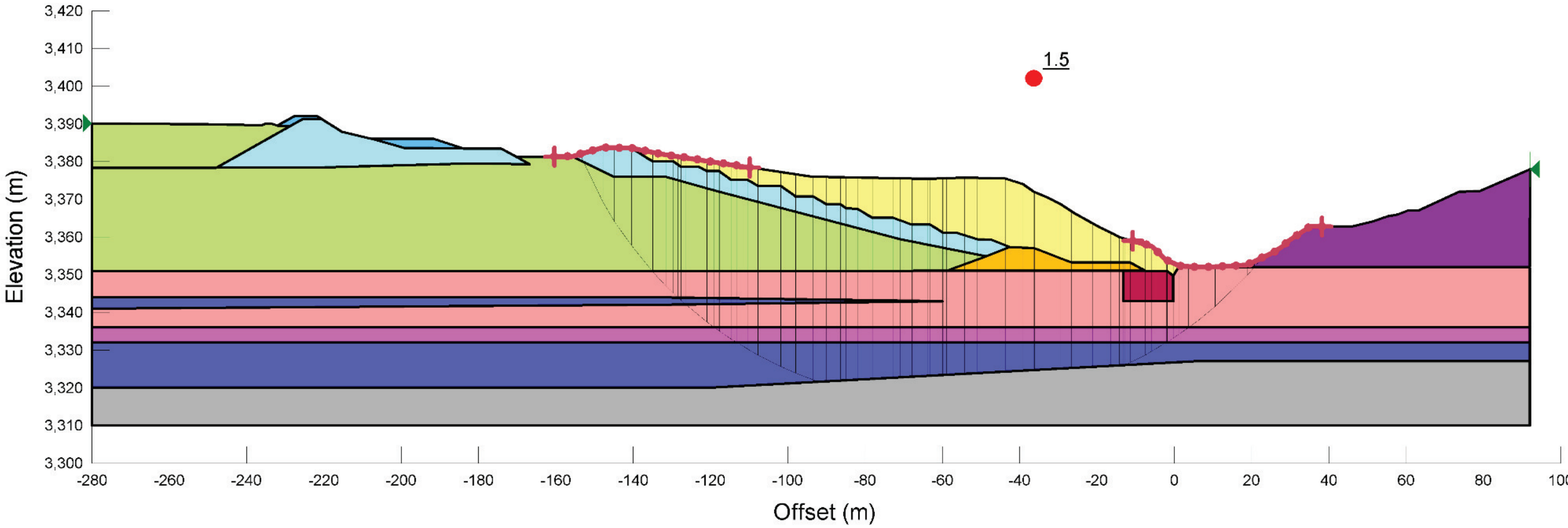


Material Type	Unit Weight (kN/m^3)	Friction Angle (°)	Tau/Sigma
Seismic Berm Fill	22	38	--
Tailings - Post-Seismic	22.5	--	0.14
Embankment Raises	22	34	--
Embankment Raises - Addition	22	34	--
Starter Embankment	22	36	--
Mine Road Fill	24	30	--
Densified Sand & Gravel	24	36	--
Unit 2a Glaciofluvial	24	34	--
Unit 2b Colluvium - Post-Seismic	24	--	0.28
Unit 3 Transitional GF - Post-Seismic	22	--	0.35
Unit 4 GLU - Post-Seismic	20	--	0.13
Glacial Till	Impenetrable Bedrock		

APA Berm Limit Equilibrium Slope Stability Analysis

Name: APA Plane 4 - Post-Seismic - Mode D

Date: 11/08/2016

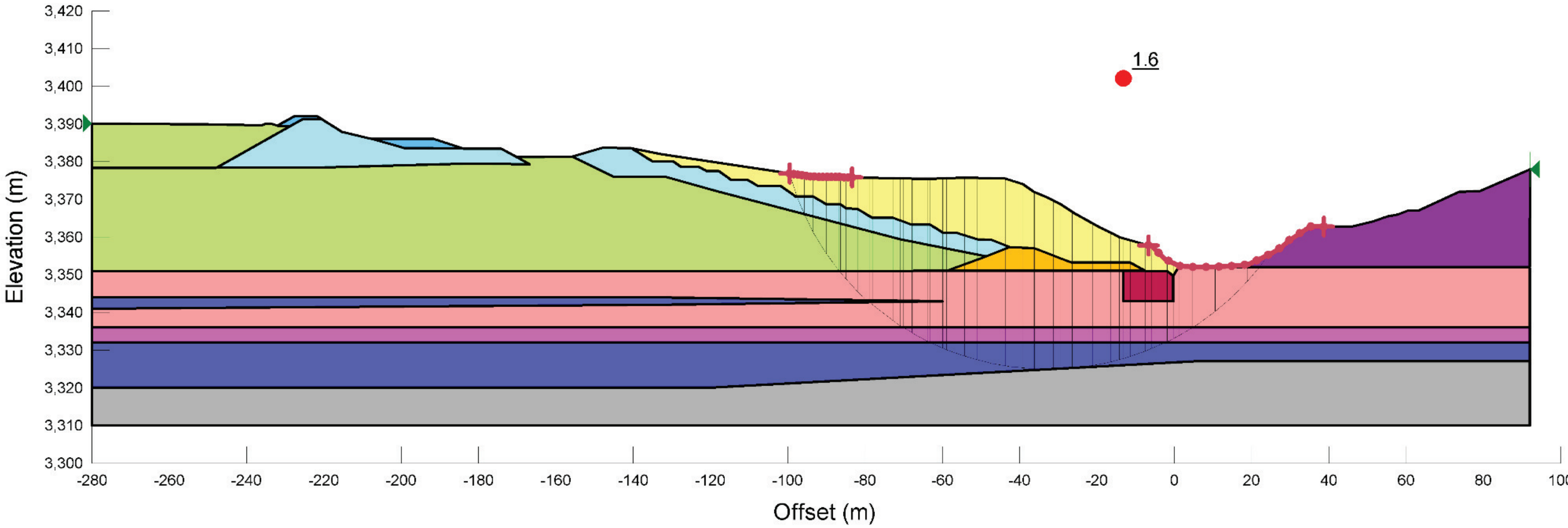


Material Type	Unit Weight (kN/m^3)	Friction Angle (°)	Tau/Sigma
Seismic Berm Fill	22	38	--
Tailings - Post-Seismic	22.5	--	0.14
Embankment Raises	22	34	--
Embankment Raises - Addition	22	34	--
Starter Embankment	22	36	--
Mine Road Fill	24	30	--
Densified Sand & Gravel	24	36	--
Unit 2a Glaciofluvial	24	34	--
Unit 2b Colluvium - Post-Seismic	24	--	0.28
Unit 3 Transitional GF - Post-Seismic	22	--	0.35
Unit 4 GLU - Post-Seismic	20	--	0.13
Glacial Till	Impenetrable Bedrock		

APA Berm Limit Equilibrium Slope Stability Analysis

Name: APA Plane 4 - Post-Seismic - Mode E

Date: 11/08/2016

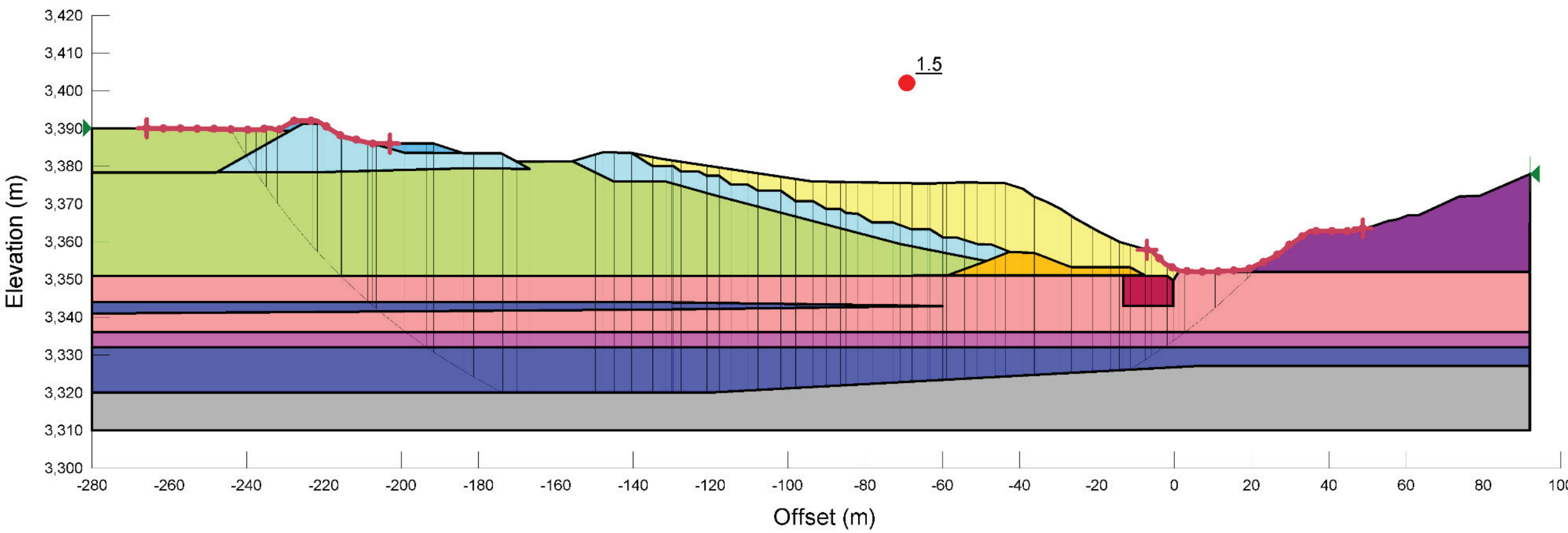


	Material Type	Unit Weight (kN/m^3)	Friction Angle (°)	Tau/Sigma
	Seismic Berm Fill	22	38	--
	Tailings - Post-Seismic	22.5	--	0.14
	Embankment Raises	22	34	--
	Embankment Raises - Addition	22	34	--
	Starter Embankment	22	36	--
	Mine Road Fill	24	30	--
	Densified Sand & Gravel	24	36	--
	Unit 2a Glaciofluvial	24	34	--
	Unit 2b Colluvium - Post-Seismic	24	--	0.28
	Unit 3 Transitional GF - Post-Seismic	22	--	0.35
	Unit 4 GLU - Post-Seismic	20	--	0.13
	Glacial Till	Impenetrable Bedrock		

APA Berm Limit Equilibrium Slope Stability Analysis

Name: APA Plane 4 - Post-Seismic - Mode F

Date: 11/08/2016

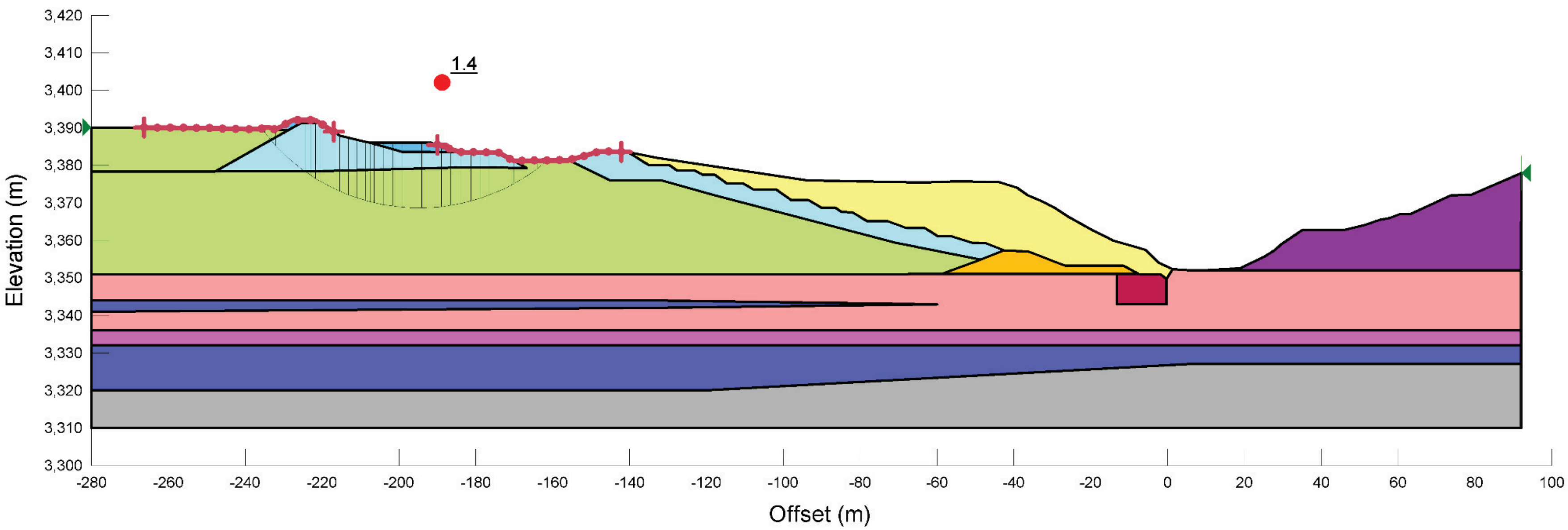


	Material Type	Unit Weight (kN/m^3)	Friction Angle (°)	Tau/Sigma
	Seismic Berm Fill	22	38	--
	Tailings - Post-Seismic	22.5	--	0.14
	Embankment Raises	22	34	--
	Embankment Raises - Addition	22	34	--
	Starter Embankment	22	36	--
	Mine Road Fill	24	30	--
	Densified Sand & Gravel	24	36	--
	Unit 2a Glaciofluvial	24	34	--
	Unit 2b Colluvium - Post-Seismic	24	--	0.28
	Unit 3 Transitional GF - Post-Seismic	22	--	0.35
	Unit 4 GLU - Post-Seismic	20	--	0.13
	Glacial Till	Impenetrable Bedrock		

APA Berm Limit Equilibrium Slope Stability Analysis

Name: APA Plane 4 - Post-Seismic - Mode G

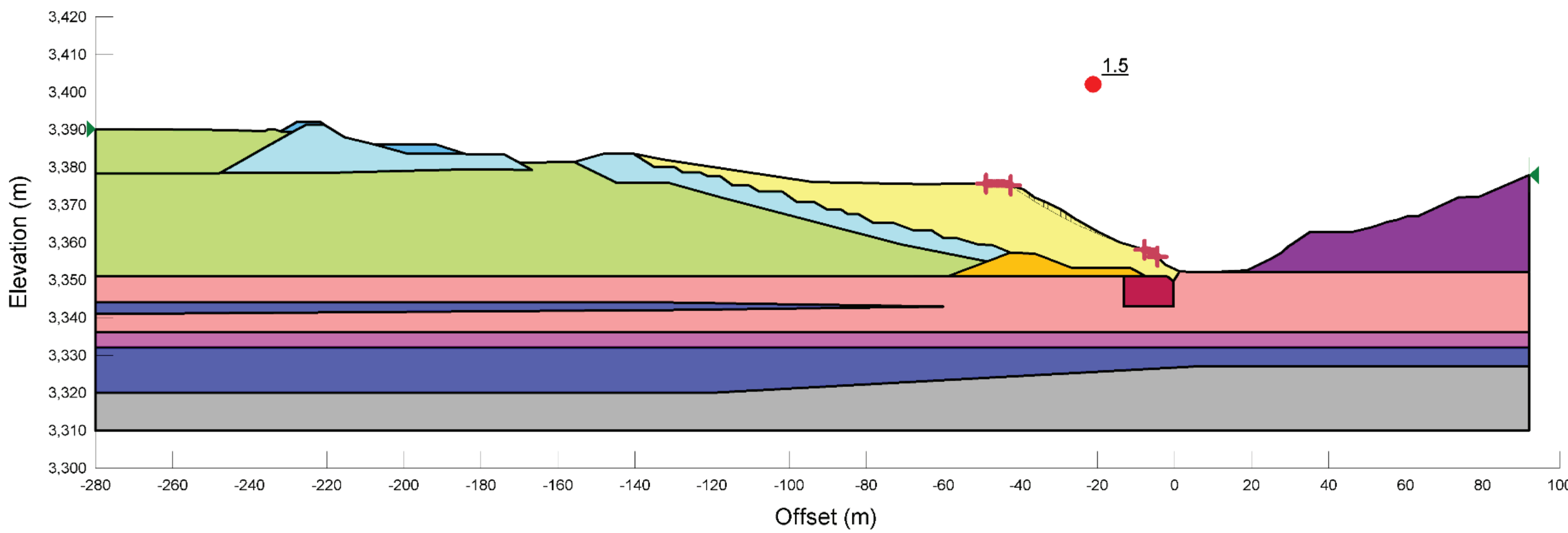
Date: 11/08/2016



Material Type	Unit Weight (kN/m^3)	Friction Angle (°)	Tau/Sigma
Seismic Berm Fill	22	38	--
Tailings - Post-Seismic	22.5	--	0.14
Embankment Raises	22	34	--
Embankment Raises - Addition	22	34	--
Starter Embankment	22	36	--
Mine Road Fill	24	30	--
Densified Sand & Gravel	24	36	--
Unit 2a Glaciofluvial	24	34	--
Unit 2b Colluvium - Post-Seismic	24	--	0.28
Unit 3 Transitional GF - Post-Seismic	22	--	0.35
Unit 4 GLU - Post-Seismic	20	--	0.13
Glacial Till	Impenetrable Bedrock		

APA Berm Limit Equilibrium Slope Stability Analysis

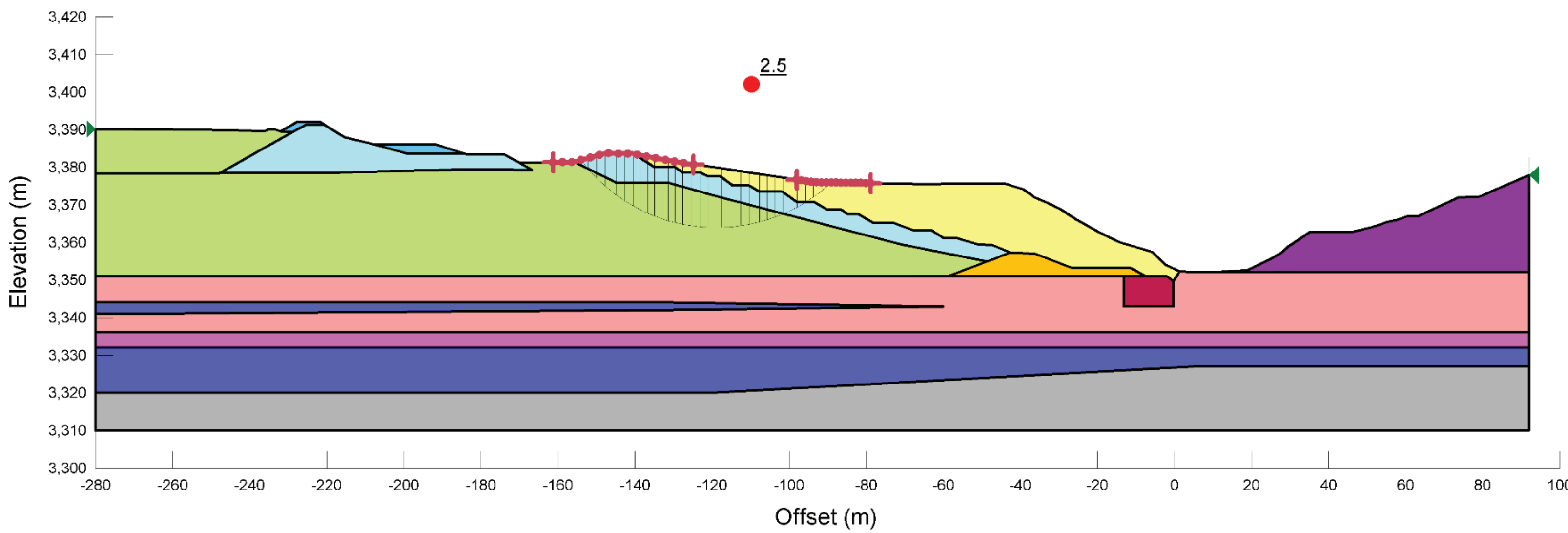
Name: APA Plane 4 - Static - Mode A
Date: 15/08/2016



Material Type	Unit Weight (kN/m^3)	Friction Angle (°)	Tau/Sigma
Seismic Berm Fill	22	38	--
Tailings - Static	22.5	--	0.2
Embankment Raises	22	34	--
Embankment Raises - Addition	22	34	--
Starter Embankment	22	36	--
Mine Road Fill	24	30	--
Densified Sand & Gravel	24	36	--
Unit 2a Glaciofluvial	24	34	--
Unit 2b Colluvium - Static	24	34	--
Unit 3 Transitional GF - Static	22	32	--
Unit 4 GLU - Static	20	--	0.22
Glacial Till	Impenetrable Bedrock		

APA Berm Limit Equilibrium Slope Stability Analysis

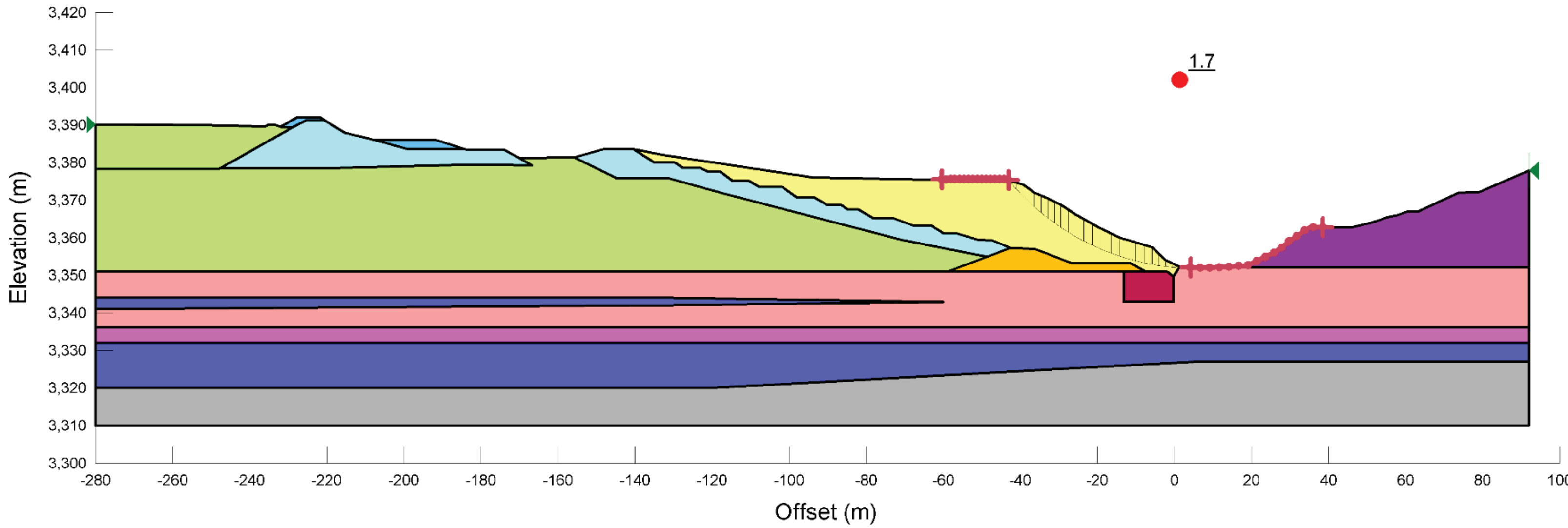
Name: APA Plane 4 - Static - Mode B
Date: 15/08/2016



Material Type	Unit Weight (kN/m^3)	Friction Angle (°)	Tau/Sigma
Seismic Berm Fill	22	38	--
Tailings - Static	22.5	--	0.2
Embankment Raises	22	34	--
Embankment Raises - Addition	22	34	--
Starter Embankment	22	36	--
Mine Road Fill	24	30	--
Densified Sand & Gravel	24	36	--
Unit 2a Glaciofluvial	24	34	--
Unit 2b Colluvium - Static	24	34	--
Unit 3 Transitional GF - Static	22	32	--
Unit 4 GLU - Static	20	--	0.22
Glacial Till	Impenetrable Bedrock		

APA Berm Limit Equilibrium Slope Stability Analysis

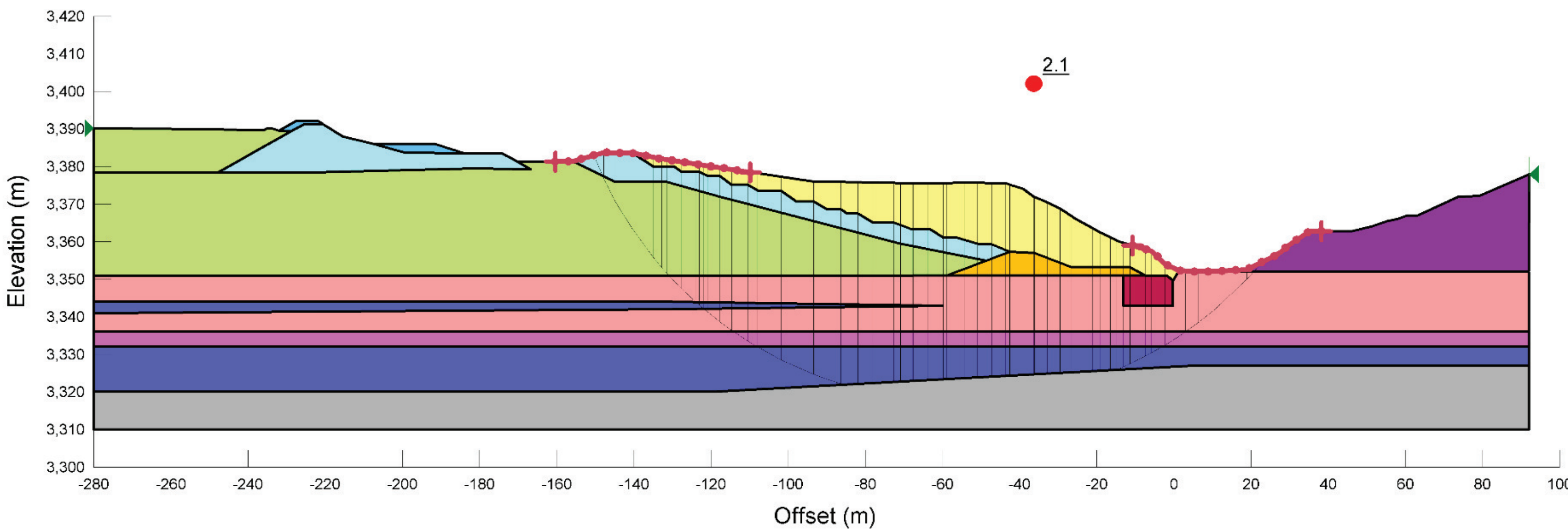
Name: APA Plane 4 - Static - Mode C
Date: 15/08/2016



Material Type	Unit Weight (kN/m^3)	Friction Angle (°)	Tau/Sigma
Seismic Berm Fill	22	38	--
Tailings - Static	22.5	--	0.2
Embankment Raises	22	34	--
Embankment Raises - Addition	22	34	--
Starter Embankment	22	36	--
Mine Road Fill	24	30	--
Densified Sand & Gravel	24	36	--
Unit 2a Glaciofluvial	24	34	--
Unit 2b Colluvium - Static	24	34	--
Unit 3 Transitional GF - Static	22	32	--
Unit 4 GLU - Static	20	--	0.22
Glacial Till	Impenetrable Bedrock		

APA Berm Limit Equilibrium Slope Stability Analysis

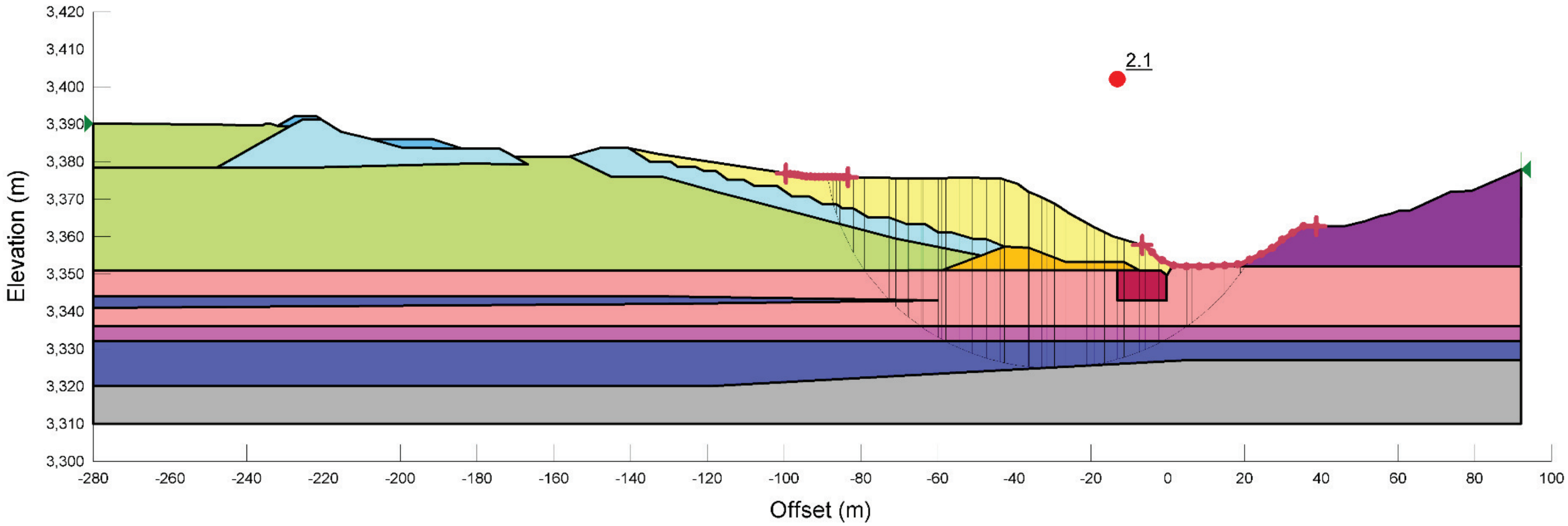
Name: APA Plane 4 - Static - Mode D
Date: 15/08/2016



Material Type	Unit Weight (kN/m^3)	Friction Angle (°)	Tau/Sigma
Seismic Berm Fill	22	38	--
Tailings - Static	22.5	--	0.2
Embankment Raises	22	34	--
Embankment Raises - Addition	22	34	--
Starter Embankment	22	36	--
Mine Road Fill	24	30	--
Densified Sand & Gravel	24	36	--
Unit 2a Glaciofluvial	24	34	--
Unit 2b Colluvium - Static	24	34	--
Unit 3 Transitional GF - Static	22	32	--
Unit 4 GLU - Static	20	--	0.22
Glacial Till	Impenetrable Bedrock		

APA Berm Limit Equilibrium Slope Stability Analysis

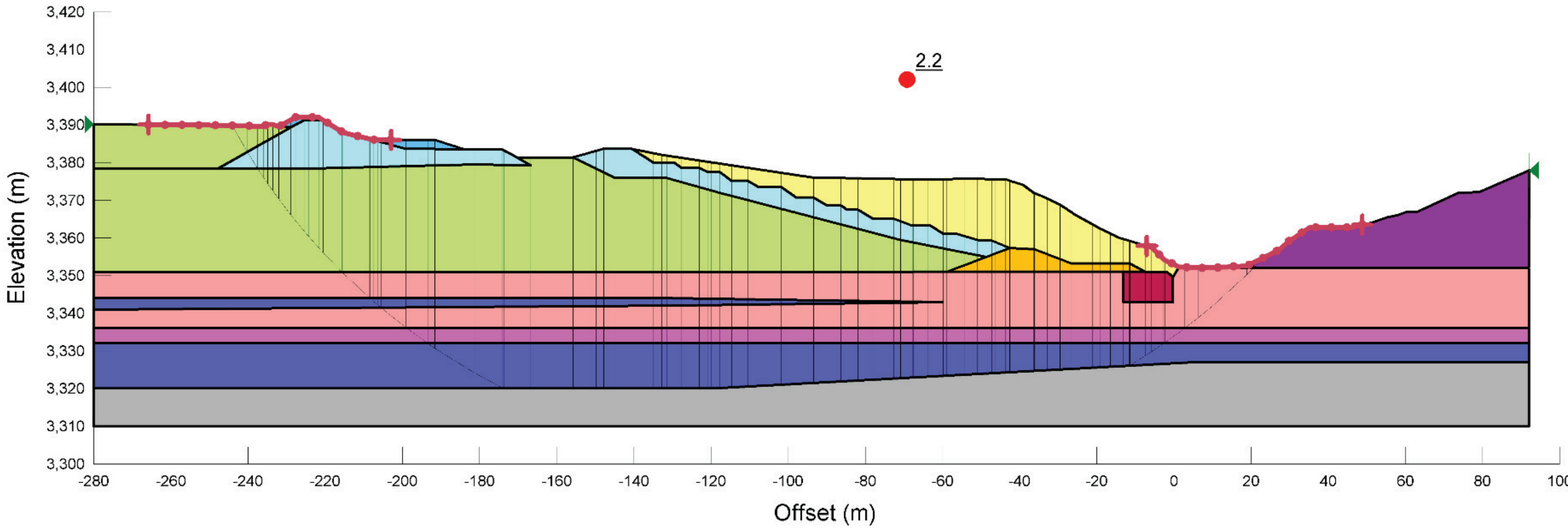
Name: APA Plane 4 - Static - Mode E
Date: 15/08/2016



	Material Type	Unit Weight (kN/m^3)	Friction Angle (°)	Tau/Sigma
	Seismic Berm Fill	22	38	--
	Tailings - Static	22.5	--	0.2
	Embankment Raises	22	34	--
	Embankment Raises - Addition	22	34	--
	Starter Embankment	22	36	--
	Mine Road Fill	24	30	--
	Densified Sand & Gravel	24	36	--
	Unit 2a Glaciofluvial	24	34	--
	Unit 2b Colluvium - Static	24	34	--
	Unit 3 Transitional GF - Static	22	32	--
	Unit 4 GLU - Static	20	--	0.22
	Glacial Till	Impenetrable Bedrock		

APA Berm Limit Equilibrium Slope Stability Analysis

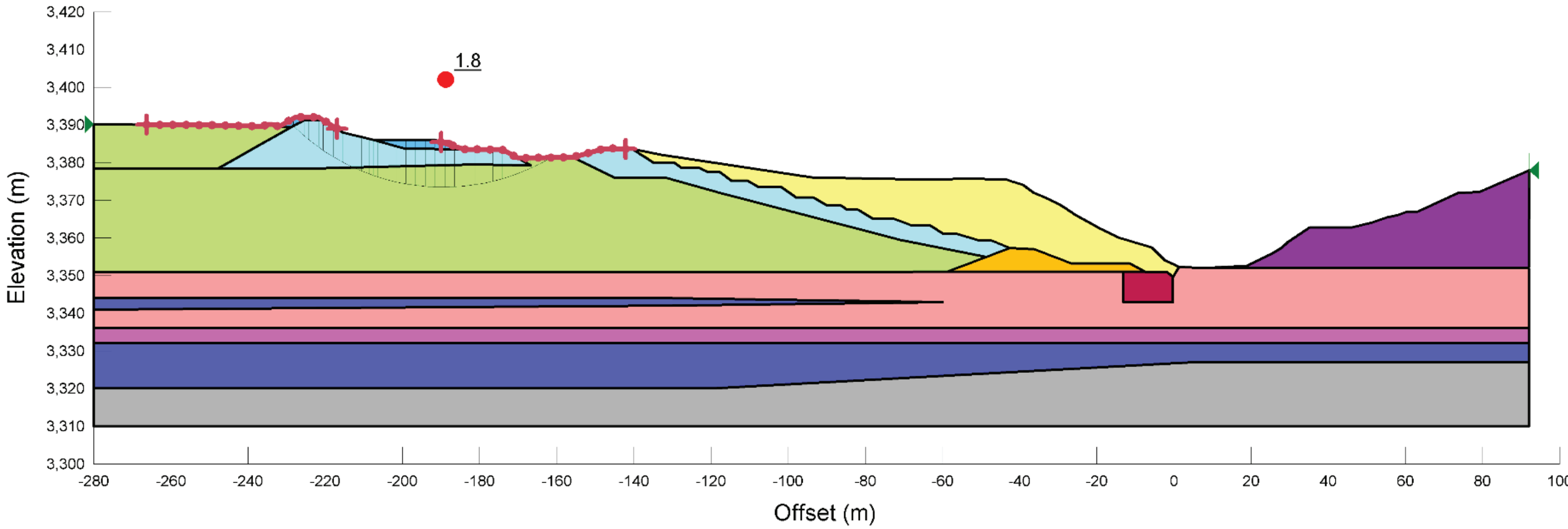
Name: APA Plane 4 - Static - Mode F
Date: 15/08/2016



Material Type	Unit Weight (kN/m^3)	Friction Angle (°)	Tau/Sigma
Seismic Berm Fill	22	38	--
Tailings - Static	22.5	--	0.2
Embankment Raises	22	34	--
Embankment Raises - Addition	22	34	--
Starter Embankment	22	36	--
Mine Road Fill	24	30	--
Densified Sand & Gravel	24	36	--
Unit 2a Glaciofluvial	24	34	--
Unit 2b Colluvium - Static	24	34	--
Unit 3 Transitional GF - Static	22	32	--
Unit 4 GLU - Static	20	--	0.22
Glacial Till	Impenetrable Bedrock		

APA Berm Limit Equilibrium Slope Stability Analysis

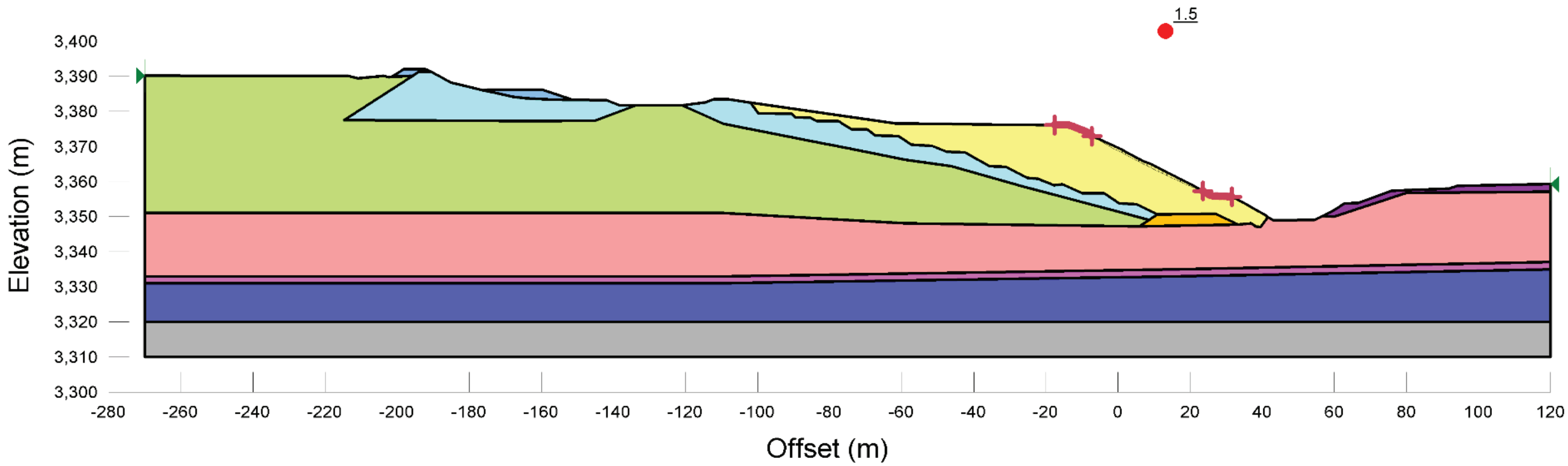
Name: APA Plane 4 - Static - Mode G
Date: 15/08/2016



Material Type	Unit Weight (kN/m^3)	Friction Angle (°)	Tau/Sigma
Seismic Berm Fill	22	38	--
Tailings - Static	22.5	--	0.2
Embankment Raises	22	34	--
Embankment Raises - Addition	22	34	--
Starter Embankment	22	36	--
Mine Road Fill	24	30	--
Densified Sand & Gravel	24	36	--
Unit 2a Glaciofluvial	24	34	--
Unit 2b Colluvium - Static	24	34	--
Unit 3 Transitional GF - Static	22	32	--
Unit 4 GLU - Static	20	--	0.22
Glacial Till	Impenetrable Bedrock		

APA Berm Limit Equilibrium Slope Stability Analysis

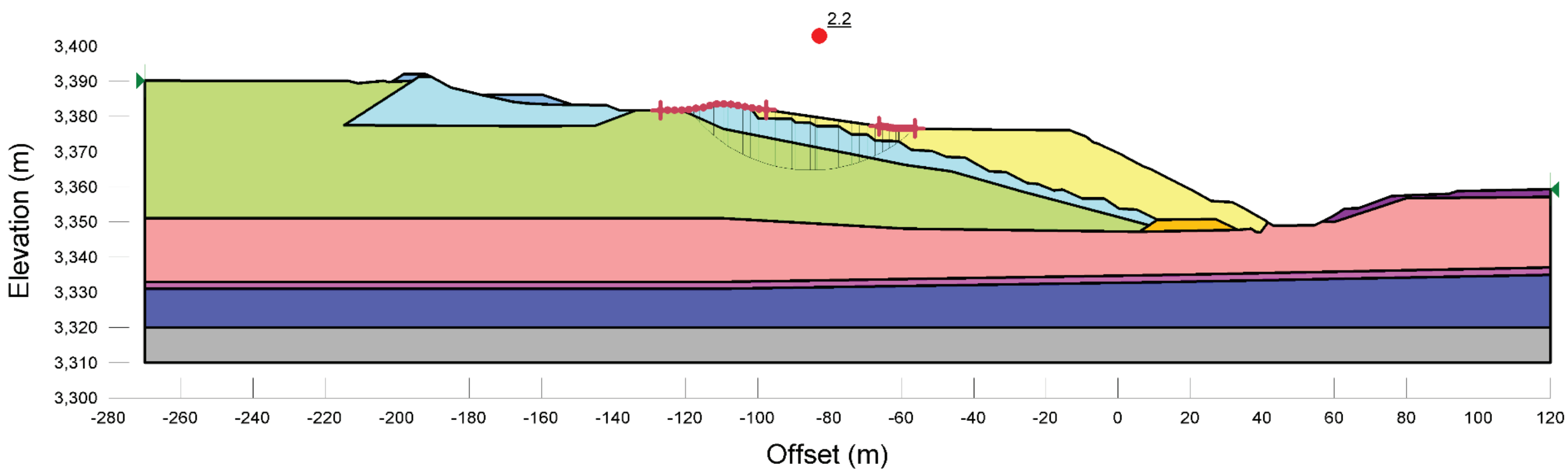
Name: APA Plane 5 - Post-Seismic - Mode A
Date: 12/08/2016



Material Type	Unit Weight (kN/m^3)	Friction Angle (°)	Tau/Sigma
Seismic Berm Fill	22	38	--
Tailings - Post-Seismic	22.5	--	0.14
Embankment Raises	22	34	--
Embankment Raises - Addition	22	34	--
Starter Embankment	22	36	--
Mine Road Fill	24	30	--
Densified Sand & Gravel	24	36	--
Unit 2a Glaciofluvial	24	34	--
Unit 2b Colluvium - Post-Seismic	24	--	0.28
Unit 3 Transitional GF - Post-Seismic	22	--	0.35
Unit 4 GLU - Post-Seismic	20	--	0.13
Glacial Till	Impenetrable Bedrock		

APA Berm Limit Equilibrium Slope Stability Analysis

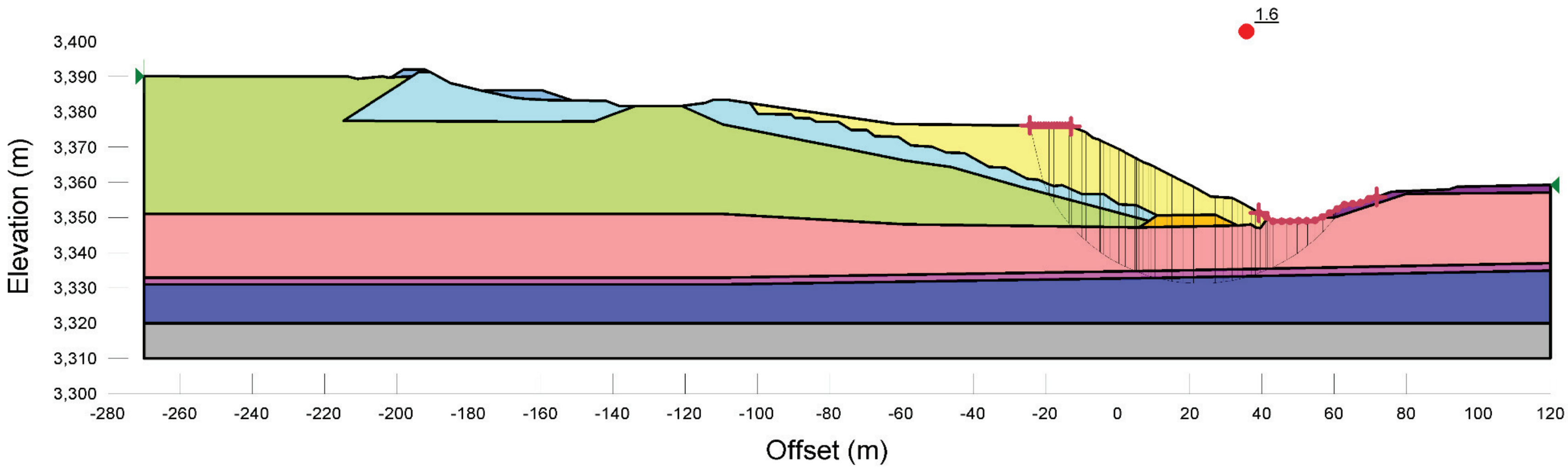
Name: APA Plane 5 - Post-Seismic - Mode B
Date: 12/08/2016



Material Type	Unit Weight (kN/m^3)	Friction Angle (°)	Tau/Sigma
Seismic Berm Fill	22	38	--
Tailings - Post-Seismic	22.5	--	0.14
Embankment Raises	22	34	--
Embankment Raises - Addition	22	34	--
Starter Embankment	22	36	--
Mine Road Fill	24	30	--
Densified Sand & Gravel	24	36	--
Unit 2a Glaciofluvial	24	34	--
Unit 2b Colluvium - Post-Seismic	24	--	0.28
Unit 3 Transitional GF - Post-Seismic	22	--	0.35
Unit 4 GLU - Post-Seismic	20	--	0.13
Glacial Till	Impenetrable Bedrock		

APA Berm Limit Equilibrium Slope Stability Analysis

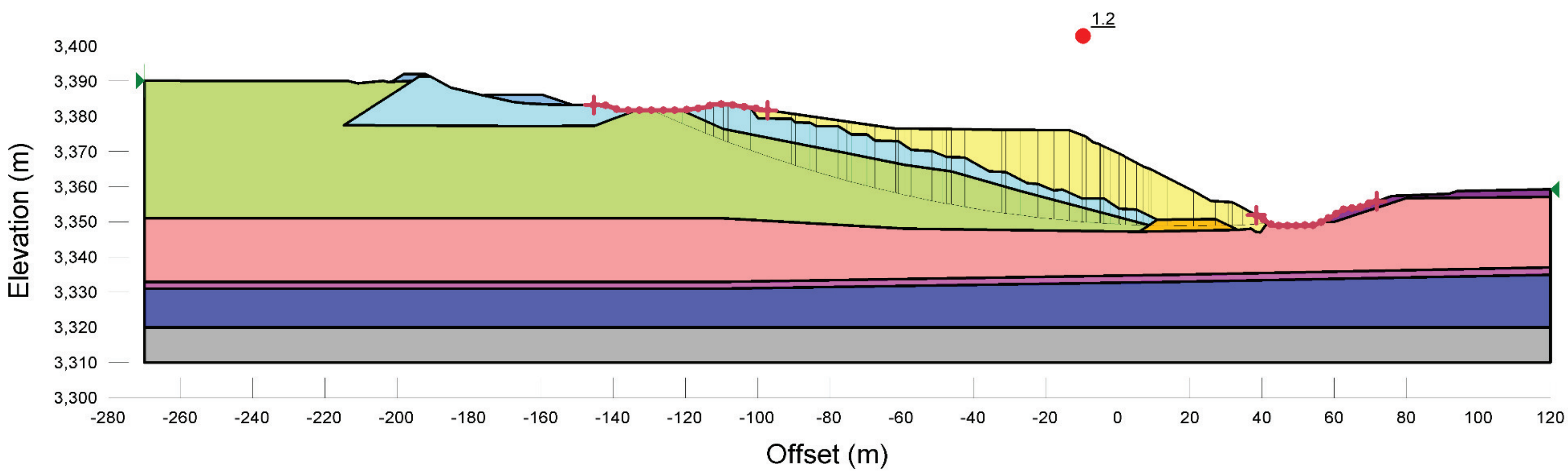
Name: APA Plane 5 - Post-Seismic - Mode C
Date: 12/08/2016



	Material Type	Unit Weight (kN/m³)	Friction Angle (°)	Tau/Sigma
	Seismic Berm Fill	22	38	--
	Tailings - Post-Seismic	22.5	--	0.14
	Embankment Raises	22	34	--
	Embankment Raises - Addition	22	34	--
	Starter Embankment	22	36	--
	Mine Road Fill	24	30	--
	Densified Sand & Gravel	24	36	--
	Unit 2a Glaciofluvial	24	34	--
	Unit 2b Colluvium - Post-Seismic	24	--	0.28
	Unit 3 Transitional GF - Post-Seismic	22	--	0.35
	Unit 4 GLU - Post-Seismic	20	--	0.13
	Glacial Till	Impenetrable Bedrock		

APA Berm Limit Equilibrium Slope Stability Analysis

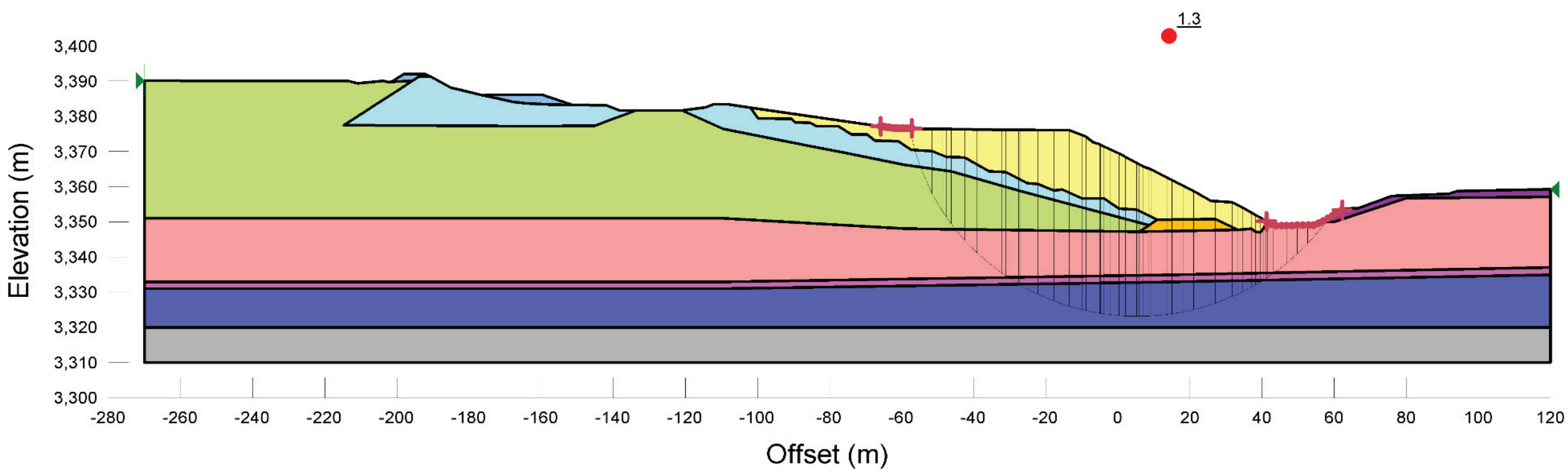
Name: APA Plane 5 - Post-Seismic - Mode D
Date: 12/08/2016



Material Type	Unit Weight (kN/m^3)	Friction Angle (°)	Tau/Sigma
Seismic Berm Fill	22	38	--
Tailings - Post-Seismic	22.5	--	0.14
Embankment Raises	22	34	--
Embankment Raises - Addition	22	34	--
Starter Embankment	22	36	--
Mine Road Fill	24	30	--
Densified Sand & Gravel	24	36	--
Unit 2a Glaciofluvial	24	34	--
Unit 2b Colluvium - Post-Seismic	24	--	0.28
Unit 3 Transitional GF - Post-Seismic	22	--	0.35
Unit 4 GLU - Post-Seismic	20	--	0.13
Glacial Till	Impenetrable Bedrock		

APA Berm Limit Equilibrium Slope Stability Analysis

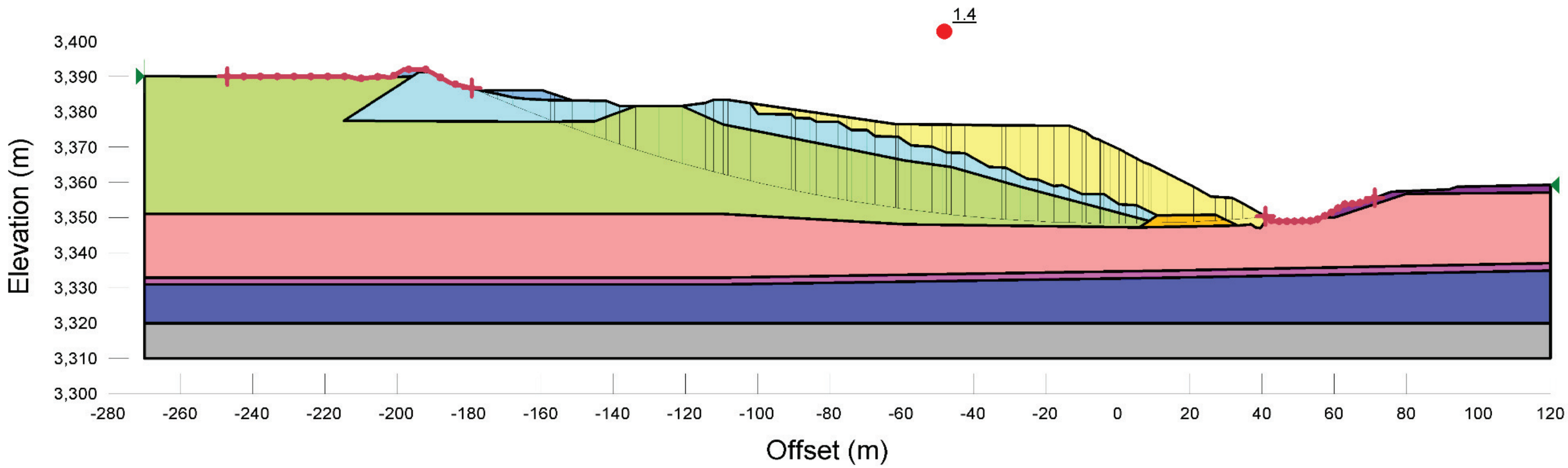
Name: APA Plane 5 - Post-Seismic - Mode E
Date: 12/08/2016



Material Type	Unit Weight (kN/m^3)	Friction Angle (°)	Tau/Sigma
Seismic Berm Fill	22	38	--
Tailings - Post-Seismic	22.5	--	0.14
Embankment Raises	22	34	--
Embankment Raises - Addition	22	34	--
Starter Embankment	22	36	--
Mine Road Fill	24	30	--
Densified Sand & Gravel	24	36	--
Unit 2a Glaciofluvial	24	34	--
Unit 2b Colluvium - Post-Seismic	24	--	0.28
Unit 3 Transitional GF - Post-Seismic	22	--	0.35
Unit 4 GLU - Post-Seismic	20	--	0.13
Glacial Till	Impenetrable Bedrock		

APA Berm Limit Equilibrium Slope Stability Analysis

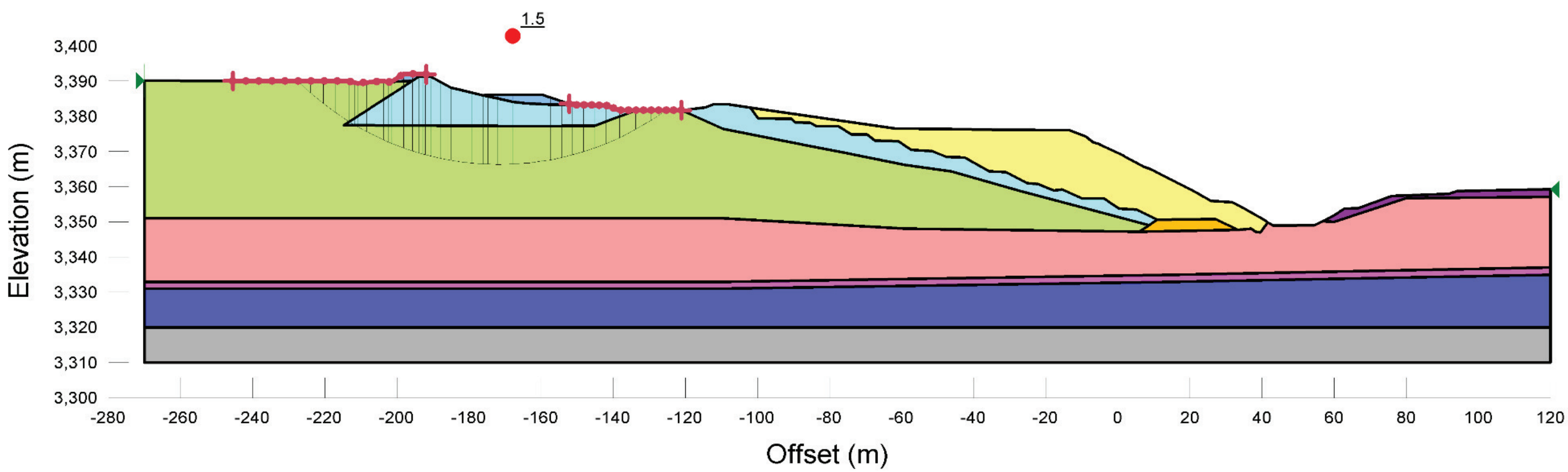
Name: APA Plane 5 - Post-Seismic - Mode F
Date: 12/08/2016



Material Type	Unit Weight (kN/m^3)	Friction Angle (°)	Tau/Sigma
Seismic Berm Fill	22	38	--
Tailings - Post-Seismic	22.5	--	0.14
Embankment Raises	22	34	--
Embankment Raises - Addition	22	34	--
Starter Embankment	22	36	--
Mine Road Fill	24	30	--
Densified Sand & Gravel	24	36	--
Unit 2a Glaciofluvial	24	34	--
Unit 2b Colluvium - Post-Seismic	24	--	0.28
Unit 3 Transitional GF - Post-Seismic	22	--	0.35
Unit 4 GLU - Post-Seismic	20	--	0.13
Glacial Till	Impenetrable Bedrock		

APA Berm Limit Equilibrium Slope Stability Analysis

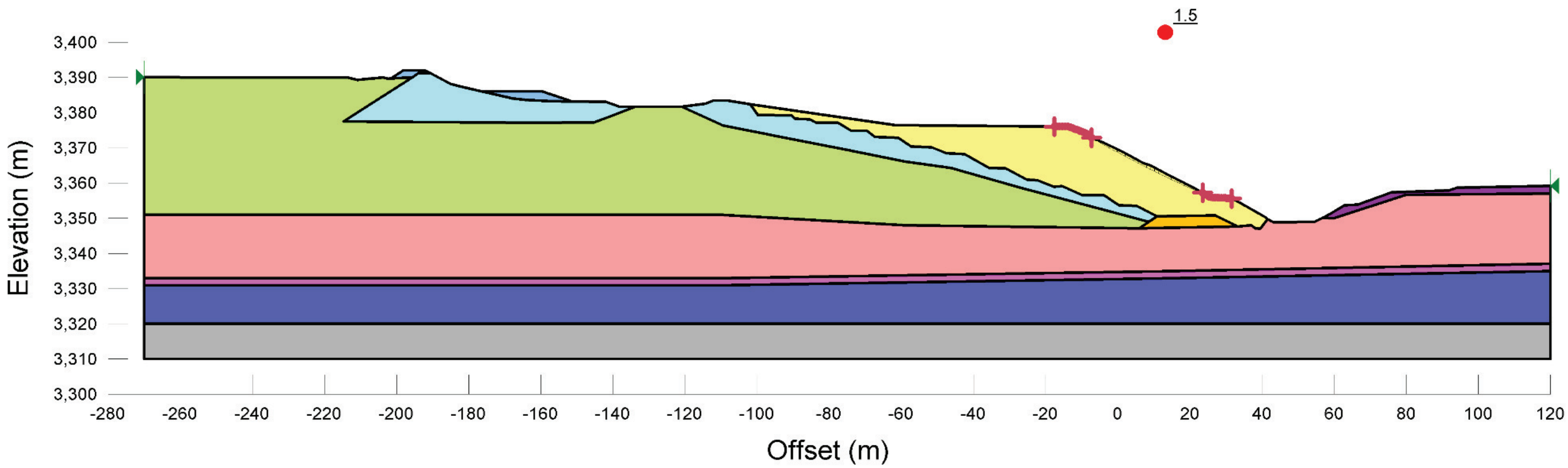
Name: APA Plane 5 - Post-Seismic - Mode G
Date: 12/08/2016



Material Type	Unit Weight (kN/m^3)	Friction Angle (°)	Tau/Sigma
Seismic Berm Fill	22	38	--
Tailings - Post-Seismic	22.5	--	0.14
Embankment Raises	22	34	--
Embankment Raises - Addition	22	34	--
Starter Embankment	22	36	--
Mine Road Fill	24	30	--
Densified Sand & Gravel	24	36	--
Unit 2a Glaciofluvial	24	34	--
Unit 2b Colluvium - Post-Seismic	24	--	0.28
Unit 3 Transitional GF - Post-Seismic	22	--	0.35
Unit 4 GLU - Post-Seismic	20	--	0.13
Glacial Till	Impenetrable Bedrock		

APA Berm Limit Equilibrium Slope Stability Analysis

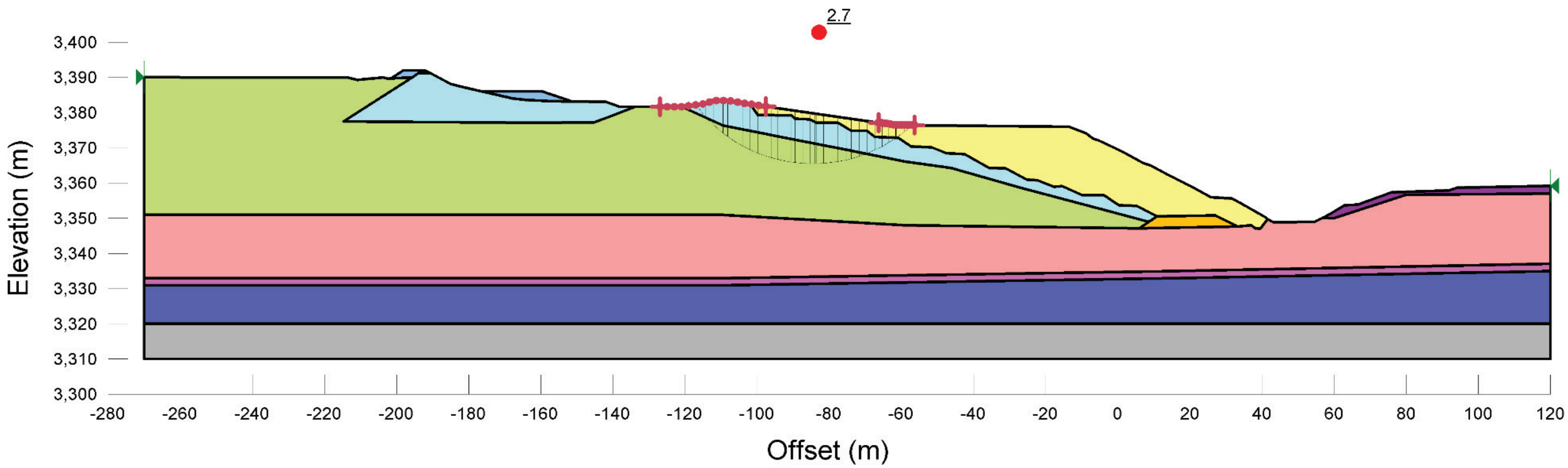
Name: APA Plane 5 - Static - Mode A
Date: 12/08/2016



Material Type	Unit Weight (kN/m^3)	Friction Angle (°)	Tau/Sigma
Seismic Berm Fill	22	38	--
Tailings - Static	22.5	--	0.2
Embankment Raises	22	34	--
Embankment Raises - Addition	22	34	--
Starter Embankment	22	36	--
Mine Road Fill	24	30	--
Densified Sand & Gravel	24	36	--
Unit 2a Glaciofluvial	24	34	--
Unit 2b Colluvium - Static	24	34	--
Unit 3 Transitional GF - Static	22	32	--
Unit 4 GLU - Static	20	--	0.22
Glacial Till	Impenetrable Bedrock		

APA Berm Limit Equilibrium Slope Stability Analysis

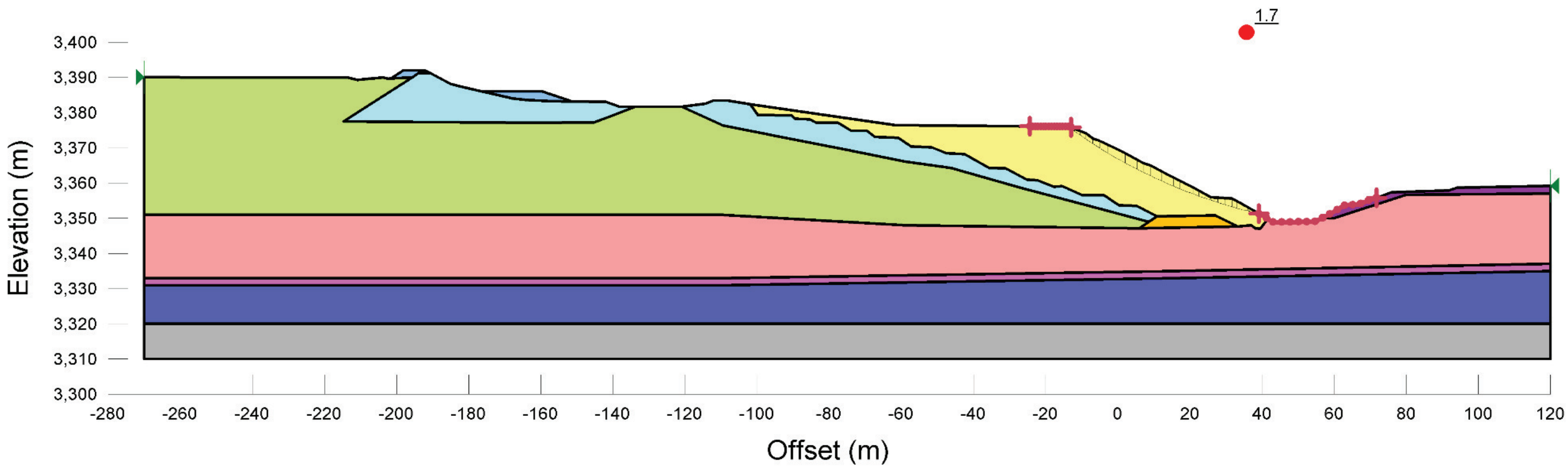
Name: APA Plane 5 - Static - Mode B
Date: 12/08/2016



Material Type	Unit Weight (kN/m^3)	Friction Angle (°)	Tau/Sigma
Seismic Berm Fill	22	38	--
Tailings - Static	22.5	--	0.2
Embankment Raises	22	34	--
Embankment Raises - Addition	22	34	--
Starter Embankment	22	36	--
Mine Road Fill	24	30	--
Densified Sand & Gravel	24	36	--
Unit 2a Glaciofluvial	24	34	--
Unit 2b Colluvium - Static	24	34	--
Unit 3 Transitional GF - Static	22	32	--
Unit 4 GLU - Static	20	--	0.22
Glacial Till	Impenetrable Bedrock		

APA Berm Limit Equilibrium Slope Stability Analysis

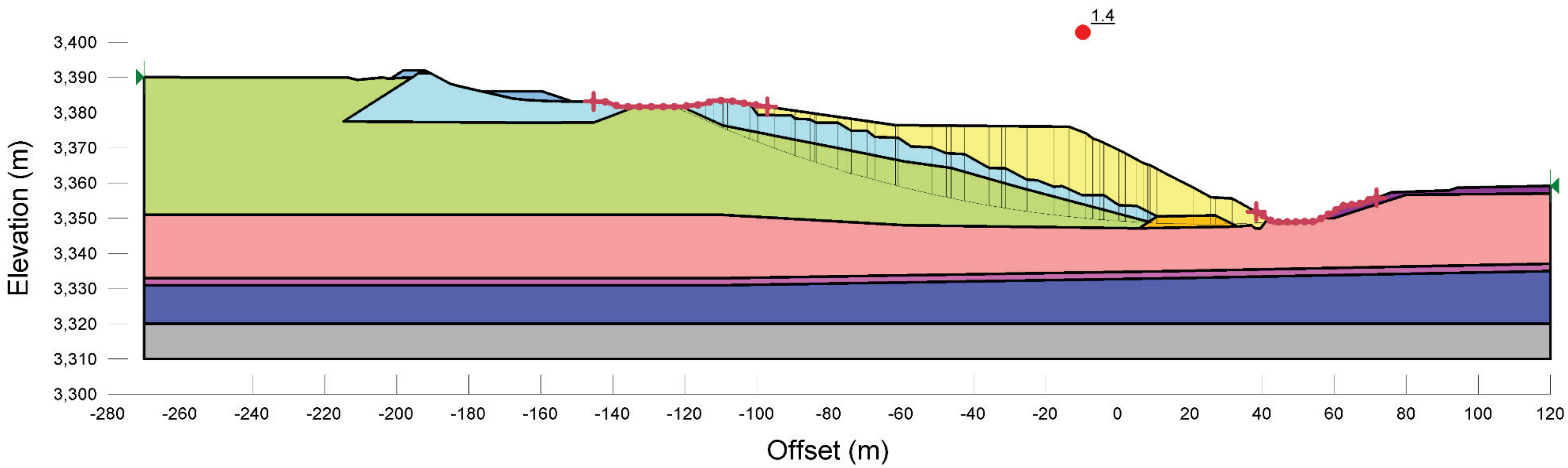
Name: APA Plane 5 - Static - Mode C
Date: 12/08/2016



	Material Type	Unit Weight (kN/m^3)	Friction Angle (°)	Tau/Sigma
	Seismic Berm Fill	22	38	--
	Tailings - Static	22.5	--	0.2
	Embankment Raises	22	34	--
	Embankment Raises - Addition	22	34	--
	Starter Embankment	22	36	--
	Mine Road Fill	24	30	--
	Densified Sand & Gravel	24	36	--
	Unit 2a Glaciofluvial	24	34	--
	Unit 2b Colluvium - Static	24	34	--
	Unit 3 Transitional GF - Static	22	32	--
	Unit 4 GLU - Static	20	--	0.22
	Glacial Till	Impenetrable Bedrock		

APA Berm Limit Equilibrium Slope Stability Analysis

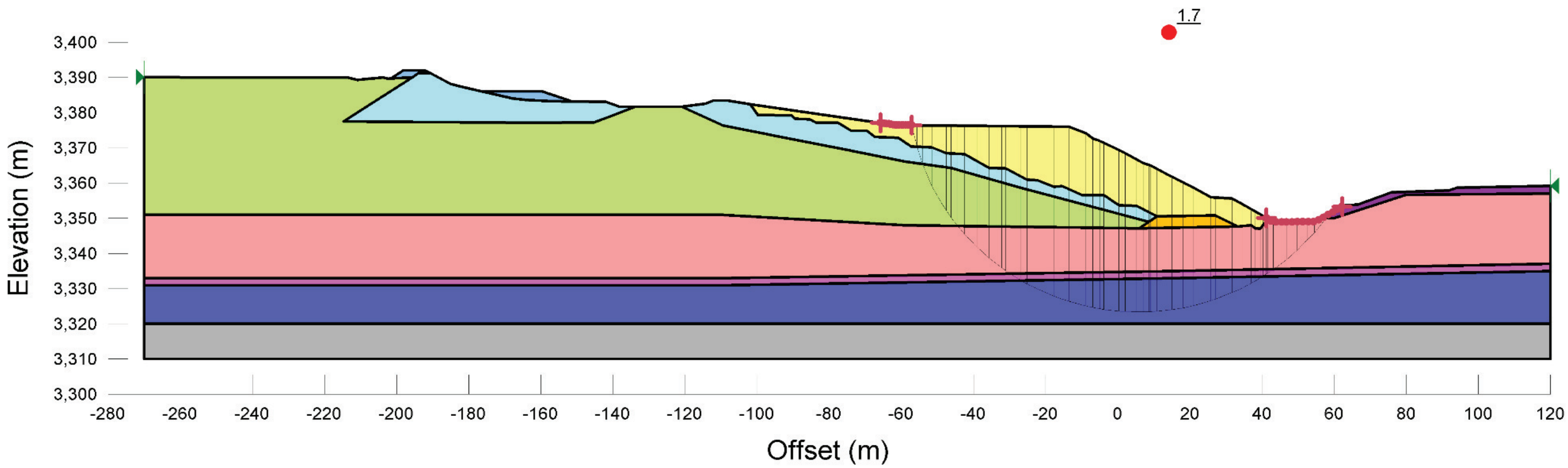
Name: APA Plane 5 - Static - Mode D
Date: 12/08/2016



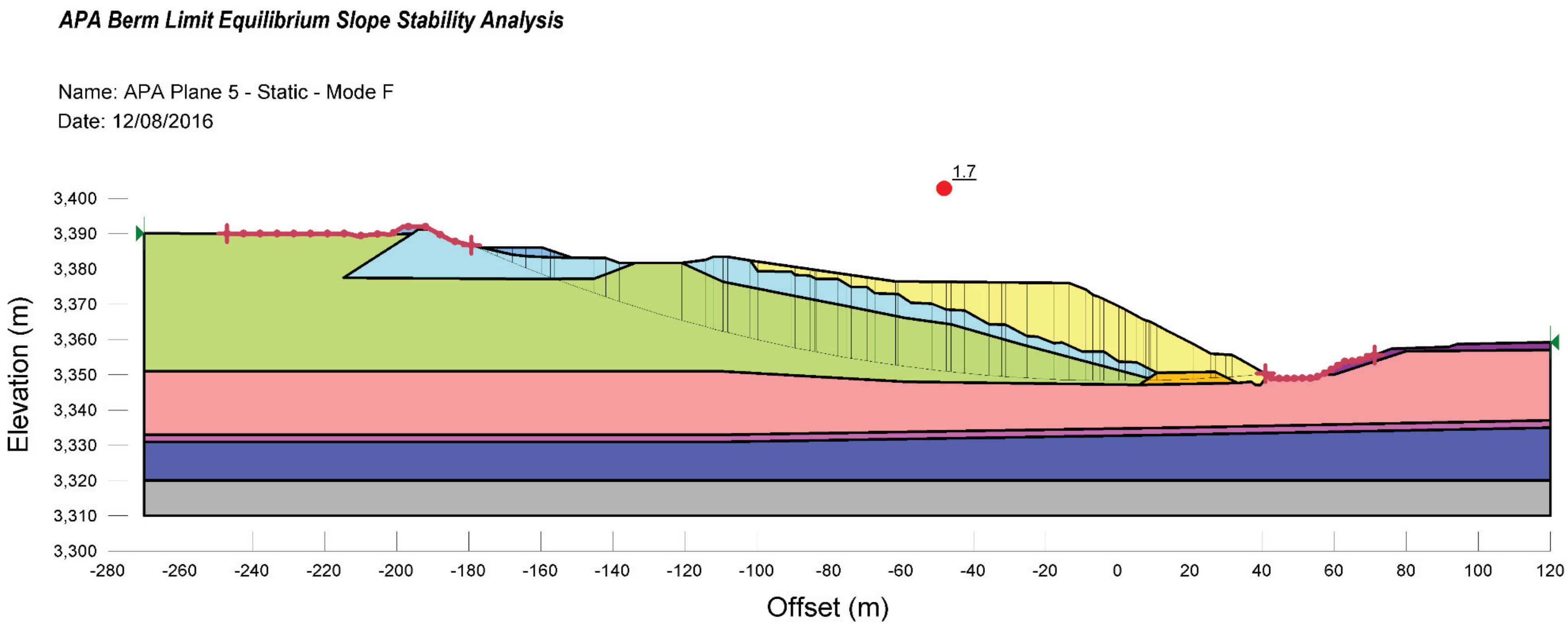
Material Type	Unit Weight (kN/m^3)	Friction Angle (°)	Tau/Sigma
Seismic Berm Fill	22	38	--
Tailings - Static	22.5	--	0.2
Embankment Raises	22	34	--
Embankment Raises - Addition	22	34	--
Starter Embankment	22	36	--
Mine Road Fill	24	30	--
Densified Sand & Gravel	24	36	--
Unit 2a Glaciofluvial	24	34	--
Unit 2b Colluvium - Static	24	34	--
Unit 3 Transitional GF - Static	22	32	--
Unit 4 GLU - Static	20	--	0.22
Glacial Till	Impenetrable Bedrock		

APA Berm Limit Equilibrium Slope Stability Analysis

Name: APA Plane 5 - Static - Mode E
Date: 12/08/2016



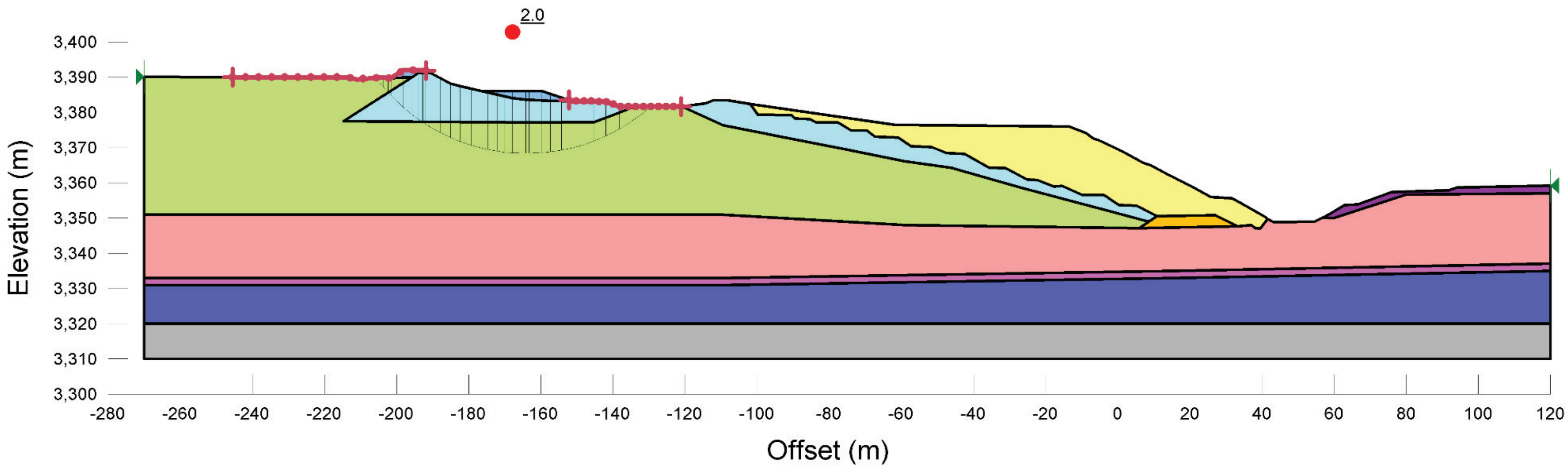
Material Type	Unit Weight (kN/m³)	Friction Angle (°)	Tau/Sigma
Seismic Berm Fill	22	38	--
Tailings - Static	22.5	--	0.2
Embankment Raises	22	34	--
Embankment Raises - Addition	22	34	--
Starter Embankment	22	36	--
Mine Road Fill	24	30	--
Densified Sand & Gravel	24	36	--
Unit 2a Glaciofluvial	24	34	--
Unit 2b Colluvium - Static	24	34	--
Unit 3 Transitional GF - Static	22	32	--
Unit 4 GLU - Static	20	--	0.22
Glacial Till	Impenetrable Bedrock		



Material Type	Unit Weight (kN/m^3)	Friction Angle (°)	Tau/Sigma
Seismic Berm Fill	22	38	--
Tailings - Static	22.5	--	0.2
Embankment Raises	22	34	--
Embankment Raises - Addition	22	34	--
Starter Embankment	22	36	--
Mine Road Fill	24	30	--
Densified Sand & Gravel	24	36	--
Unit 2a Glaciofluvial	24	34	--
Unit 2b Colluvium - Static	24	34	--
Unit 3 Transitional GF - Static	22	32	--
Unit 4 GLU - Static	20	--	0.22
Glacial Till	Impenetrable Bedrock		

APA Berm Limit Equilibrium Slope Stability Analysis

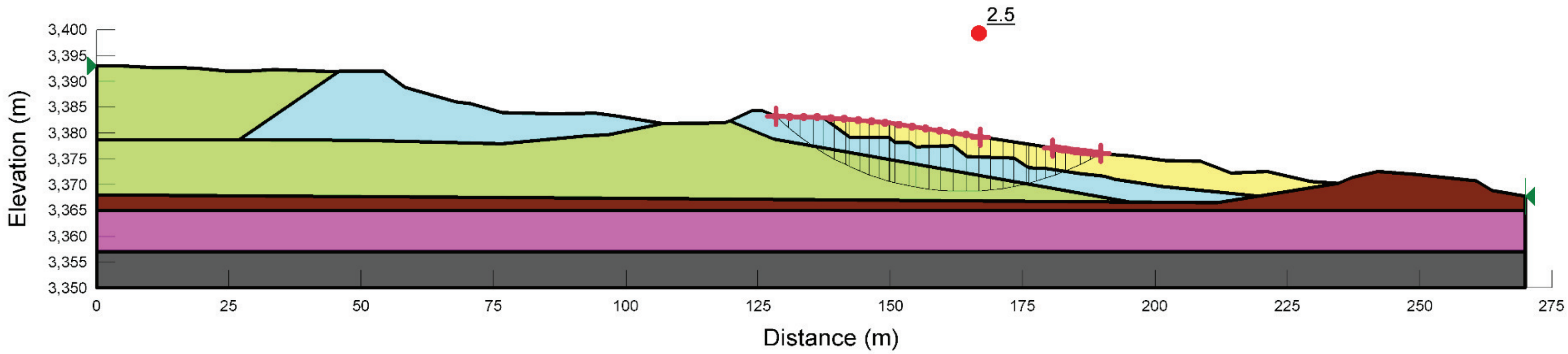
Name: APA Plane 5 - Static - Mode G
Date: 12/08/2016



Material Type	Unit Weight (kN/m^3)	Friction Angle (°)	Tau/Sigma
Seismic Berm Fill	22	38	--
Tailings - Static	22.5	--	0.2
Embankment Raises	22	34	--
Embankment Raises - Addition	22	34	--
Starter Embankment	22	36	--
Mine Road Fill	24	30	--
Densified Sand & Gravel	24	36	--
Unit 2a Glaciofluvial	24	34	--
Unit 2b Colluvium - Static	24	34	--
Unit 3 Transitional GF - Static	22	32	--
Unit 4 GLU - Static	20	--	0.22
Glacial Till	Impenetrable Bedrock		

APA Berm Limit Equilibrium Slope Stability Analysis

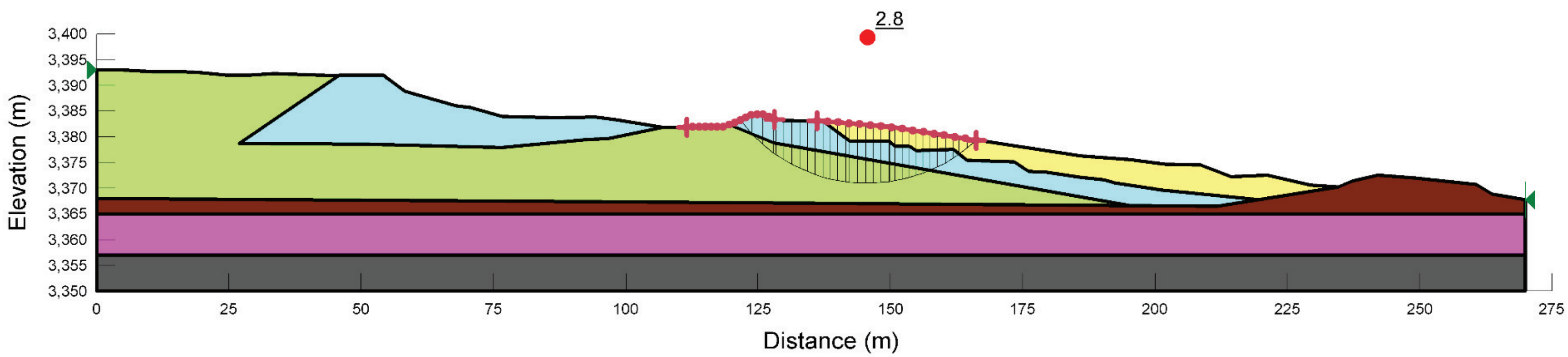
Name: APA Plane 6 - Post-Seismic Case - Mode A
Date: 17/08/2016



	Material Type	Unit Weight (kN/m^3)	Friction Angle (°)	Tau/Sigma
	Seismic Berm Fill	22	38	--
	Tailings - Post-Seismic	22.5	--	0.14
	Embankment Raises	22	34	--
	Embankment Raises - Addition	22	34	--
	Starter Embankment	22	36	--
	Mine Road Fill	24	30	--
	Densified Sand & Gravel	24	36	--
	Unit 2a Glaciofluvial	24	34	--
	Unit 2b Colluvium - Post-Seismic	24	--	0.28
	Unit 3 Transitional GF - Post-Seismic	22	--	0.35
	Unit 4 GLU - Post-Seismic	20	--	0.13
	Glacial Till	Impenetrable Bedrock		

APA Berm Limit Equilibrium Slope Stability Analysis

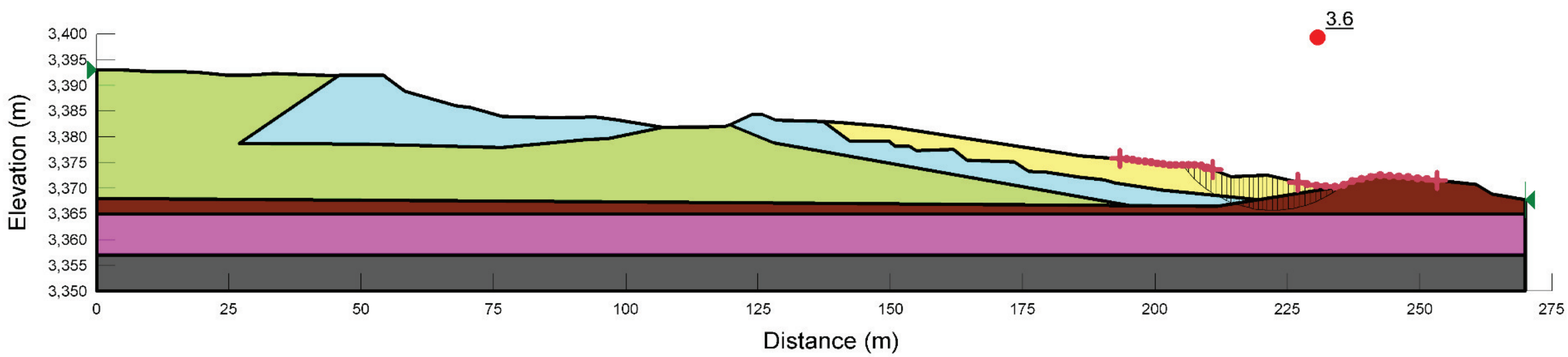
Name: APA Plane 6 - Post-Seismic Case - Mode B
Date: 17/08/2016



	Material Type	Unit Weight (kN/m³)	Friction Angle (°)	Tau/Sigma
	Seismic Berm Fill	22	38	--
	Tailings - Post-Seismic	22.5	--	0.14
	Embankment Raises	22	34	--
	Embankment Raises - Addition	22	34	--
	Starter Embankment	22	36	--
	Mine Road Fill	24	30	--
	Densified Sand & Gravel	24	36	--
	Unit 2a Glaciofluvial	24	34	--
	Unit 2b Colluvium - Post-Seismic	24	--	0.28
	Unit 3 Transitional GF - Post-Seismic	22	--	0.35
	Unit 4 GLU - Post-Seismic	20	--	0.13
	Glacial Till	Impenetrable Bedrock		

APA Berm Limit Equilibrium Slope Stability Analysis

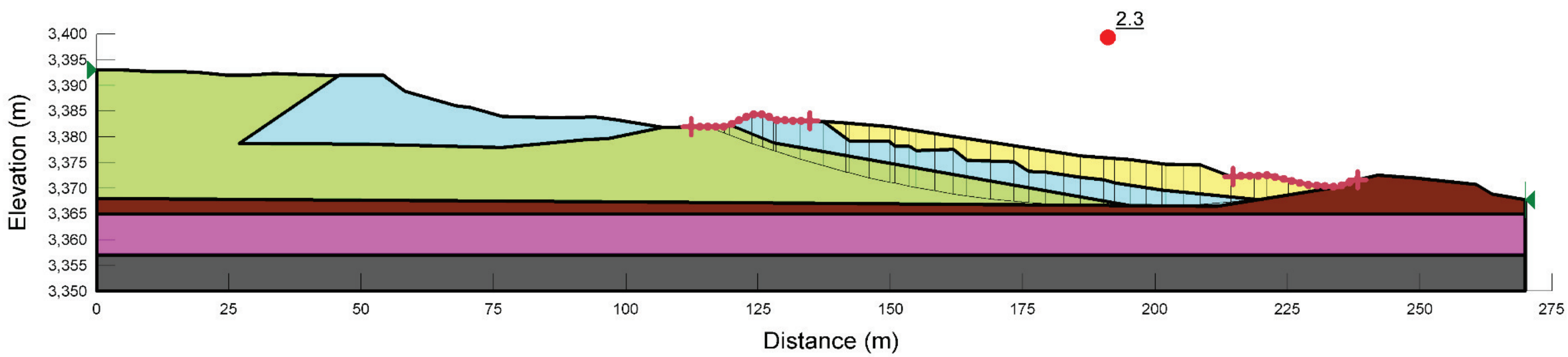
Name: APA Plane 6 - Post-Seismic Case - Mode C
Date: 17/08/2016



Material Type	Unit Weight (kN/m^3)	Friction Angle (°)	Tau/Sigma
Seismic Berm Fill	22	38	--
Tailings - Post-Seismic	22.5	--	0.14
Embankment Raises	22	34	--
Embankment Raises - Addition	22	34	--
Starter Embankment	22	36	--
Mine Road Fill	24	30	--
Densified Sand & Gravel	24	36	--
Unit 2a Glaciofluvial	24	34	--
Unit 2b Colluvium - Post-Seismic	24	--	0.28
Unit 3 Transitional GF - Post-Seismic	22	--	0.35
Unit 4 GLU - Post-Seismic	20	--	0.13
Glacial Till	Impenetrable Bedrock		

APA Berm Limit Equilibrium Slope Stability Analysis

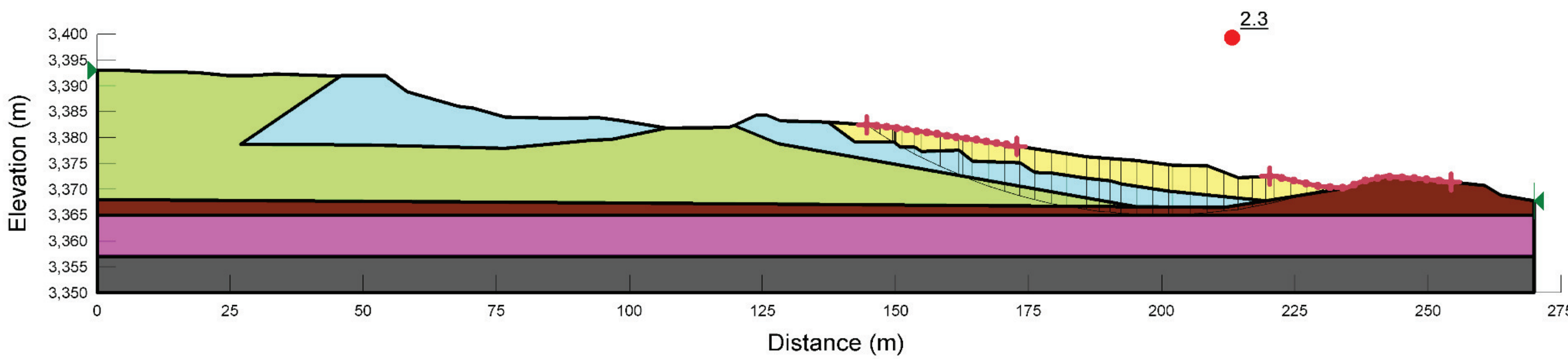
Name: APA Plane 6 - Post-Seismic Case - Mode D
Date: 17/08/2016



Material Type	Unit Weight (kN/m^3)	Friction Angle (°)	Tau/Sigma
Seismic Berm Fill	22	38	--
Tailings - Post-Seismic	22.5	--	0.14
Embankment Raises	22	34	--
Embankment Raises - Addition	22	34	--
Starter Embankment	22	36	--
Mine Road Fill	24	30	--
Densified Sand & Gravel	24	36	--
Unit 2a Glaciofluvial	24	34	--
Unit 2b Colluvium - Post-Seismic	24	--	0.28
Unit 3 Transitional GF - Post-Seismic	22	--	0.35
Unit 4 GLU - Post-Seismic	20	--	0.13
Glacial Till	Impenetrable Bedrock		

APA Berm Limit Equilibrium Slope Stability Analysis

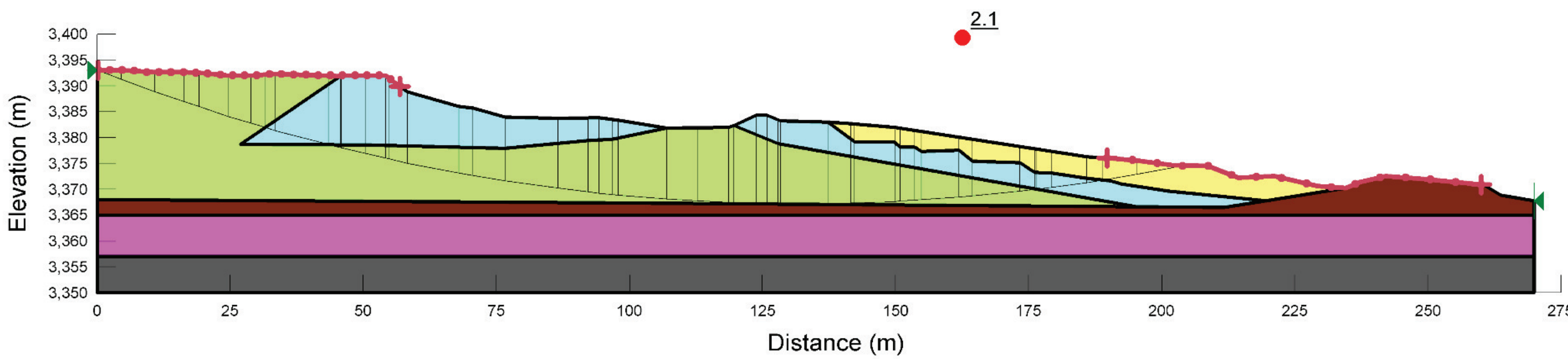
Name: APA Plane 6 - Post-Seismic Case - Mode E
Date: 17/08/2016



Material Type	Unit Weight (kN/m^3)	Friction Angle (°)	Tau/Sigma
Seismic Berm Fill	22	38	--
Tailings - Post-Seismic	22.5	--	0.14
Embankment Raises	22	34	--
Embankment Raises - Addition	22	34	--
Starter Embankment	22	36	--
Mine Road Fill	24	30	--
Densified Sand & Gravel	24	36	--
Unit 2a Glaciofluvial	24	34	--
Unit 2b Colluvium - Post-Seismic	24	--	0.28
Unit 3 Transitional GF - Post-Seismic	22	--	0.35
Unit 4 GLU - Post-Seismic	20	--	0.13
Glacial Till	Impenetrable Bedrock		

APA Berm Limit Equilibrium Slope Stability Analysis

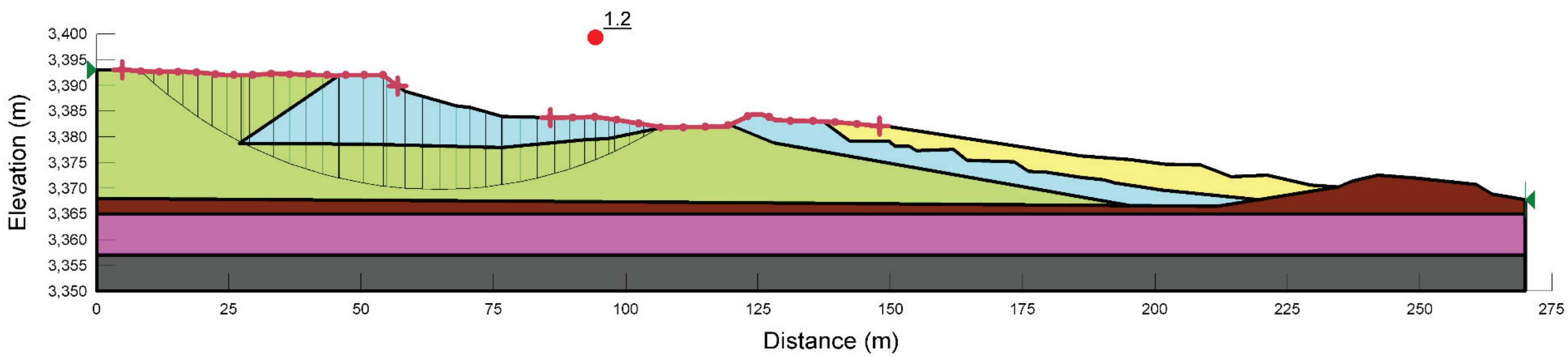
Name: APA Plane 6 - Post-Seismic Case - Mode F
Date: 17/08/2016



Material Type	Unit Weight (kN/m^3)	Friction Angle (°)	Tau/Sigma
Seismic Berm Fill	22	38	--
Tailings - Post-Seismic	22.5	--	0.14
Embankment Raises	22	34	--
Embankment Raises - Addition	22	34	--
Starter Embankment	22	36	--
Mine Road Fill	24	30	--
Densified Sand & Gravel	24	36	--
Unit 2a Glaciofluvial	24	34	--
Unit 2b Colluvium - Post-Seismic	24	--	0.28
Unit 3 Transitional GF - Post-Seismic	22	--	0.35
Unit 4 GLU - Post-Seismic	20	--	0.13
Glacial Till	Impenetrable Bedrock		

APA Berm Limit Equilibrium Slope Stability Analysis

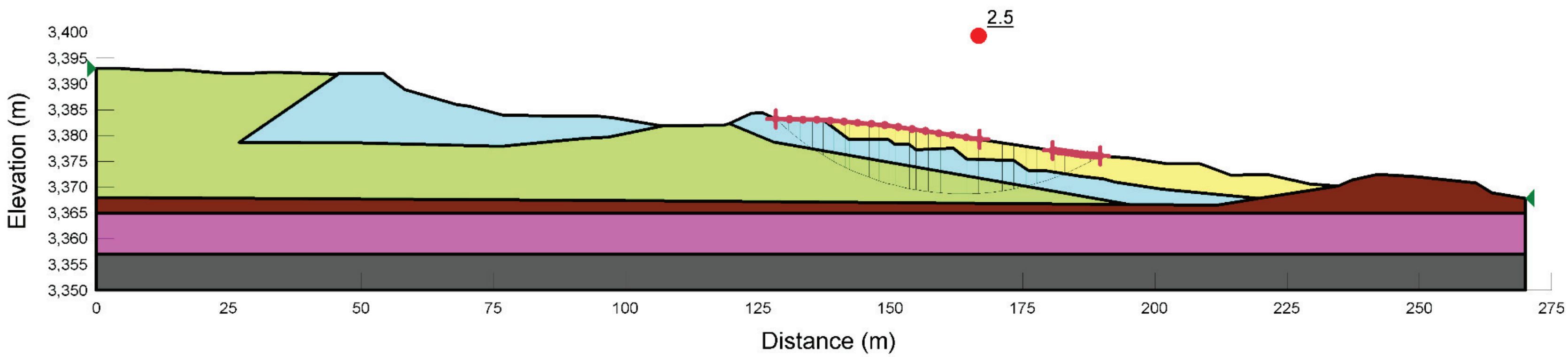
Name: APA Plane 6 - Post-Seismic Case - Mode G
Date: 17/08/2016



Material Type	Unit Weight (kN/m^3)	Friction Angle (°)	Tau/Sigma
Seismic Berm Fill	22	38	--
Tailings - Post-Seismic	22.5	--	0.14
Embankment Raises	22	34	--
Embankment Raises - Addition	22	34	--
Starter Embankment	22	36	--
Mine Road Fill	24	30	--
Densified Sand & Gravel	24	36	--
Unit 2a Glaciofluvial	24	34	--
Unit 2b Colluvium - Post-Seismic	24	--	0.28
Unit 3 Transitional GF - Post-Seismic	22	--	0.35
Unit 4 GLU - Post-Seismic	20	--	0.13
Glacial Till	Impenetrable Bedrock		

APA Berm Limit Equilibrium Slope Stability Analysis

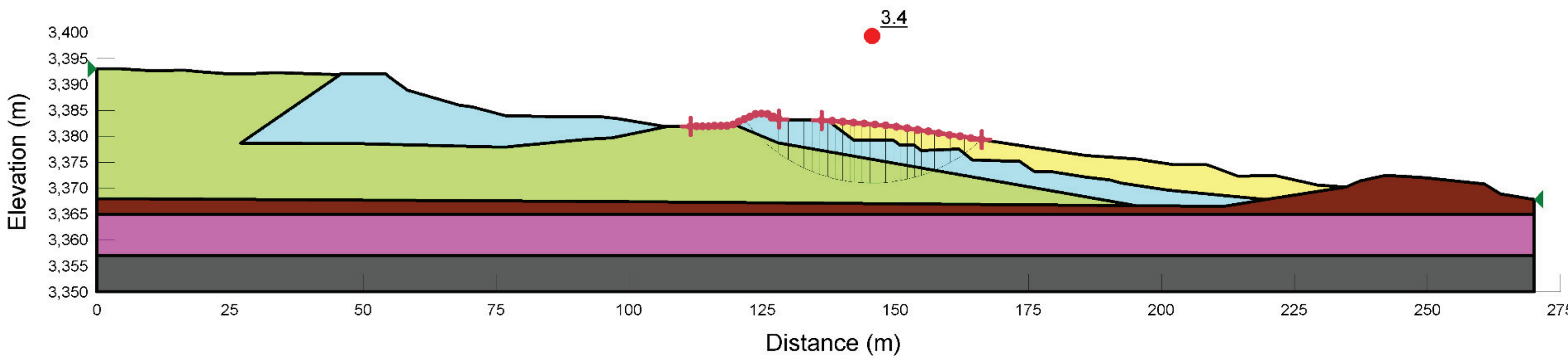
Name: APA Plane 6 - Static Case - Mode A
Date: 15/08/2016



	Material Type	Unit Weight (kN/m^3)	Friction Angle (°)	Tau/Sigma
	Seismic Berm Fill	22	38	--
	Tailings - Static	22.5	--	0.2
	Embankment Raises	22	34	--
	Embankment Raises - Addition	22	34	--
	Starter Embankment	22	36	--
	Mine Road Fill	24	30	--
	Densified Sand & Gravel	24	36	--
	Unit 2a Glaciofluvial	24	34	--
	Unit 2b Colluvium - Static	24	34	--
	Unit 3 Transitional GF - Static	22	32	--
	Unit 4 GLU - Static	20	--	0.22
	Glacial Till	Impenetrable Bedrock		

APA Berm Limit Equilibrium Slope Stability Analysis

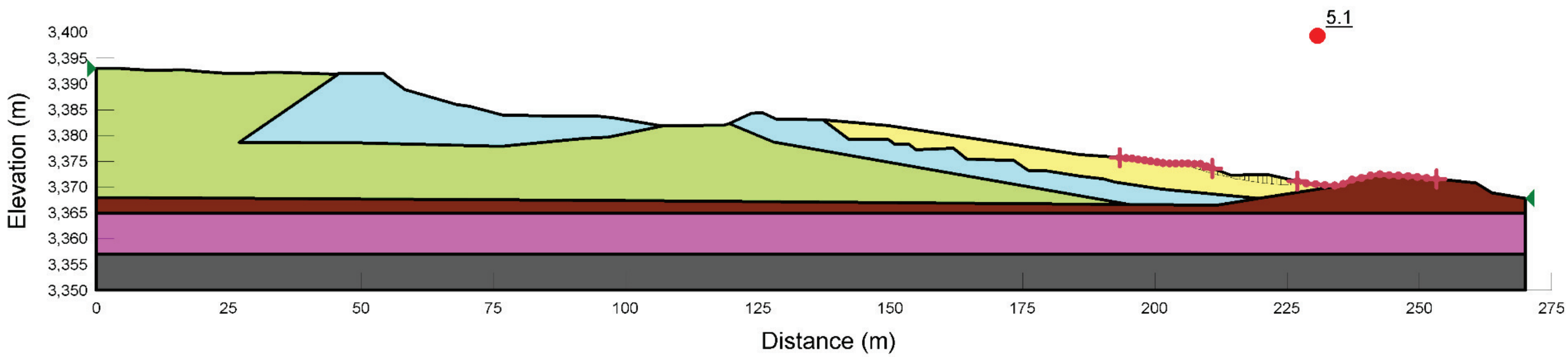
Name: APA Plane 6 - Static Case - Mode B
Date: 15/08/2016



	Material Type	Unit Weight (kN/m^3)	Friction Angle (°)	Tau/Sigma
	Seismic Berm Fill	22	38	--
	Tailings - Static	22.5	--	0.2
	Embankment Raises	22	34	--
	Embankment Raises - Addition	22	34	--
	Starter Embankment	22	36	--
	Mine Road Fill	24	30	--
	Densified Sand & Gravel	24	36	--
	Unit 2a Glaciofluvial	24	34	--
	Unit 2b Colluvium - Static	24	34	--
	Unit 3 Transitional GF - Static	22	32	--
	Unit 4 GLU - Static	20	--	0.22
	Glacial Till	Impenetrable Bedrock		

APA Berm Limit Equilibrium Slope Stability Analysis

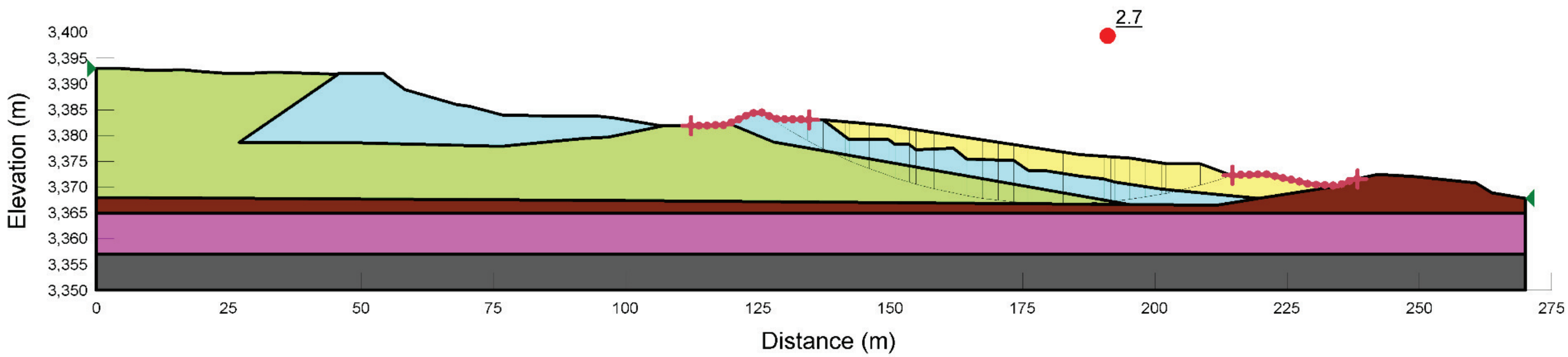
Name: APA Plane 6 - Static Case - Mode C
Date: 15/08/2016



Material Type	Unit Weight (kN/m^3)	Friction Angle (°)	Tau/Sigma
Seismic Berm Fill	22	38	--
Tailings - Static	22.5	--	0.2
Embankment Raises	22	34	--
Embankment Raises - Addition	22	34	--
Starter Embankment	22	36	--
Mine Road Fill	24	30	--
Densified Sand & Gravel	24	36	--
Unit 2a Glaciofluvial	24	34	--
Unit 2b Colluvium - Static	24	34	--
Unit 3 Transitional GF - Static	22	32	--
Unit 4 GLU - Static	20	--	0.22
Glacial Till	Impenetrable Bedrock		

APA Berm Limit Equilibrium Slope Stability Analysis

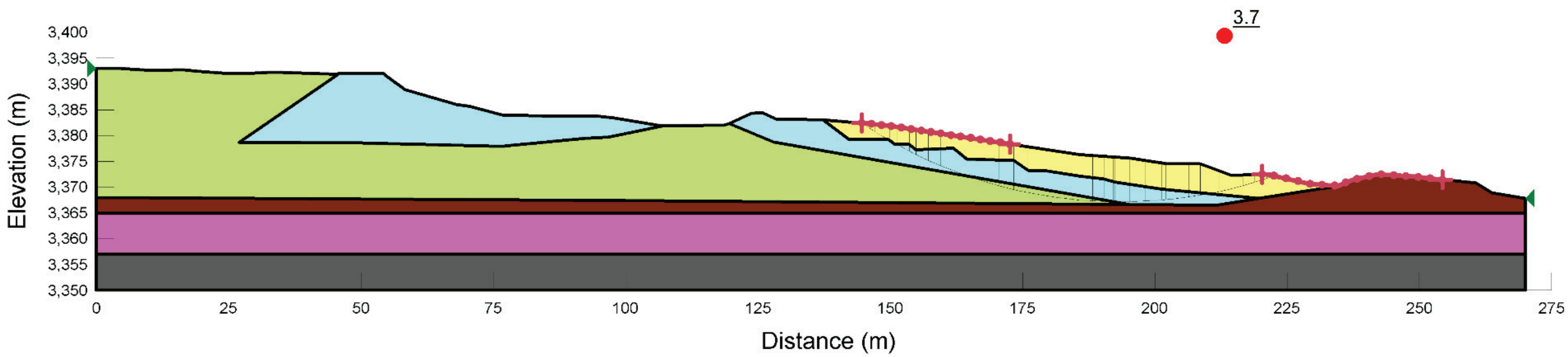
Name: APA Plane 6 - Static Case - Mode D
Date: 15/08/2016



	Material Type	Unit Weight (kN/m^3)	Friction Angle (°)	Tau/Sigma
	Seismic Berm Fill	22	38	--
	Tailings - Static	22.5	--	0.2
	Embankment Raises	22	34	--
	Embankment Raises - Addition	22	34	--
	Starter Embankment	22	36	--
	Mine Road Fill	24	30	--
	Densified Sand & Gravel	24	36	--
	Unit 2a Glaciofluvial	24	34	--
	Unit 2b Colluvium - Static	24	34	--
	Unit 3 Transitional GF - Static	22	32	--
	Unit 4 GLU - Static	20	--	0.22
	Glacial Till	Impenetrable Bedrock		

APA Berm Limit Equilibrium Slope Stability Analysis

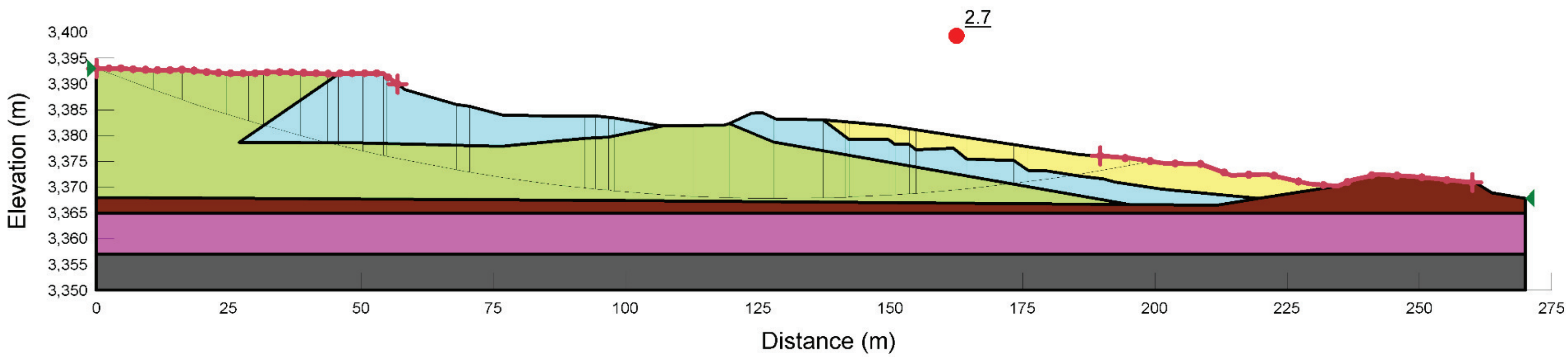
Name: APA Plane 6 - Static Case - Mode E
Date: 15/08/2016



Material Type	Unit Weight (kN/m^3)	Friction Angle (°)	Tau/Sigma
Seismic Berm Fill	22	38	--
Tailings - Static	22.5	--	0.2
Embankment Raises	22	34	--
Embankment Raises - Addition	22	34	--
Starter Embankment	22	36	--
Mine Road Fill	24	30	--
Densified Sand & Gravel	24	36	--
Unit 2a Glaciofluvial	24	34	--
Unit 2b Colluvium - Static	24	34	--
Unit 3 Transitional GF - Static	22	32	--
Unit 4 GLU - Static	20	--	0.22
Glacial Till	Impenetrable Bedrock		

APA Berm Limit Equilibrium Slope Stability Analysis

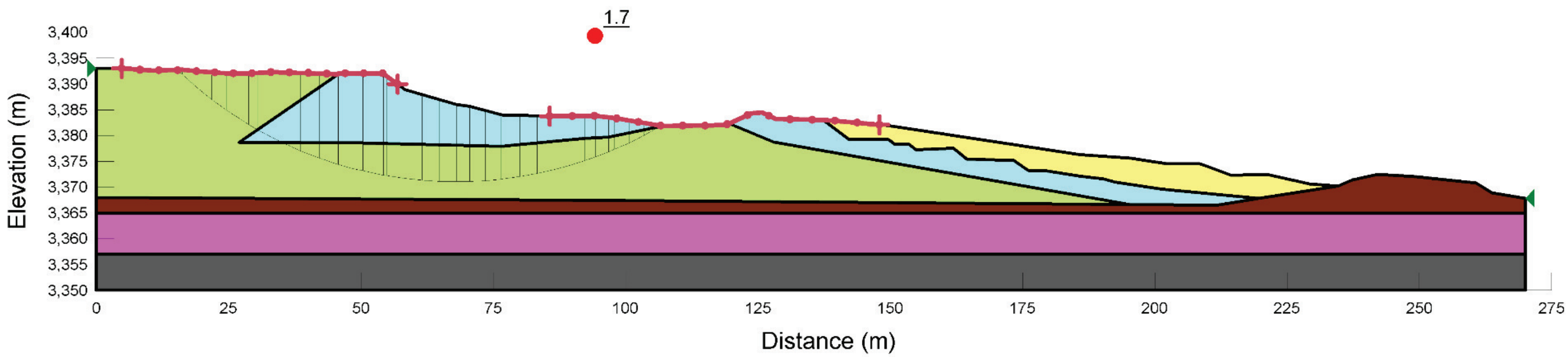
Name: APA Plane 6 - Static Case - Mode F
Date: 15/08/2016



	Material Type	Unit Weight (kN/m^3)	Friction Angle (°)	Tau/Sigma
	Seismic Berm Fill	22	38	--
	Tailings - Static	22.5	--	0.2
	Embankment Raises	22	34	--
	Embankment Raises - Addition	22	34	--
	Starter Embankment	22	36	--
	Mine Road Fill	24	30	--
	Densified Sand & Gravel	24	36	--
	Unit 2a Glaciofluvial	24	34	--
	Unit 2b Colluvium - Static	24	34	--
	Unit 3 Transitional GF - Static	22	32	--
	Unit 4 GLU - Static	20	--	0.22
	Glacial Till	Impenetrable Bedrock		

APA Berm Limit Equilibrium Slope Stability Analysis

Name: APA Plane 6 - Static Case - Mode G
Date: 15/08/2016



Material Type	Unit Weight (kN/m^3)	Friction Angle (°)	Tau/Sigma
Seismic Berm Fill	22	38	--
Tailings - Static	22.5	--	0.2
Embankment Raises	22	34	--
Embankment Raises - Addition	22	34	--
Starter Embankment	22	36	--
Mine Road Fill	24	30	--
Densified Sand & Gravel	24	36	--
Unit 2a Glaciofluvial	24	34	--
Unit 2b Colluvium - Static	24	34	--
Unit 3 Transitional GF - Static	22	32	--
Unit 4 GLU - Static	20	--	0.22
Glacial Till	Impenetrable Bedrock		

APPENDIX F

Dynamic Deformation Analyses
Detailed Technical Discussion

APPENDIX F
Dynamic Deformation Analyses
Detailed Technical Discussion

F1.0 INTRODUCTION

A two dimensional (2-D), nonlinear, dynamic finite element (FE) model using the proprietary software LSDYNA v.978 by Livermore Software Technology was developed to estimate potential static and seismic deformations of the Old TDF. The cross-section considered in the model was based on Plane 5. Plane 5 was chosen for more detailed dynamic analysis because the tailings and glaciolacustrine deposits are thickest at this location compared to other planes as well as the lowest static and post-seismic factors of safety which were for Mode D. The Plane 5 cross-section and soil stratigraphy is shown in Figure 4-5 in Section 4.0.

The limit equilibrium slope stability analyses assessed potential downstream modes of failure and used post seismic residual strengths in those materials considered susceptible to initial transient liquefaction under design levels of earthquake shaking. The following materials, presented in order of increasing depth, were considered to undergo transient pore pressure build-up and cyclic strain accumulation ("cyclic mobility") for purposes of the Plane 5 dynamic deformation modeling:

- Tailings consisting predominantly of silt to clayey silt tailings containing thin silt laminations. The tailings has a low to moderate plasticity index, typically in the range of 4 to 17 percent. There was no evidence of significant sand or silty sand layers within the tailings in the vicinity of the cross-section being modeled based on available electronic cone penetration test data.

- Looser interlayers of transitional glaciofluvial (Unit 3) layers.

- Glaciolacustrine deposits (GLU) consisting predominantly of clay/silt mixtures with thin laminations of fine sand and silty sand. The GLU has a plasticity index typically in the range of 0 to 16 percent.

The glaciofluvial deposits (Unit 2a) consisting of sand and gravel with cobbles and boulders were assumed to have a compact to very dense state of compaction although locally looser layers were also present. Review of available Becker Density Test data and equivalent Standard Penetration Test $N_{1,60}$ discussed in Appendix C indicates mean and 20th percentile $N_{1,60}$ values of 51 and 29, respectively. Based on correlations between cyclic liquefaction resistance, stress level and $N_{1,60}$ values, the factor of safety against initial liquefaction triggering (FSL) in these materials using the 20th percentile $N_{1,60}$ value is about 1.3 to 1.4 at the stress levels being considered in the soil profile (Idriss and Boulanger, 2008, 2014). On this basis, correlations between cyclic pore pressure generation and FSL indicate excess pore pressure ratios (excess pore pressures developed at the end of shaking divided by the pre-earthquake mean effective confining stress) would be less than about 0.20 (Tokimatsu and Seed, 1987). Correlations using mean $N_{1,60}$ values suggest that excess pore pressure ratios are essentially zero (Tokimatsu and Seed, 1987).

On this basis, it was considered reasonable to neglect cyclic pore pressure generation in the Unit 2a materials in the dynamic modeling.

Geotechnical material properties (unit weights, static strength, post seismic strength) of the various soil units used in limit equilibrium analyses summarized in Table 4-1 in Section 4.0. Detailed discussions of the geotechnical material properties are provided in Appendices C and D and are based on review of available site investigation and laboratory test data. Additional properties used in dynamic deformation modeling are discussed in the Sections F2.0 and F3.0

F2.0 ONE DIMENSIONAL SEISMIC WAVE PROPAGATION ANALYSIS

One dimensional (1-D), seismic wave propagation analysis was completed to provide initial calculations of cyclic shear stress levels developed within the Plane 5 soil profile under design levels of earthquake shaking. The peak cyclic shear stress levels, normalized with respect to effective vertical overburden stresses at the depth under consideration, are termed cyclic shear stress ratios (CSRs) and were used to estimate the potential for initial liquefaction triggering in the tailings, transitional glaciofluvial (Unit 3), and deeper GLU (Unit 4) deposits.

1-D analysis only models vertical propagation of earthquake-induced shear waves through a soil column. The computer program DESRA-2C and a later version of the program referred to as DESRAMOD (Lee and Finn, 1978) were used for this purpose. The DESRAMOD output was compared against predictions made using a I-D model developed using the LSDYNA, in order to perform initial calibrations and checks required for the 2-D LSDYNA modeling.

Two 1-D soil columns were considered at the locations shown in Figure 4-5. These soil columns are referred to as Soil Profile 1 and Soil Profile 2.

The program DESRAMOD models the nonlinear, shear stress – shear strain response of the various soil layers. Cyclic hysteretic response of each soil layer is simulated assuming Masing-type load-unload behavior. The program requires as input the peak shear strength (τ_{max}) and small strain shear stiffness (G_{max}) for each soil layer. The latter were derived from downhole borehole and seismic CPT measurements of shear wave velocity (V_s) carried out within the tailings, glaciofluvial sediments, and GLU (Amec Foster Wheeler, 2015a, 2016a, 2016b). G_{max} is related to V_s using isotropic elasticity theory given by the equation $G_{max} = (\gamma_t/g)\ V_s^2$ where γ_t is the total mass density of the soil at a particular depth and g is the gravitational constant.

A plot of overburden stress normalized shear wave velocity (V_{s1}) versus depth within the tailings, glaciofluvial and GLU deposits based on downhole measurements in boreholes and CPT's located in close proximity to Plane 5 is shown in Figure F-1. Here V_{s1} is defined as $V_{s1} = V_s\ (P_{atm}/\sigma'_{v0})0.25$ where P_{atm} is atmospheric pressure and σ'_{v0} is the vertical effective overburden stress at the depth under consideration.

Figure F-1: Overburden Stress Normalized V_{s1} with Depth

The dynamic material properties used in one and two dimensional, total stress site response analyses (DESRAMOD and LSDYNA) for Soil Profile 1 and Soil Profile 2 are provided in Table F-1 and Table F-2, respectively.

Shear strengths for the upper layers of embankment fill, tailings, glaciofluvial deposits, and lower GLU deposits were calculated using Mohr-Coulomb strength theory using the following equation:

$$\tau_{max} = c' \cos \varphi' + 0.5\ (\sigma'_{vo} + \sigma'_{ho}) \sin \varphi'$$

where $\sigma'_{ho} = K_0\ \sigma'_{vo}$ and K_0 is the coefficient of earth pressure at rest. Effective stresses σ'_{vo} and σ'_{ho} are assumed to be major principle stresses in the ground. It is seen that shear strength is dependent on the Mohr-Coulomb effective cohesion c' and effective friction angle φ', vertical effective stress σ'_{vo} and the coefficient of earth pressure at rest K_0. Drained shear strength properties were assumed in the embankment fill, Unit 2a glaciofluvial and Unit 3 transitional glaciofluvial deposits based on a peak φ' value. Undrained shear strengths were assumed during dynamic shaking in the tailings and GLU based on undrained strength ratios (S_u/σ'_{vo}) of 0.2 and 0.22, respectively. Dynamic rate effects on undrained strengths were neglected which was considered reasonable based on the relatively low plasticity indices of these materials. It is noted that LSDYNA calculates shear strengths based on application of Mohr-Coulomb strength criterion. Therefore the peak shear strength will depend not only on c', φ', and σ'_{vo} but also on K_0. The latter was selected equal to 0.75 and 0.60 for the tailings and GLU soil layers based on examination of FE model stress output in the LSDYNA model. An equivalent φ' with zero c' was then selected for the tailings and GLU to match the target S_u/σ'_{vo} ratio.

The following initial pore pressure (U_0) conditions were assumed in order to calculate pre-earthquake effective stresses within those materials assumed to be saturated:

- Pore pressures in the native foundation soils (glaciofluvial, GLU and glacial till soils) calculated using a hydrostatic pore pressure distribution with zero pore pressure at the top of the foundation soils (base of tailings).

- Pore pressures within the tailings assumed to be zero at the top and bottom of the tailings, increasing to a maximum value of 80 kPa at the mid-depth of the deposit.

The underlying dense glacial till was assumed to be strongly dilative and respond in an undrained manner during dynamic shaking. Undrained shear strengths were calculated assuming pore water cavitation to -1 atmosphere. Shear strengths (τ_{max}) were calculated using the following equation for cohesionless soils (Seed and Lee, 1967):

$$\tau_{max} = \left(\sigma_{3,init}' + U_0 + P_{atm}\right)\left[\frac{\sin\varphi_{cv}}{1-\sin\varphi_{cv}}\right]$$

where,

$\sigma_{3,init}'$	=	initial minor principal effective stress at the depth under consideration = $K_0\,\sigma'_{vo}$;
U_0	=	static water pressure at the depth under consideration;
P_{atm}	=	atmospheric pressure (= 101.3 kPa);
φ_{cv}	=	constant volume friction angle (assumed equal to 35°) deg

Excess pore water pressures generated by seismic shaking were neglected in both the DESRAMOD and LSDYNA analyses. This is referred to as a "total stress" dynamic analysis and would be expected to lead to upper bound predictions of cyclic shear stresses within those soil materials considered to be subject to transient liquefaction (e.g. tailings, transitional glaciofluvial, and GLU deposits present within the Plane 5 cross-section). Cyclic strain development would also be correspondingly reduced using this assumption.

Table F-1: Dynamic Material Properties – Soil Profile 1

Model Layer No.	Soil Type	Layer Thickness (m)	Total Unit Weight, γ_t (kN/m^3)	Stress Normalized Shear Wave Velocity, V_{s1} (m/sec)	Shear Wave Velocity, V_s (m/sec)	K_0
1	Embankment Raises	3.5	22	261	195	0.44
2 - 8	Tailings	28	22.5	195	202-309	0.75
9 - 13	Dense Glaciofluvial Sand and Gravel with loose layers (Unit 2a)	18	24	210	344-364	0.44
14	Transitional Silty Sand (Unit 3)	2	22	203	358	0.47
15 - 17	Glaciolacustrine Silt and Clay (Unit 4)	21	20	175	311-320	0.6
18 - 19	Glacial Till	10	24	240	450	0.75

Table F-2: Dynamic Material Properties – Soil Profile 2

Model Layer No.	Soil Type	Layer Thickness (m)	Total Unit Weight, γ_t (kN/m^3)	Stress Normalized Shear Wave Velocity, V_{s1} (m/sec)	Shear Wave Velocity, V_s (m/sec)	K_0
1 - 3	Seismic Berm	12	22	314	234-367	0.38
4	Embankment Raises	3	22	261	346	0.44
5 - 7	Tailings	14	22.5	195	268-299	0.75
8 - 12	Dense Glaciofluvial Sand and Gravel with loose layers (Unit 2a)	14	24	210	338-358	0.44
13	Transitional Silty Sand (Unit 3)	2	22	203	349	0.47
14 - 17	Glaciolacustrine Silt and Clay (Unit 4)	27.5	20	175	303-319	0.6
18 - 19	Glacial Till	10	24	240	450	0.75

A 1-D model for Soil Profile 1 was developed using LSDYNA using the same number of soil layers and dynamic soil properties as used in DESRAMOD. The model is shown in Figure F-2 and incorporates linear elastic elements along the lateral boundaries of the model and a layer of elastic elements for the bottom layer (Layer 19) of the model. These elastic lateral and bottom boundaries permit the transmission of seismic wave energy beyond the boundaries of the model and prevent seismic wave reflections back into the model. The elastic element moduli were calculated using small strain G_{max} values for a given layer depth and the following Poisson's ratios (μ): (a) μ= 0.49 in all native foundation materials and within the tailings assuming nearly complete saturation of these materials; (b) μ= 0.35 for all granular fill materials above the water table.

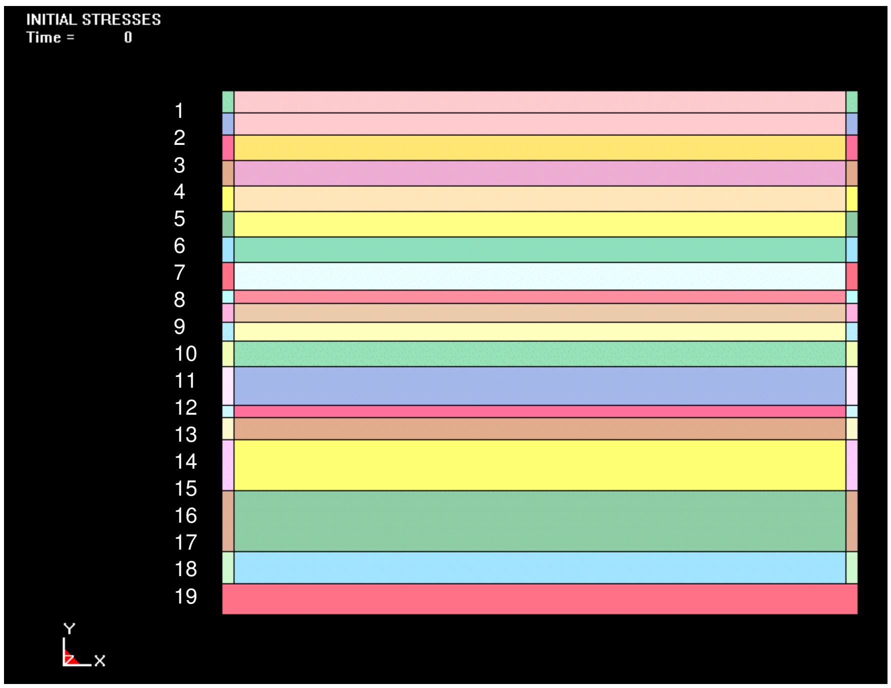


Figure F-2: One Dimensional LSDYNA Model Developed for Soil Profile 1 from the Plane 5 Cross-Section

Nonlinear soil elements are present within the interior of the 1-D LSDYNA model, i.e. those elements excluding the elastic elements along the boundary of the model. Descriptions of the nonlinear, cyclic stress-strain models used in LSDYNA are provided in Section F3.1. A nonlinear "geologic cap model" incorporating cyclic hysteresis was used in the 1-D modeling (Chen and Baladi, 1985: Weidlinger Associates,1978).

Stages of the LSDYNA modeling included:

- Stage 1 – Application of vertical gravity load with the base of the model constrained against vertical Y displacement and "at rest" lateral earth pressures applied to the lateral boundaries of the 1-D soil column. The "at rest" earth pressures were computed using the K_0 values given in Table 5.1.The lateral boundaries were free to settle in the Y direction during application of gravitational loading. Total unit weights were used in the embankment fill (assumed to have no perched water) and buoyant unit weights were used in foundation soil units below the base of the tailings. Within the tailings, effective unit weights were used to match vertical effective stresses calculated using an Excel spreadsheet based on a total unit weight of 22.5 kN/ m^3 (to calculate total vertical stresses) and then subtracting off the initial pore pressure U_0 distribution described earlier for the tailings.

- Stage 2 – The same as Stage 1 above except using total unit weights for all soil layers as a precursor to the start of lateral shaking of the model using an input base motion. Since change in unit weights of the soil layers results in a change in initial vertical effective stress under vertical gravity loading prior to the start of earthquake shaking, it was necessary to adjust the vertical gravitational constant to achieve similar vertical effective stresses at different depths in the model as calculated during Stage 1.

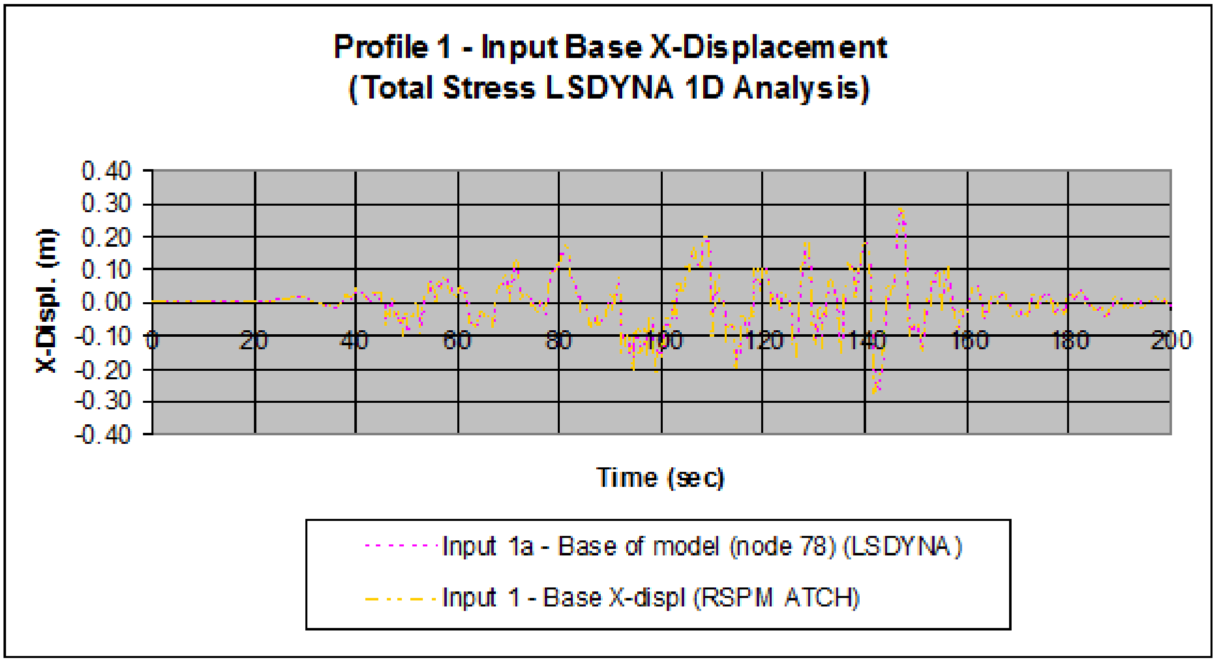
- Stage 3 – The dynamic phase of analysis incorporating an energy absorbing bottom (glacial till) boundary. Earthquake shear wave motion input was applied using dynamic shear stresses (lateral nodal forces) applied along the rock base. The dynamic shear stress time history (τ_{dyn}) was computed using the following equation (Lysmer and Kuhlemeyer, 1969)

$$\tau_{dyn} = (\gamma_t/g)\ V_s\ v_{oc}\ (t)$$

where γ_t is the total unit weight of the glacial till, Vs is the till shear wave velocity (= 450 m/sec) and $v_{oc}(t)$ is the horizontal velocity time history of the Site Class C (glacial till) outcrop motions.

The shear stresses were multiplied by the finite element width along the base of the model to give lateral nodal forces. Checks were made that the computed lateral X-displacement time histories at the base of the model matched closely the input base displacement computed after filtering and baseline correction using RSPMATCH09. An example of this comparison is shown in Figure F-3.

Boundary conditions during dynamic shaking included: (a) base of model constrained against vertical Y-displacement but free to move laterally in the X-direction, (b) lateral boundaries constrained against vertical Y movement and free to move only in the X-direction, (c) K_0 lateral boundary stresses were applied to the lateral boundaries of the model, (d) all interior nodes of the model were constrained to undergo only lateral X-displacement during shaking, (e) a bottom boundary was used permitting shear wave energy transmission. The above boundary conditions are identical to those used in the DESRAMOD model to permit direct comparison of output from the two models.



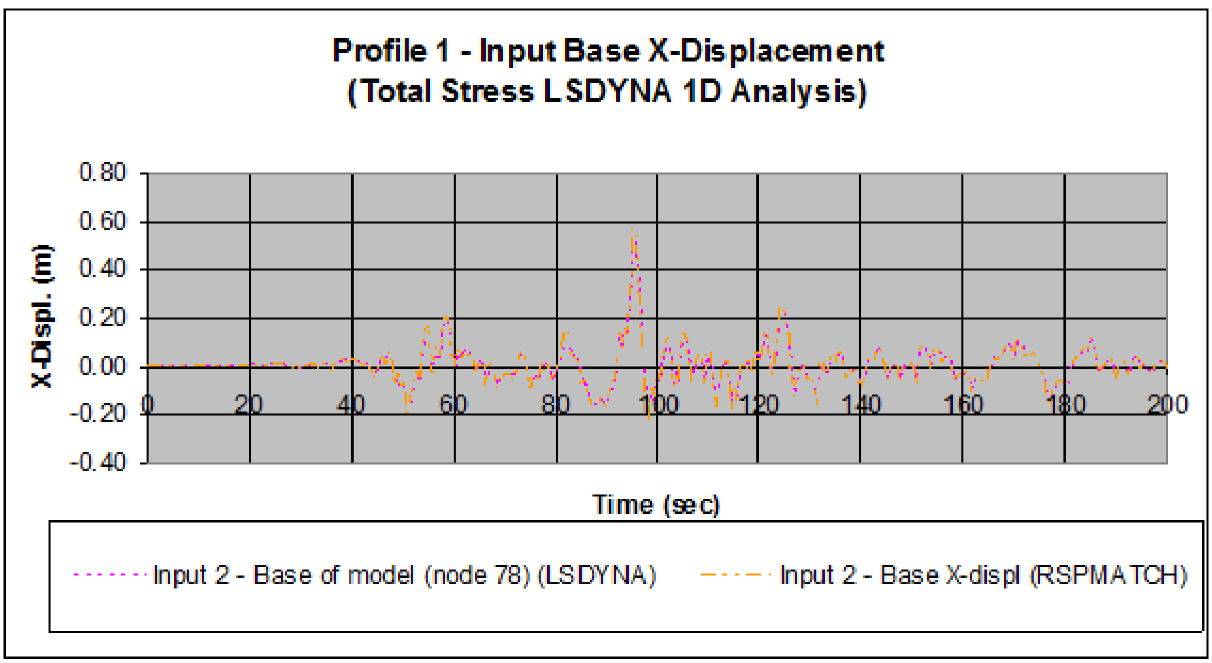


Figure F-3: Comparison of Computed Lateral X-Displacements Versus Time at the Base of the 1-D LSDYNA Model and Comparison of Input Base X-displacements from RSPMATCH09 Analysis Considering Input Earthquake Records 1 and 2

The following contributions to soil damping were considered in both the DESRAMOD and LSDYNA analyses:

a) internal hysteretic energy losses within the soil mass.

b) elastic wave energy transmitted below the bottom boundary of the soil layer model based on a shear wave velocity of 450 m/sec.

c) Stiffness proportional Rayleigh damping using a damping ratio of 2% was considered at a frequency of 5 Hz and higher damping levels for higher frequencies. This was found to attenuate higher frequency (short period) vibration energy.

Computed cyclic shear stress – shear strain response from the LSDYNA model for Soil Profile 1 considering Earthquake Record 1 input within an element of tailings is shown in Figure F-4. This illustrates the nonlinear cyclic stress-strain response and that considerable hysteretic damping is simulated within each soil element.

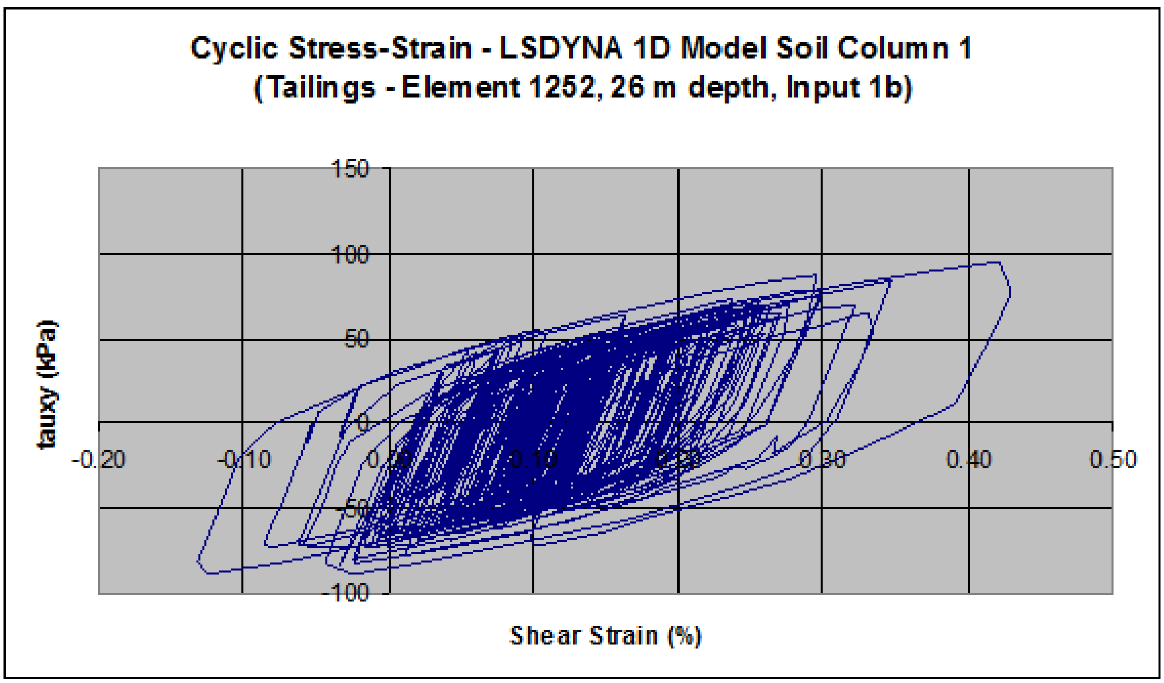


Figure F-4: Computed Cyclic Shear Stress – Shear Strain Response from the LSDYNA Model within an Element of Tailings for Soil Profile 1 Considering Earthquake Record 1 Input

Plots of peak CSR versus depth computed using DESRAMOD for the 4 input ground motions applied at the base of the 1-D model are shown in Figure F-5 for both Soil Profiles 1 and 2. There is a relatively tight band of computed peak CSR for all input ground motions with values in the range of 0.15 to 0.2. This indicates that peak cyclic shear stresses in the range of 70 to 100% of the peak undrained strengths of the tailings and GLU develop during shaking. Peak CSR's computed using the LSDYNA model for Soil Profile 1 are also compared with the DESRAMOD output in Figure F-5 considering earthquake records 1 and 2. This comparison indicates close agreement between the 2 models. Peak CSR output from LSDYNA was found to be sensitive to the equivalent friction angle and K_0 value used in the tailings and GLU in order to have close agreement between shear strengths used in the 2 models.

Peak shear strains versus depth are shown in Figure F-6 for both Soil Profiles 1 and 2 from the DESRAMOD output and for Soil Profile 1 from the LSDYNA output. Both models indicate similar trends of shear strain versus depth. Largest shear strains (in the range of 0.5 to 1.5%) are computed in the tailings and GLU, suggesting that largest contributions to downslope lateral movements of the Old TDF will occur due to cyclic straining in these materials. Smaller strains and lateral movements are computed within the higher strength glaciofluvial materials.

A comparison of lateral displacement versus time relative to the input base displacement Soil Profile 1 and considering input record (1) between the DESRAMOD and LSDYNA models is shown in Figure F-7. The calculated relative lateral X-displacement results from integration of the shear strain time histories versus depth computed from both models. Agreement is excellent between the two models in terms of peak displacement amplitudes, in frequency content of the displacement time history, and in residual displacements at the end of shaking.

The above comparisons indicate satisfactory agreement between the two 1-D models.

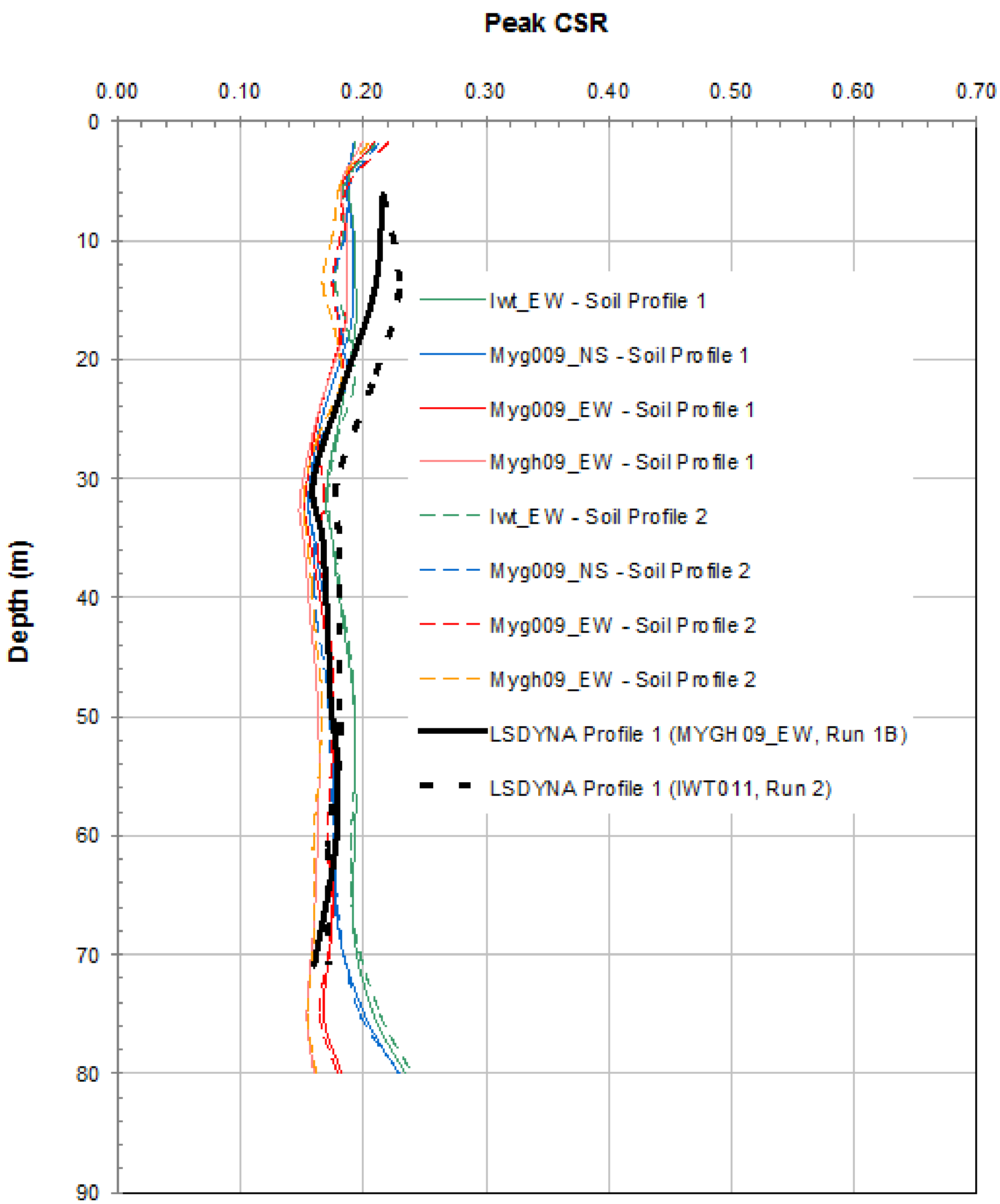


Figure F-5: Comparison of Computed Peak CSR's Versus Depth from 1-D DESRAMOD and LSDYNA Models

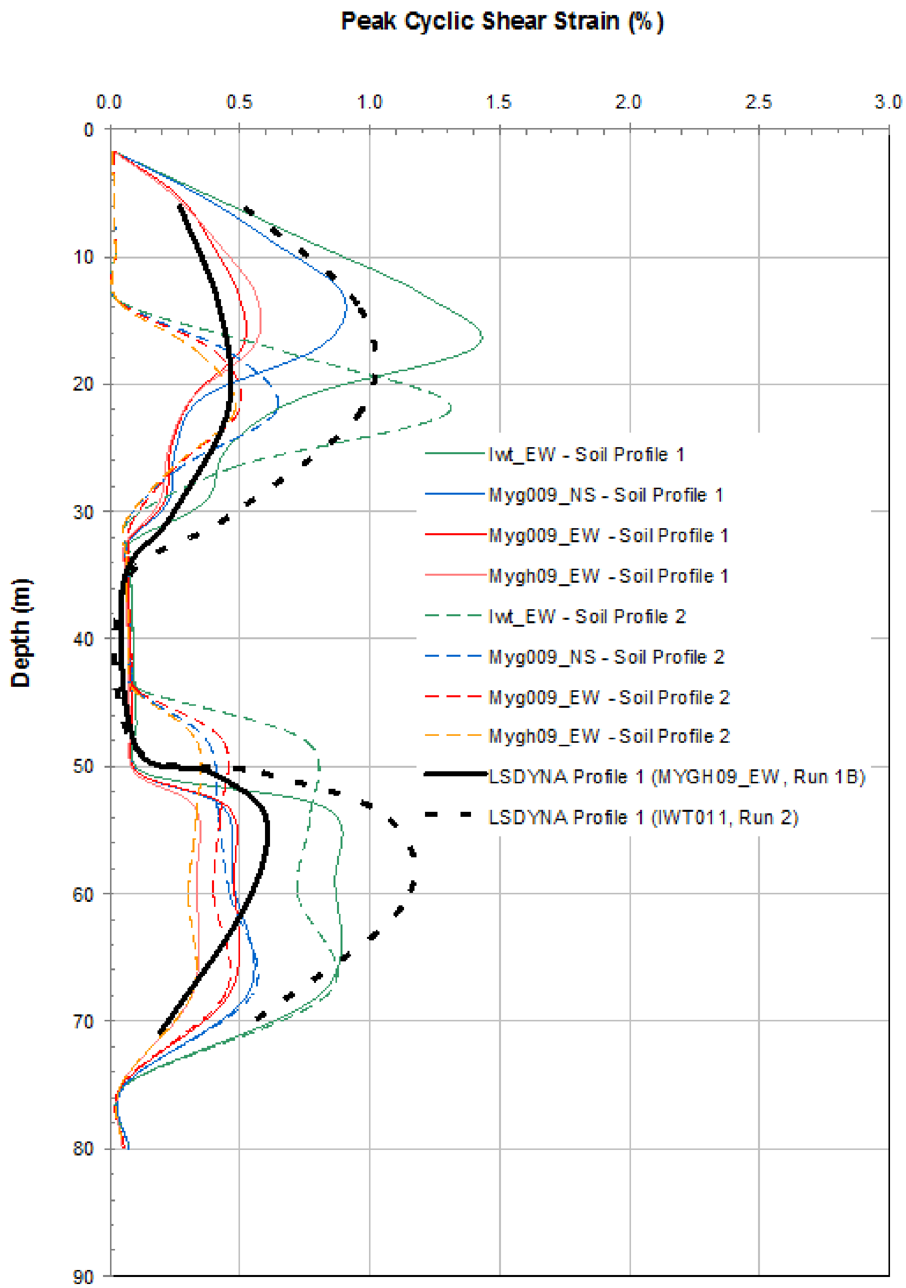


Figure F-6: Comparison of Computed Peak Shear Strains Versus Depth from 1-D DESRAMOD and LSDYNA Models

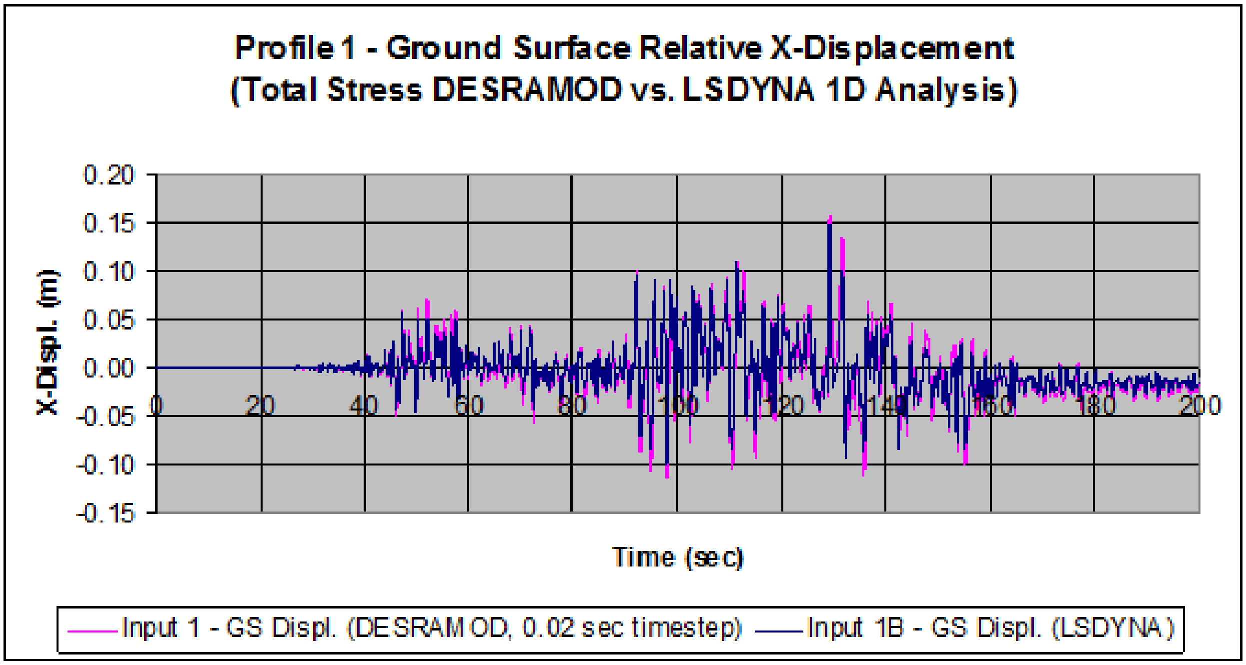


Figure F-7: Comparison of Computed Relative Lateral X-Displacement Versus Time at Ground Surface for Soil Profile 1 Considering Earthquake Record 1 Input from 1-D DESRAMOD and LSDYNA Models

F3.0 TWO DIMENSIONAL SEISMIC WAVE PROPAGATION ANALYSIS

A two dimensional (2-D) finite element model of the Plane 5 cross-section subjected to vertical propagating seismic shear waves was next developed using LSDYNA incorporating the following soil layers in order of increasing elevation: dense glacial till, GLU, Unit 3 transitional glaciofluvial, Unit 2A glaciofluvial, tailings and granular embankment fills comprising the seismic berm, embankment raises and the starter embankment. The 2-D model is shown in Figure F-8. The model was zoned laterally to account for differing overburden stress conditions and to permit stress level variations in small strain shear modulus G_{max} and elastic bulk modulus B_{elas} to be considered. Stress level dependent variations in shear strength were considered based on the use of a Mohr-Coulomb strength criterion. Soil zones 1 to 7 were considered with Zone 1 located at the left hand side of the model. Soil zone numbering increased from left to right across the model. The boundaries of each zone are indicated by the vertical lines shown in Figure F-8. Within each of these zones different material "parts" were included, necessitating definition of nonlinear stress – strain model parameters for each part. Elastic boundaries were included along the left and right edges of the 2-D model and along the base of the model to permit seismic wave energy to be transmitted beyond the lateral and bottom boundaries and to prevent seismic waves being reflected back into the model during dynamic shaking.

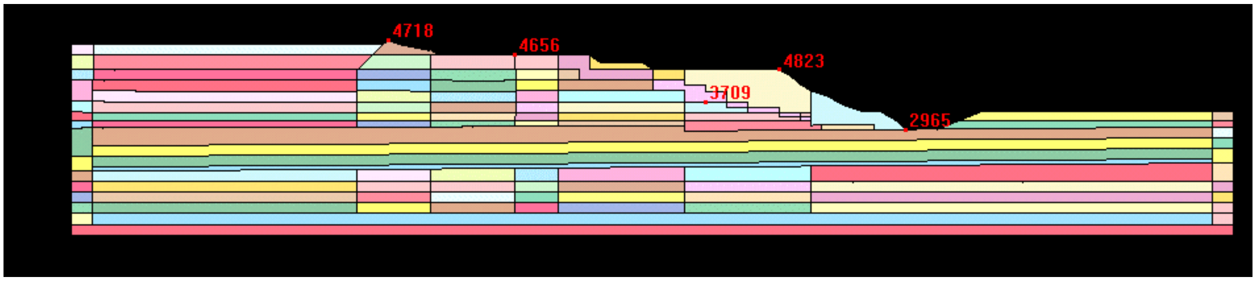


Figure F-8: Two Dimension LSDYNA Model of "Plane 5" Cross-Section, Showing Locations of Nodal Output

The left hand and right hand side of the model has a height of 90 m and 58 m, respectively. The width of the model is 550 m. The 7 soil zones are demarcated by the vertical lines in the model with Zone 1 at the left hand side and Zone 7 at the right hand side. Elastic elements are placed on the left hand, right hand and bottom boundaries of the model.

F3.1 Nonlinear Stress-Strain Response Using Geologic Cap Model

LSDYNA can consider a variety of nonlinear soil stress-strain models and can incorporate large strain soil deformations (Chen and Baladi, 1985: Weidlinger Associates,1978). It also uses very sophisticated convergence criteria, important for analysing deformations due to strong seismic shaking. The various soil zones have been assumed to have nonlinear, hysteretic response during cyclic shearing with limiting shear strength calculated using a Mohr-Coulomb failure criterion. Nonlinear material response for both static and cyclic loading was modelled using a nonlinear "geologic cap model" or GCM (Livermore Software Technology, 2001). For small changes in shear stress, response is linear elastic and elastic strains occur. For larger changes in shear stress, plastic strains occur resulting in nonlinear shear stress – shear strain response. Shear stresses are limited according to the Mohr-Coulomb failure criterion.

(a) Static Stress-Strain Response

For static gravitational loading prior to earthquake shaking, granular soils (Unit 2A glaciofluvial, Unit 3 transitional glaciofluvial, various embankment dam materials) were assumed to undergo fully drained response and drained shear strengths were calculated in terms of peak friction angles φ'. The very dense glacial till was assumed undrained in response and to develop high undrained shear strengths based on the assumption of pore water cavitation, discussed previously in Section F1.0. The tailings and GLU layers were assumed to be undrained in their static response and equivalent friction angles were used to calculate stress-level dependent undrained strengths and undrained strength ratios (undrained strength divided by vertical effective stress) based on average K_0 values in these materials. The strength ratios will therefore depend on K_0 which can vary somewhat throughout the model from the average K_0= 0.75 assumed. Higher K_0 values will lead to higher undrained strength ratios using Mohr-Coulomb strength theory and an equivalent friction angle. For both drained and undrained static response, stress-strain behaviour was simulated using the elastic-plastic GCM model characterized by an elastic shear

(G_{max}) and elastic bulk modulus (B_{elas}) for stress states below the Mohr-Coulomb failure envelope, or the moving plastic hardening surface ("cap") defined in terms of stress invariants. An associative flow rule is presumed for stress states on the failure envelope or cap, and plastic strains are computed at failure using this flow rule.

A summary of key parameters used in assessment of initial stresses within the 2-D model under static gravitational loading conditions is provided in Table 6-1. It is noted that B_{elas} was set equal to $10G_{max}$ for all saturated native foundation materials below the groundwater table (glacial till, GLU, glaciofluvial materials) as well as for the tailings. This relationship assures nearly zero volume change conditions during dynamic shaking and was also assumed for initial static analysis. For unsaturated materials (seismic berm fill, embankment raise fill, starter embankment) the bulk modulus was estimated based on published relationships between B_{elas} and mean effective confining stress for various types of coarse grained granular materials.

(b) Cyclic Stress – Strain Response

Under cyclic (earthquake) loading conditions, the geologic cap model is also able to simulate cyclic hysteretic response (which results in internal hysteretic damping within the soil mass) and permanent strain accumulation resulting from cyclic shearing. This cyclic hysteretic response was demonstrated previously in the 1-D site response model and shown in Figure F-4. The primary limitation of LSDYNA is that cyclic pore pressure development (pore pressures in excess of pre-earthquake initial pore pressures in the ground) during seismic shaking cannot be explicitly considered using soil stress-strain models currently implemented in the computer code. For saturated soil zones considered to be subject to significant cyclic pore pressure generation during seismic shaking (the GLU and tailings and, to a lesser extent, the Unit 3 transitional glaciofluvial soils), the present modeling used a "total stress" approach to describe the cyclic shear stress-shear strain response. The cyclic pore pressure build-up and resultant strain accumulation is termed cyclic mobility in the engineering literature. Where cyclic pore pressures approach the pre-earthquake vertical effective stress within a particular soil element, transient liquefaction is said to occur, accompanied by relatively large shear strains. It does not necessarily result in permanent loss of soil shear strength (relative to pre-earthquake conditions) since after the end of cyclic earthquake loading, compact to dense granular soils may dilate during shear loading (imposed by the weight of the dam) and regain most of the shear strength temporarily lost during shaking. Looser granular soils and normally consolidated, low plasticity cohesive soils (e.g. the tailings and GLU) may lose significant shear strength during and following seismic shaking due to sustained positive pore pressure build-up.

The geologic cap model was used in single element calibrations to simulate cyclic shear strain accumulation at a specified cyclic resistance ratio (or CRR) for 25 to 30 constant amplitude shear stress cycles. The number of effective cycles used in the calibrations was considered representative of the number of effective shaking cycles in a large M9 subduction event for granular soils (Idriss and Boulanger, 2008). Here a CRR is defined as a constant amplitude shear stress divided by the vertical effective overburden stress. The model can also simulate development of a large strain (post transient liquefaction) residual strength where deemed appropriate.

Unsaturated soil zones and the saturated, dense to very dense glaciofluvial units were assumed to not generate any significant excess pore pressures during seismic shaking. The same soil parameters used in initial static modeling were used during analysis of seismic shaking. The latter parameters also include the effects of cyclic hysteresis.

In order to carry out the single element calibrations for the GLU and tailings materials, cyclic direct simple shear test data provided in Appendix C and D was reviewed. Plots were made of number of CRR cycles versus cumulative cyclic shear strain measured in the laboratory tests. These are shown in Figure F-9 and Figure F-10 for the GLU and tailings, respectively. Data points are shown for various CRR levels. While there is considerable scatter in the data, the data show that cumulative shear strains increase with number of cyclic stress cycles and increasing CRR levels.

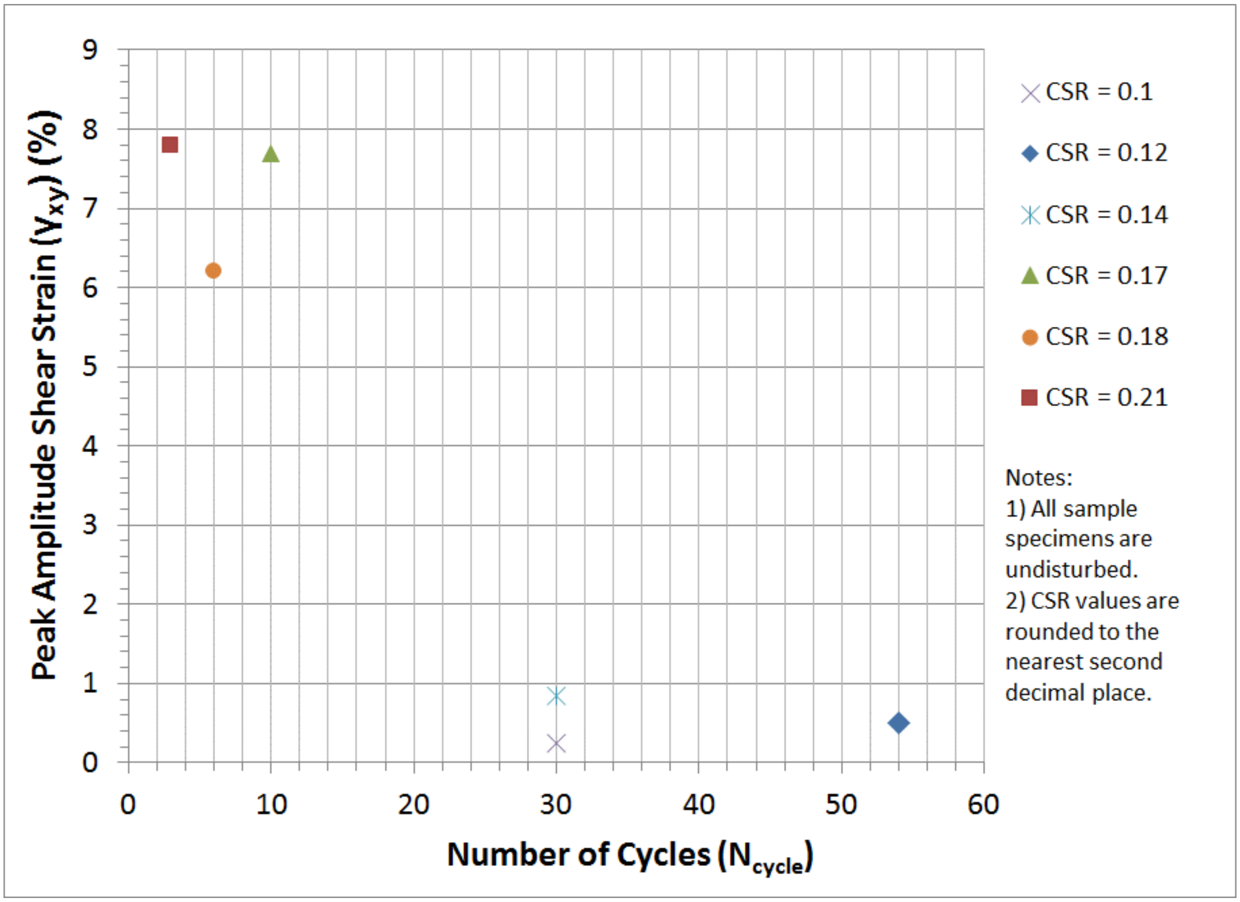


Figure F-9: Cumulative Cyclic Shear Strain Versus Number of Constant Amplitude Shear Stress Cycles for Various CSR Levels Based on Cyclic Lab Testing in the GLU

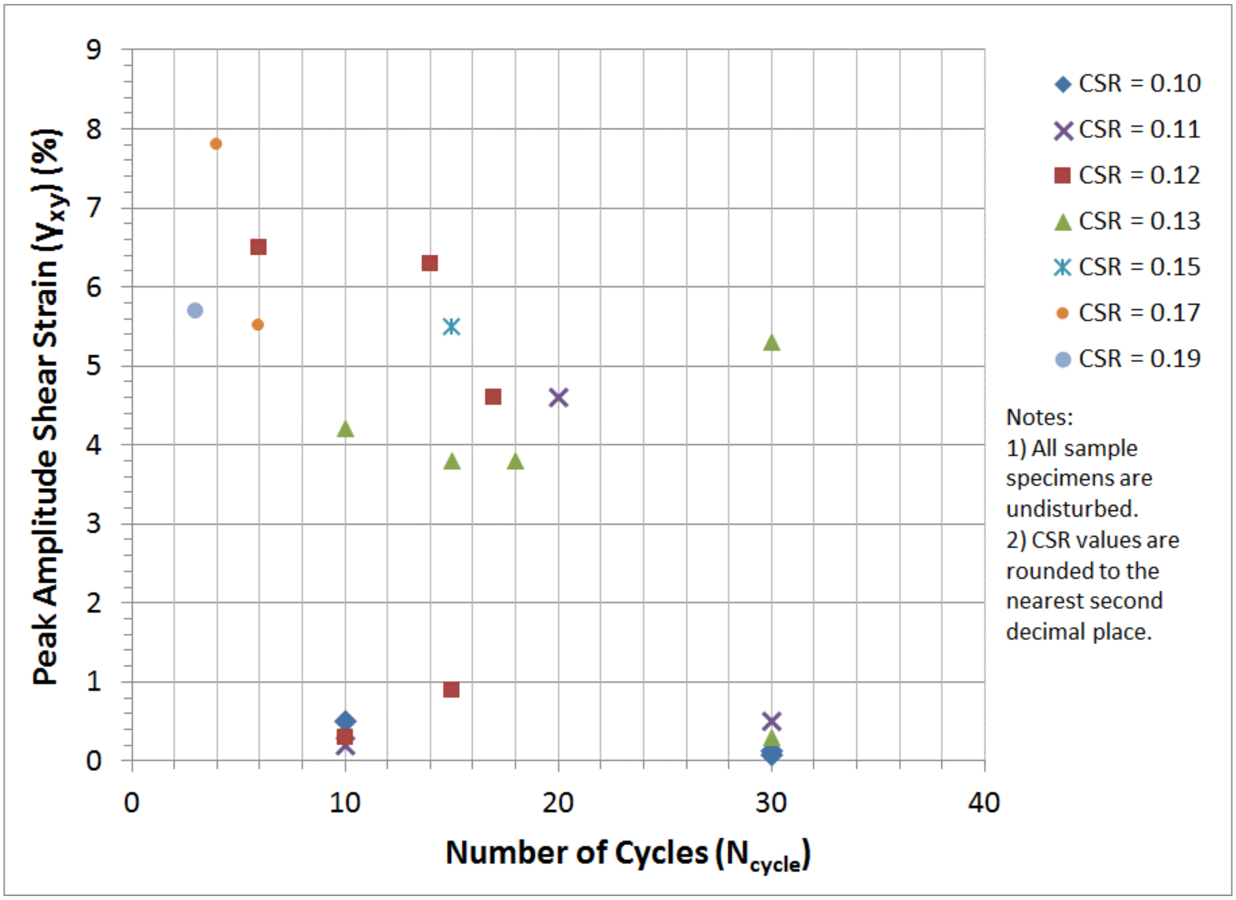


Figure F-10: Cumulative Cyclic Shear Strain versus Number of Constant Amplitude Shear Stress Cycles for Various CSR Levels in Tailings

The 1-D site response analysis (SRA) indicates that peak CSR levels are typically in the range of 0.15 to 0.22 with an average of about 0.2 in both the tailings and GLU. These peak CSR values were also confirmed by the results of 2-D seismic site response analysis, discussed subsequently. The cyclic shear stress (and CSR) time histories in the tailings and GLU computed from the SRA are not uniform over time but exhibit peaks and lows. The "effective" constant amplitude CSR has been found to be about 0.5 times the peak based on calculations of the Root Mean Square (RMS) of the random CSR time history compared to a constant amplitude, sinusoidal CSR variation with time considering 30 stress cycles. The RMS of a particular CSR time history is a measure of the amplitude and duration of cyclic stressing and therefore the intensity of shaking to cause cyclic pore pressure build-up and strain accumulation. The correlation between peak and effective CSR was found from analysis of both 1-D and 2-D site response analyses considering earthquake input records (1) and (2).

Examining Figure F-9 for the GLU, the data suggest that a cumulative shear strain of about 1% after about 30 stress cycles should be expected for an effective CSR = 0.1 (= 0.5 times the peak CSR of 0.2). For higher CSR levels in the range of 0.17 to 0.21 (just slightly below the undrained static shear strength ratio for the GLU) cumulative shear strains in the range of 6 to 8% after 10 constant amplitude cycles should be expected. Examining Figure F-10 for the tailings indicates that a cumulative shear strain of about 5% after about 20 to 30 cycles should be expected for the tailings for a CRR in the range of 0.10 to 0.13.

The above cyclic lab test data was used to calibrate single element models for the GLU and tailings to give cumulative shear strains after N cycles of constant amplitude CSR loading in reasonable agreement with the cyclic lab test data. It is noted that cumulative shear strains developed in the single element calibrations within the tailings and GLU may not be fully developed in the full 2-D model due to constraints imposed by the buttressing effect of the seismic berm or other adjacent soil regions.

For the single element calibrations carried out for the Unit 3 transitional glaciofluvial soils a design $N_{1,60}$ value of 18 was adopted for these materials (approximately the 33rd percentile of $N_{1,60}$ values measured in these materials). The cyclic resistance ratio (CRR) for these materials was estimated using approaches given by Idriss and Boulanger (2008, 2014). Here a CRR is defined as the constant amplitude, cyclic shear stress after N cycles of shaking required to trigger transient liquefaction and excessive cyclic shear strain development. A CRR value of 0.13 was estimated based on the following parameters:

- Pre-earthquake vertical effective stress levels in the range of 400 to 1000 kPa within Zones 5 to 7 where cyclic shear strains are expected to be most significant in the Unit 3 materials.
- Fines content of 35% resulting in an equivalent clean sand N1,60,cs= 24.
- Magnitude 9 design seismic event.

Since effective CSR values of 0.10 under the design levels of earthquake shaking are expected in the Unit 3 materials based on the 1-D and 2-D SRA results, there is a factor of safety against transient liquefaction triggering of about 1.3. For these factors of safety, cumulative shear strains after N = 30 cycles of shaking at an effective CSR = 0.10 are likely to be in the range of 1 to 2%. (Wu, 2003; Ishihara and Yoshimine, 1992) A target cumulative strain level of 2% after 30 cycles at a CSR = 0.10 was used in single element calibrations.

Single element, cyclic simple shear models were developed using LSDYNA for various soil zones with each element subjected to an initial vertical effective and K_0 lateral effective stress prior to the start of cyclic loading. The peak shear stress amplitude was calculated as CRR x the initial vertical effective stress. Cyclic sinusoidal shear loading was applied as horizontal nodal forces to the top of the element using a 1 Hz loading frequency. Average shear strains in the element were computed as the average lateral X-displacement at the top of the element divided by the element height (2.0 m). The geologic cap model was used to simulate cyclic strain accumulation within the element through judicious selection of the following key parameters:

- Equivalent friction angle Φ_{equiv} designed to simulate a specified undrained shear strength ratio S_u/σ'_{vo} based on a pre-earthquake K_0 value for the GLU, tailings or Unit 3 glaciofluvial (silty sand) materials. The strength ratio lies between the peak strength ratio used for static loading analysis and the post-seismic residual strength ratio for a particular material type. Equivalent friction angles for other materials were set equal to peak drained friction angles.

- Shear modulus $G = G_{max}$.

- Bulk modulus $B = 10$ times G_{max} to achieve approximately zero volume change during cyclic undrained loading.

- Cyclic strain accumulation factor R used in the geologic cap model formulation (Weidlinger Associates, 1978).

Achieving the required degree of cyclic shear strain accumulation was sensitive to Φ_{equiv}, G_{max} and R.

A summary of key cyclic loading parameters used in the seismic modeling for various soil zones is provided in Figure F-3.

Table F-3: Summary of Geotechnical Properties Used in 2-D Modeling

Material Type	Total Unit Weight, γ (kN/m³)	Stress Normalized Seismic Shear Wave Velocity, V_{s1} (m/sec)	Seismic Shear Wave Velocity, V_s (m/sec)	Small Strain Shear Modulus, G_{max} (MPa)	K_0	Elastic Bulk Modulus, B_{elas} (MPa)	Static: Peak Friction Angle, Φ' (deg.)	Static: S_u/σ'_{vo}	Cyclic: Equivalent Peak Friction Angle, Φ_{equiv} (deg.)	Cyclic: S_u/σ'_{vo}	Cyclic: R
Seismic Berm	22	270	221-291	110-190	0.38	71-123	38	n/a	38(5)	n/a	2(5)
Embankment Raises	22	270	244-336	133-253	0.44	87-143	34	n/a	34(5)	n/a	2(5)
Tailings	22.5	195	160-330(2)	60-250(2)	0.75	600-2500	13.2	0.20	11.9	0.18	150
Starter Embankment	22	280	383	329	0.41	183	36	n/a	36(5)	n/a	2(5)
Dense Glaciofluvial Sand and Gravel with loose layers (Unit 2a)	24	210	350-370	300-330	0.44	3000-3300	34	n/a	34(5)	n/a	150
Transitional Silty Sand (Unit 3)	22	203	359(3)	290 (3)	0.47	2900	32	n/a	29	0.35	150
Glaciolacustrine Silt and Clay (Unit 4)	20	175	285-335(2)	165-230(2)	0.75	1650-2300	14.5	0.22	14.5(5)	0.22(5)	150
Dense Glacial Till	24	≈230-240	450	495	0.75	4950	35(1)	n/a	35(1)	n/a	2(5)

Φ_{equiv} = equivalent peak friction angle designed to match target shear strength ratio

(1) Shear strength calculated assuming pore water cavitation
(2) V_s (and G_{max}) depends on stress level according to relationship with V_{s1}
(3) Average V_s, G_{max} and B_{elas} properties across all zones were selected for the Unit 3 transitional glaciofluvial materials which were assumed to have a 2m thickness across the model.
(4) Average V_s, G_{max} and B_{elas} properties across all zones were selected for the Unit 2A glaciofluvial materials which was assumed to have a thickness of 18 m across the model except at the far RHS of the model where the thickness increased by approximately 7 m. Depth dependence of these material parameters was considered.
(5) No change from initial static properties
n/a = not applicable

(c) System Damping

Relatively low degrees of additional Rayleigh-type stiffness and mass-proportional damping were used for all dynamic analyses, corresponding to 2 to 5% critical damping levels over the 0.5 to 2 second period range and up to 8% damping in the period range of 0.05 to 0.5 seconds. Note that the small strain fundamental period of the seismic berm and underlying foundation soils is calculated equal to $4H/V_{s,avg}$ where H is the thickness of the overburden soils above the dense till and $V_{s,avg}$ is the average shear wave velocity of the soil profile above the till. Using an average soil profile for Soil Zone 6 under the crest of the seismic berm, an average shear wave velocity of 308 m/sec is calculated with H = 68 m. This results in a small strain fundamental period of 0.88 seconds. Under design levels of seismic shaking this fundamental period would be expected to lengthen to at least 1.5 times the small strain value. Thus, Rayleigh damping is expected to result in up to 5% critical damping over the above period range.

The dominant damping in the 2-D model is expected to result from internal damping generated by cyclic shearing. Radiation damping is also permitted in the model corresponding to the use of energy transmitting lateral and bottom boundaries.

F3.2 Stages of Modeling

The various stages of 2-D finite element modeling are analogous to the stages used in the 1-D site response modeling. To iterate, these included:

- Stage 1 – Initial self-weight gravity loading of the dam (seismic berm, APA berm and embankment fills) and its foundations to calculate pre-earthquake effective stresses throughout the model. Initial pore pressures in the native foundation soils (glaciofluvial, GLU and glacial till soils) were calculated using a hydrostatic pore pressure distribution with zero pore pressure at the top of the foundation soils (base of tailings). Pore pressures within the tailings were assumed to be zero at the top and bottom of the tailings, increasing to a maximum value of 80 kPa at the mid-depth of the deposit. Simulation of initial pore pressure distributions required calculation of equivalent unit weights within the various soil regions (or “parts” using LSDYNA terminology) to give the correct vertical effective stresses. Gravitational loading was simulated by applying equivalent vertical nodal loads within a particular soil part. These were calculated as the area of the soil part times the equivalent unit weight and divided by the number of nodes in the part. This approach was found to give excellent agreement with vertical effective stresses calculated using the “gravity turn on” command within LSDYNA and specifying the equivalent unit weights for each soil part. The advantage of the equivalent nodal force approach is that one can specify the “real” total unit weights (comprised of the soil mass and water mass) for each soil part necessary for calculation of seismic horizontal inertial loads during seismic shaking.

- Stage 2 – Dynamic modeling of dam response with no cyclic strain accumulation considered from cyclic pore pressure generation. This was done to check on cyclic shear stress levels in the tailings, GLU and Unit 3 glaciofluvial soils. The model also provided lower bound estimates of dam deformations. Pre-earthquake static soil properties were used as presented in Table 6-1. Cyclic hysteresis in the stress-strain models was considered. Four input records were considered (Records (1) to (4)) as presented in Section 4.2). A shear stress time history was applied at the base of the model computed for a particular input record to simulate earthquake excitation of the model, as described in Section 5.1.

Dynamic modeling of dam response with cyclic strain accumulation resulting from pore pressure generation using the methods described previously and the soil stress-strain parameters presented in Table 5.4. Earthquake records 1 through 4 and 6 were used in this evaluation to assess the effect of the earthquake record on computed dam displacements.

- Stage 3 – Post-seismic stability analysis of dam response assuming cyclic shaking results in strength reduction relative to pre-earthquake strengths within the tailings, GLU and Unit 3 glaciofluvial soils. A static analysis using self-weight gravitational loading (based on the equivalent vertical nodal load approach) was used after completion of the Record 1 input motion to check whether the dam was stable following mobilization of these reduced post-seismic strengths.

F3.3 Model Boundary Conditions

During initial self-weight gravity loading, the following boundary conditions were used:

- Base of model fixed in the lateral (X) and vertical (Y) directions.
- Lateral elastic boundaries free to move in X and Y directions but lateral K_0 stresses were applied to the boundaries.

During dynamic shaking, the following boundary conditions were used:

- Base of model free in the X direction but constrained against vertical motion.
- Lateral elastic boundaries free to move in X and Y directions but lateral K_0 stresses were applied to the boundaries.
- Because of the imbalance of lateral forces applied to the lateral boundaries (forces on left hand side of boundary was greater than those on the right hand side) and that the base of the model was free to move in the X direction, it was necessary to apply a corrective shear load on the bottom boundary to prevent lateral drift of the model. This corrective shear load was calculated as the difference in the lateral loads applied to the left hand and right hand boundaries.
- Seismic wave energy was permitted to be transmitted through the lateral and bottom boundaries using a “non-reflecting boundary” command.

F3.3.1 Results of 2-D Modeling

The following results are presented:

- Contours of pre-earthquake vertical effective stresses throughout the model are shown in Figure F-11.

- Contours of maximum shear strain at the end of seismic shaking using input Record (1) with no consideration of cyclic degradation in the GLU, tailings and Unit 3 materials, i.e. based on the pre-earthquake "static" soil properties are shown in Figure F-12. Examination of this figure indicates a critical slip surface (zone of highest shear strains) extending from upstream of the seismic berm and exiting near the toe of the seismic berm. This mode of deformation is similar to the critical potential failure surface identified by Surface D from SLOPE-W modeling which indicated a post-seismic factor of safety of 1.2 for this mode of potential failure. ***The shear strains output from LSDYNA (termed "Green's strains") are ½ of engineering shear strains used in normal engineering practice***.

- Contours of maximum shear strain at the end of seismic shaking using input Earthquake Record 1 with consideration of cyclic degradation in the GLU, tailings and Unit 3 materials, i.e. based on the "cyclic" soil properties are provided in Figure F-13. The broadening of the zones of maximum shear strain are apparent using the cyclic soil properties when comparing Figure F-12 and Figure F-13.

- Plots of peak cyclic shear stress ratio versus depth are provided in Figure F-14 within various soil zones (Zones 2, 4, 5 and 6) considering input Earthquake Records 1 through 4. The CSR's show a reasonably tight band versus depth considering the various input earthquake records. A reasonable average peak CSR within the GLU, Unit 3 glaciofluvial and tailings is 0.2 with values as high as 0.25 due to locally higher K_0 values during shaking. The peak CSR's indicate that the maximum undrained shear strengths within the tailings and GLU are reached during shaking.

- A summary of computed permanent lateral displacements at the end of shaking considering five input ground motions is given in Table F-4. The model was run with and without the effects of cyclic degradation. As expected, including the effects of cyclic degradation increased post-seismic displacements by up to 35%. Maximum displacements were computed using input Earthquake Record 1 which has the highest Arias Intensity of all records considered. ***Maximum lateral displacements at the crest of the seismic berm of 0.96 m were computed which is considered to be within acceptable limits.*** The relatively limited lateral displacements computed are considered to be the result of the relatively flat slopes and buttressing effect of the seismic berm, as well as the large amount of damping considered in the model due to cyclic hysteresis within the tailings and GLU.

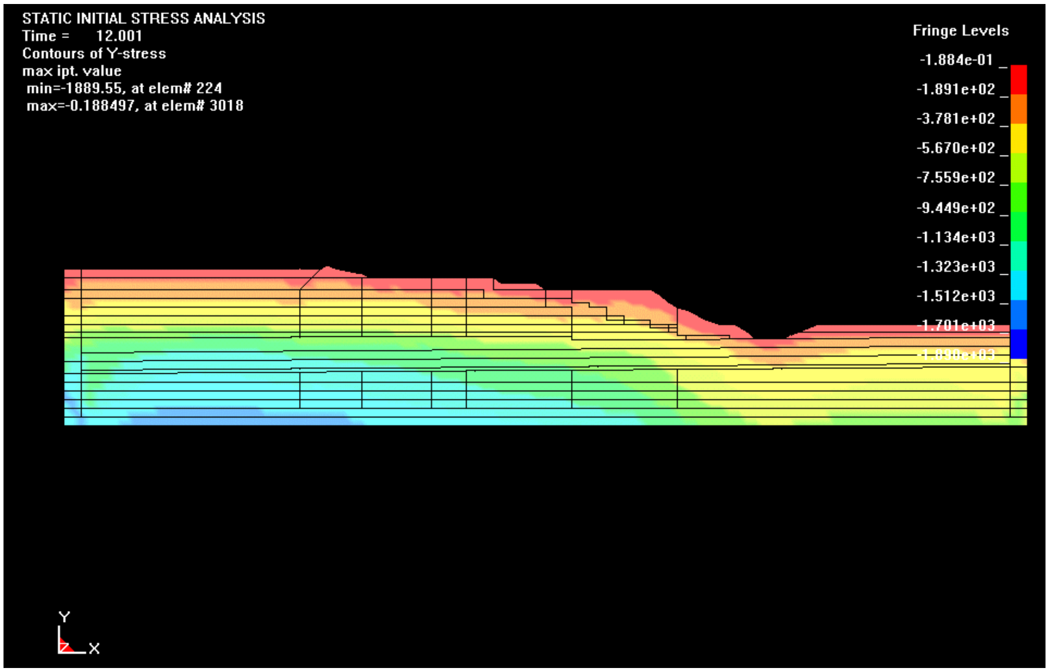


Figure F-11: Contours of Pre-Earthquake Vertical Effective Stress

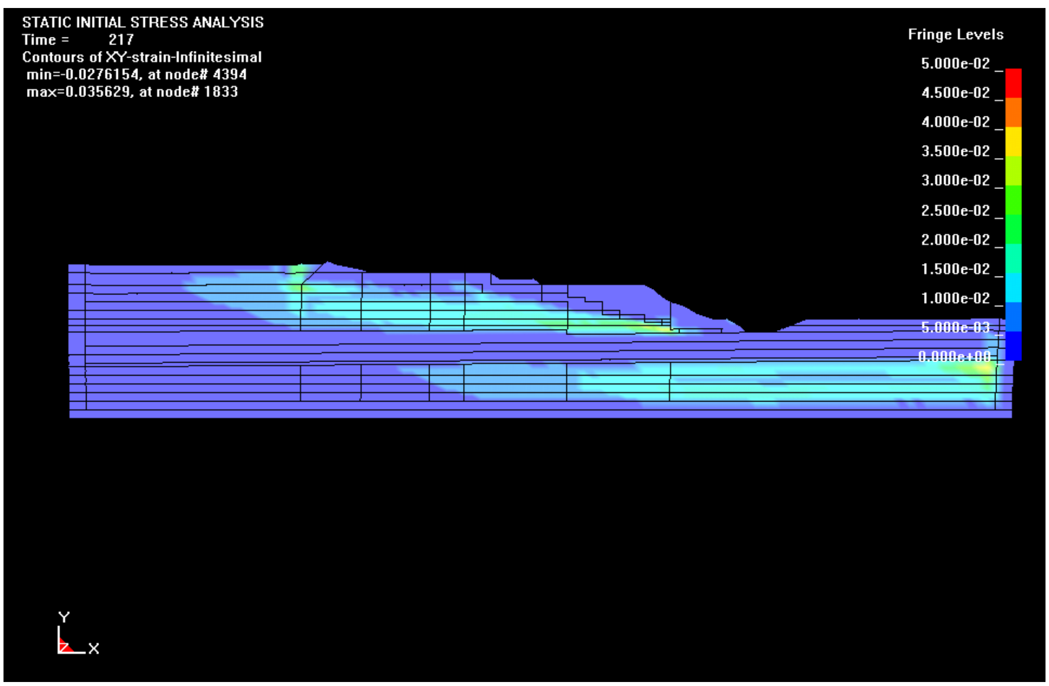


Figure F-12: Contours of Post-Earthquake Maximum Shearing Strains with No Effects of Cyclic Degradation

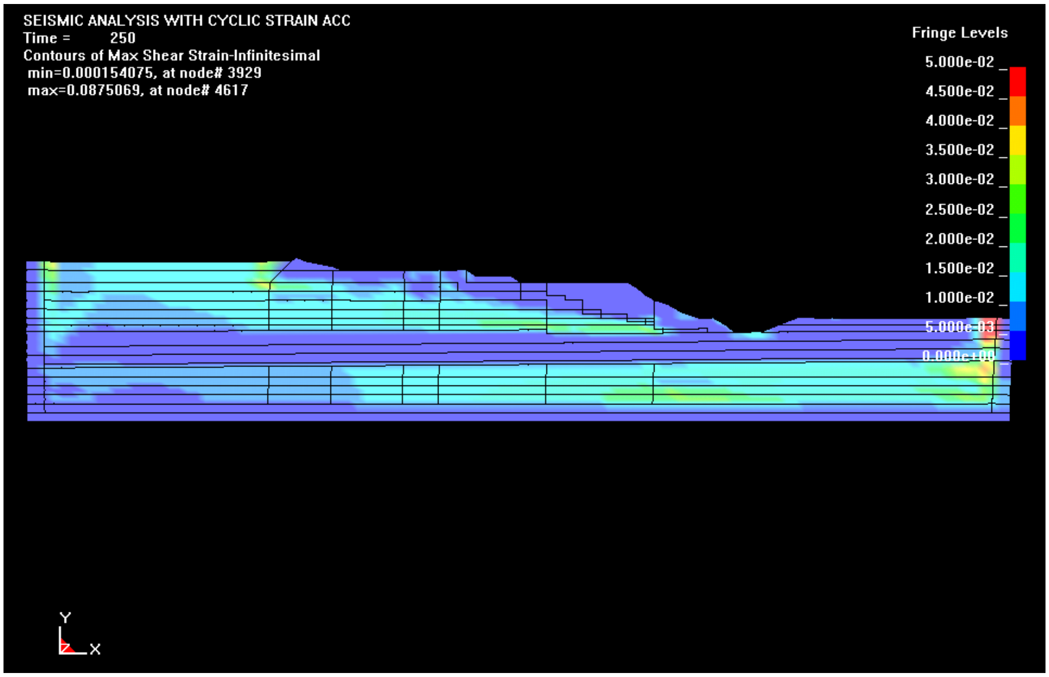


Figure F-13: Contours of Post-earthquake Maximum Shearing Strains with Effects of Cyclic Degradation

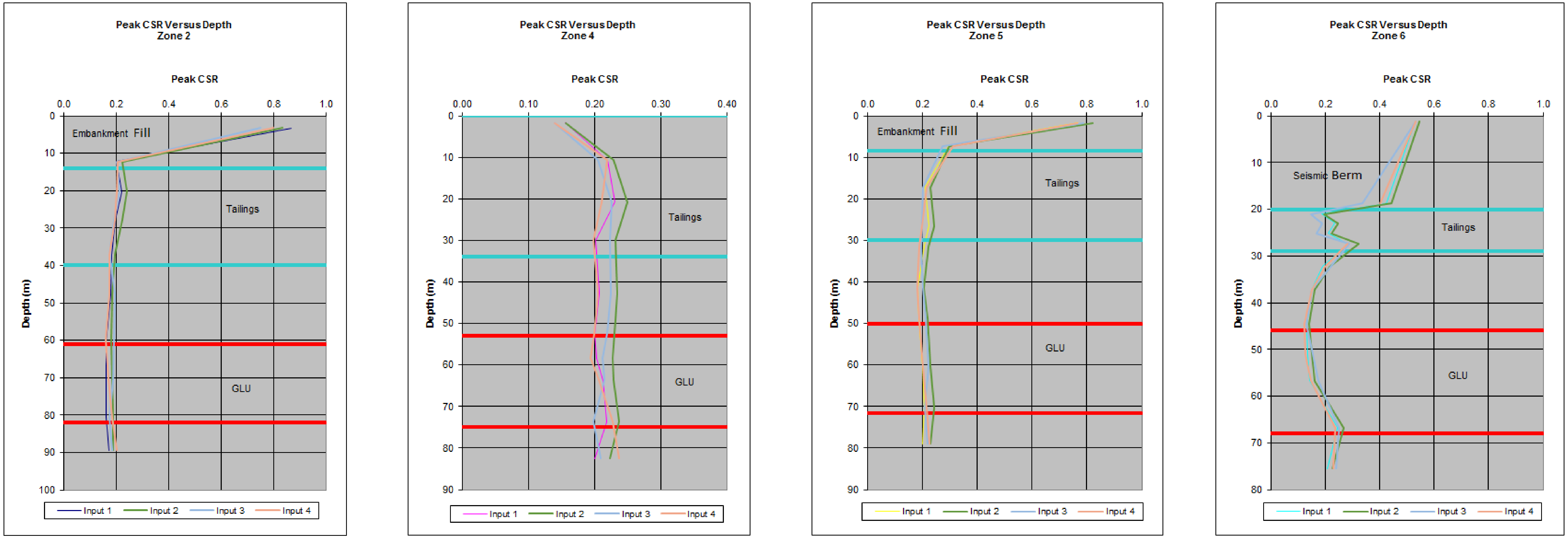


Figure F-14: CSR for Earthquake Records 1 through 4

F3.3.2 Approximate Methods to Estimate Seismic Displacements

Approximate methods were used to estimate post-seismic dam displacements to provide a check against the FE model results using the following approaches:

- **Bray and Travasarou (2007).** The Bray and Travasarou (B&T) model is based on a statistical analysis of a simplified 1-D nonlinear slope model considering a variety of seismic input motions over a range of earthquake magnitudes. The B&T displacements are sensitive to yield acceleration coefficient (k_y) and effective horizontal acceleration time history at the centroid of the slide mass. The latter is indirectly related to the input ground motion spectral acceleration characteristics which need to be specified at a structural period equal to 1.5 times the small strain fundamental period of the dam. The latter has been previously estimated to be approximately 0.9 seconds under the crest of the seismic berm. Using the B&T empirical equations and using a critical yield acceleration coefficient of 0.10 g determined from SLOPE-W analyses for Plane 5, the mean post-seismic displacements at the crest of the seismic berm for a M9 design earthquake are computed to be 1.0 m.

- **Newmark (1965)** subjected to an input base acceleration time history. An in house computer program was used to compute cumulative displacements of the rigid block which occur progressively whenever the base acceleration exceeds the yield acceleration of the soil mass above a critical slip surface. The horizontal acceleration time history computed by LSDYNA at node 3709 using input Record (1) with no consideration of cyclic degradation effects was used in the Newmark analysis. Node 3709 is located at the approximate centroid of the soil mass above a critical slip surface determined from SLOPE-W analysis (Surface D). A k_y of 0.10 also was used in the modeling. The Newmark model predicted a post-seismic lateral displacement of 1.09 m.

- **Idriss and Boulanger (2008)** which involves estimating post-seismic shear strains at different depths in a 1-D soil profile and integrating these strains versus depth to compute X-displacements at the soil surface. Considering a 1-D soil profile under the crest of the seismic berm (Zone 6), shear strain potentials were estimated within the tailings and GLU based on effective CSR's of 0.10 in these layers and using available cyclic lab test data. Maximum strain potentials of 5% and 1% were used in the tailings and GLU, respectively Shear strain potentials in the Unit 3 glaciofluvial were estimated to be equal to 2% as described in Section 5.2.1 (c). The shear strain profile assumed is shown in Figure F-15. Integration of this strain potential predicted a horizontal ground displacement at the crest of the seismic berm equal to 0.91 m. The horizontal shear strains computed using the 2-D LSDYNA for this soil profile and considering earthquake record 1, which predicted maximum post-seismic X-displacement at the crest of the seismic berm, are also shown in Figure F-15. The latter indicate lower strain development in the tailings due to the buttressing effect of the adjacent seismic berm, and larger strain development in the deeper GLU than estimated using the strain potential method.

The above simplified estimates of post-seismic lateral displacement at the crest of the seismic berm are in close agreement with the 2-D finite element model results.

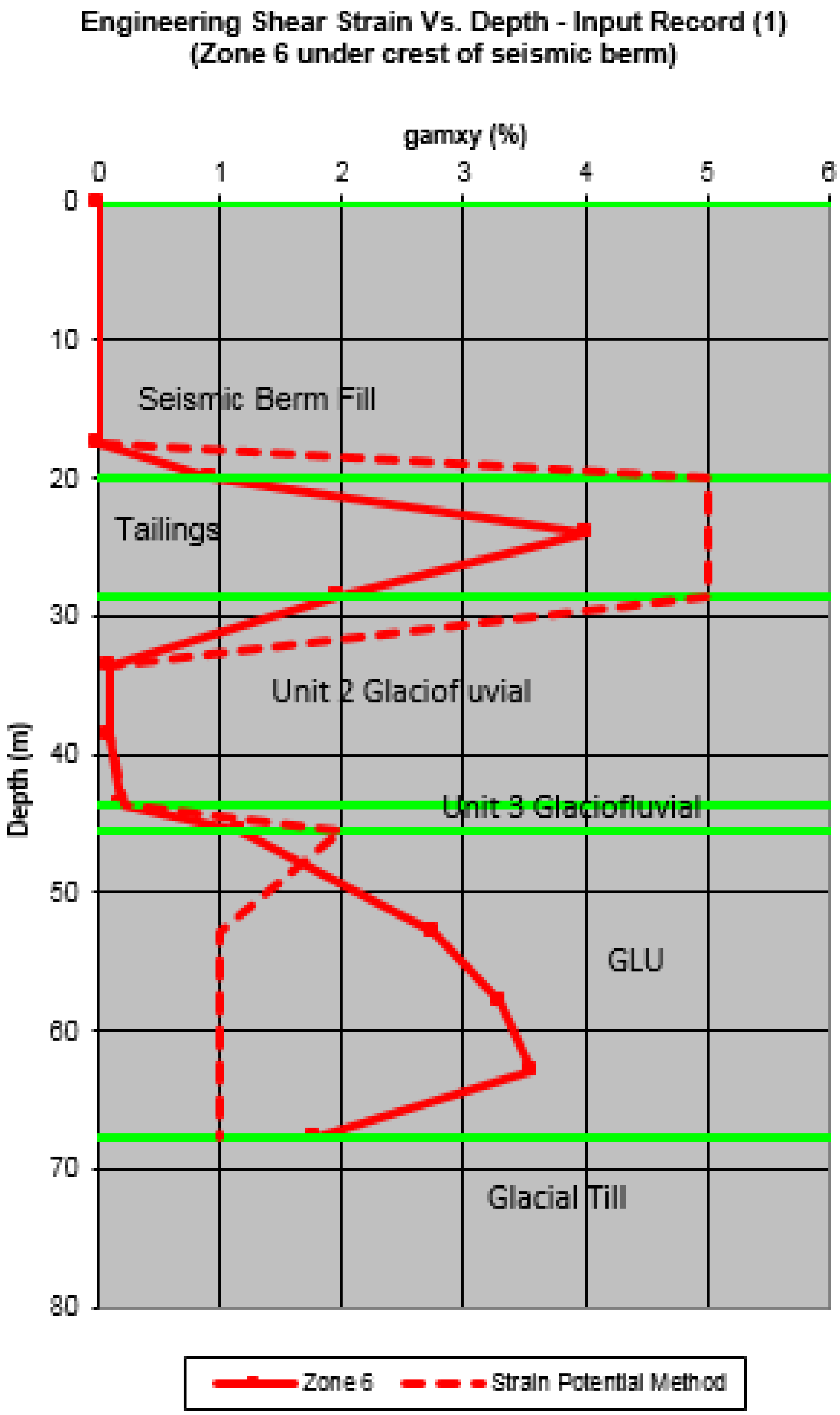


Figure F-15: Post-Seismic Shear Strains Versus Depth under Crest of Seismic Berm within Zone 6 Computed from LSDYNA for Input Record (1) and Estimated using Strain Potential Method

F3.3.3 Post-Seismic Stability Assessment

Cyclic straining under design levels of seismic shaking is considered to result in post-seismic strength reduction in the tailings, GLU and Unit 3 transitional glaciofluvial materials. The following post-seismic residual strength ratios were considered:

- Tailings S_r/σ'_{v0}= 0.14
- GLU S_r/σ'_{v0}= 0.13
- Unit 3 glaciofluvial (silty sand) S_r/σ'_{v0}= 0.35

K_0 values of 1.0 were assigned to the tailings and GLU, considered representative of post transient liquefaction conditions, giving equivalent friction angles of 8° and 7.5°, respectively. A K_0 value of 0.47 was assigned to the Unit 3 transitional glaciofluvial since transient liquefaction is not expected within these materials under design levels of shaking. An equivalent friction angle of 29° was assigned to the Unit 3 materials.

It was also assumed that following seismic shaking drained deformations would occur in the Unit 2A glaciofluvial and Unit 3 transitional glaciofluvial deposits. Because of the low permeability and long drainage path lengths in the tailings and GLU, it was assumed that deformations following seismic shaking would be essentially undrained.

The 2-D model was then re-run following the end of seismic shaking considering input Earthquake Record 1, which predicted the maximum seismic lateral displacements of all the records considered. Computed post-seismic lateral and vertical displacements are presented in Table F-4 and Table F-5, respectively. Using the reduced post-seismic strengths, the Old TDF is predicted to be stable and no additional lateral and vertical displacement results following the end of seismic shaking at the ground surface points output. Excess pore pressures are expected to be induced in the tailings and GLU which will dissipate slowly years following a major seismic event. This will lead to additional long term settlements of the dam which have not been considered in the present evaluation.

Table F-4: Summary of Computed Post-Seismic Horizontal Displacements

Earthquake Record	Comments	Lateral Displacement (m)			
		Crest of APA Berm (Node 4718)	Toe of APA Berm (Node 4656)	Crest of Seismic Berm (Node 4823)	Toe of Seismic Berm (Node 2965)
1	No cyclic degradation	0.65	0.69	0.73	0.60
2	No cyclic degradation	0.60	0.65	0.71	0.56
3	No cyclic degradation	0.39	0.44	0.50	0.40
4	No cyclic degradation	0.53	0.56	0.66	0.52
1	With cyclic degradation	0.85	0.95	0.96	0.79
2	With cyclic degradation	0.85	0.94	0.94	0.76
3	With cyclic degradation	0.48	0.56	0.63	0.50
4	With cyclic degradation	0.62	0.71	0.79	0.65
6	With cyclic degradation	0.58	0.65	0.68	0.53
1	Post-seismic residual strengths	0.85	0.95	0.96	0.79

Table F-5: Summary of Computed Post-Seismic Vertical Displacements

Earthquake Record	Comments	Vertical Displacements (m)			
		Crest of APA Berm (Node 4718)	Toe of APA Berm (Node 4656)	Crest of Seismic Berm (Node 4823)	Toe of Seismic Berm (Node 2965)
1	Post-seismic residual strengths	0.21	0.28	0.09	-0.06

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APPENDIX G

Permit Level Drawings

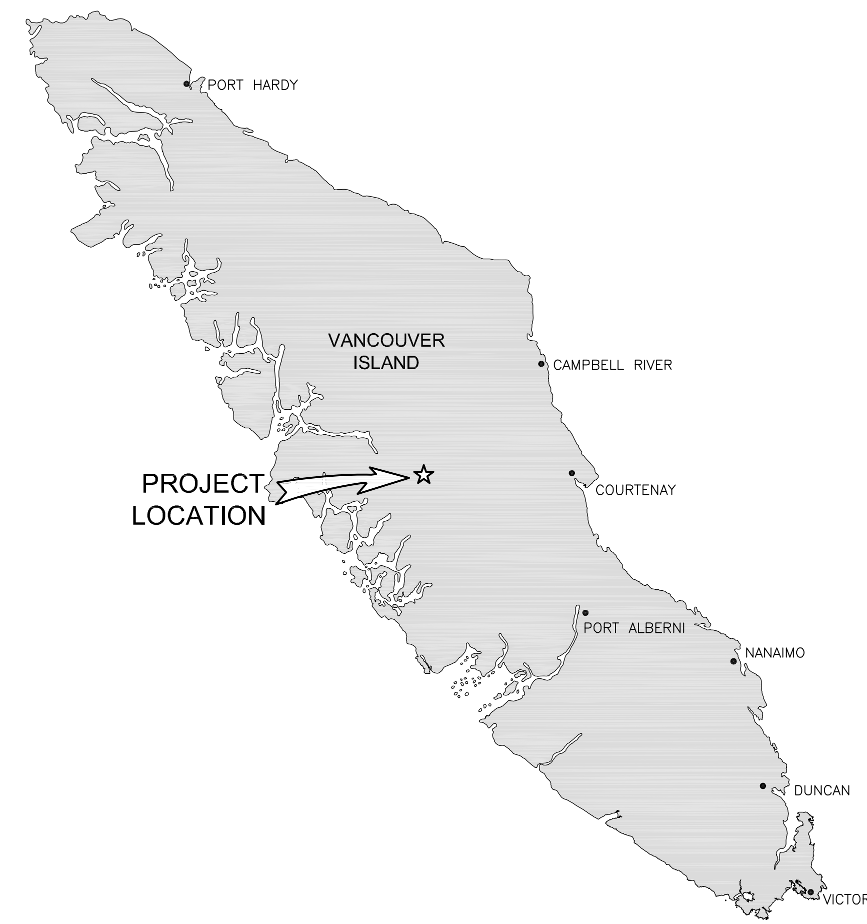
MYRA FALLS MINE SITE

AMALGAMATED PASTE AREA (APA)
BERM STABILIZATION
DETAILED DESIGN

nyrstar

LIST OF DRAWINGS

SHEET NO.	DRAWING NO.	DRAWING TITLE	ISSUE / REVISION
1	2001	COVER SHEET	A
2	2002	LEGEND	A
3	2003	OVERALL SITE PLAN	A
4	2101	OVERALL PLAN AND PROFILE	A
5	2102	SECTIONS A AND B	A
6	3103	SECTIONS C, D AND E	A
7	2104	WEST AND EAST DECANTS, DETAILS 1 AND 2	A
8	2201	SPECIFICATIONS	A



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ISSUED FOR REVIEW

REVISION A, 2016-07-29

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G						
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C						
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A	29	07	16	ISSUED FOR REVIEW	DA	DA

amec foster wheeler

Client: nyrstar

Designed By: DA
Drawn By: CMS
Checked By: DCH
Approved By: DA
Scale: SHOWN

Project: MYRA FALLS MINE SITE
APA BERM STABILIZATION

Title: COVER SHEET

Project No.: NX14001D
CADD File: 14001-2001.dwg
Date: July 2016
Drawing No.: 2001
Sheet No.: 1 of 8

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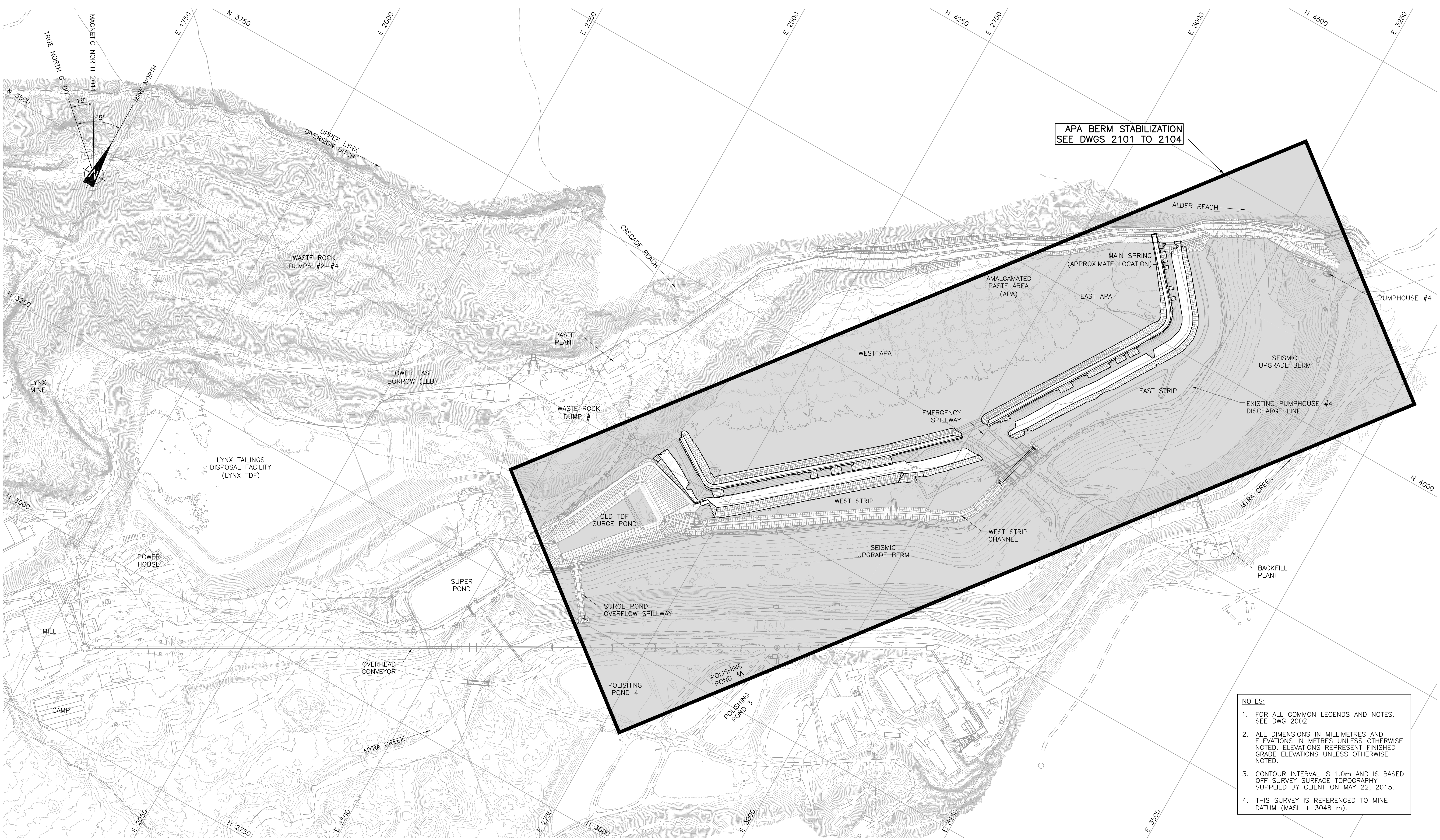

NOTES:

1. FOR ALL COMMON LEGENDS AND NOTES, SEE DWG 2002.
2. ALL DIMENSIONS IN MILLIMETRES AND ELEVATIONS IN METRES UNLESS OTHERWISE NOTED. ELEVATIONS REPRESENT FINISHED GRADE ELEVATIONS UNLESS OTHERWISE NOTED.
3. CONTOUR INTERVAL IS 1.0m AND IS BASED OFF SURVEY SURFACE TOPOGRAPHY SUPPLIED BY CLIENT ON MAY 22, 2015.
4. THIS SURVEY IS REFERENCED TO MINE DATUM (MASL + 3048 m).

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Client: nyrstar

Designed By:	DA
Drawn By:	CMS
Checked By:	DCH
Approved By:	DA
Scale:	1:2500

Project: MYRA FALLS MINE SITE
APA BERM STABILIZATION

Title: OVERALL SITE PLAN

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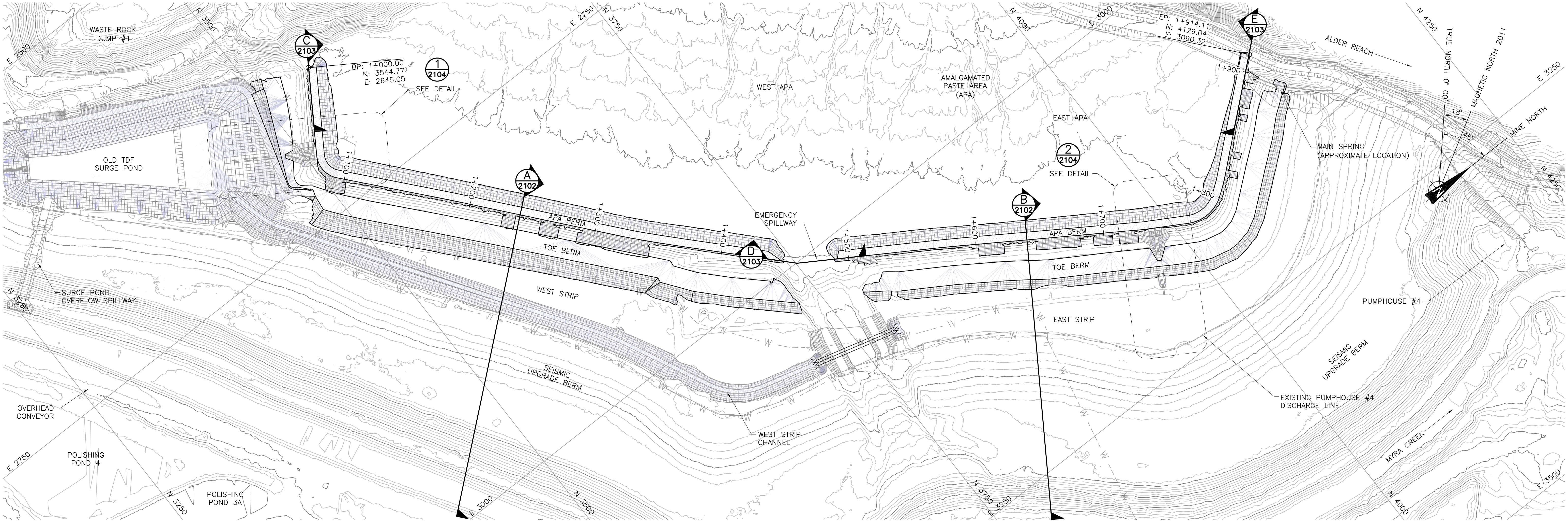
CADD File:	14001–2003.dwg
Date:	July 2016
Drawing No.:	2003
Sheet No.:	3 of 8



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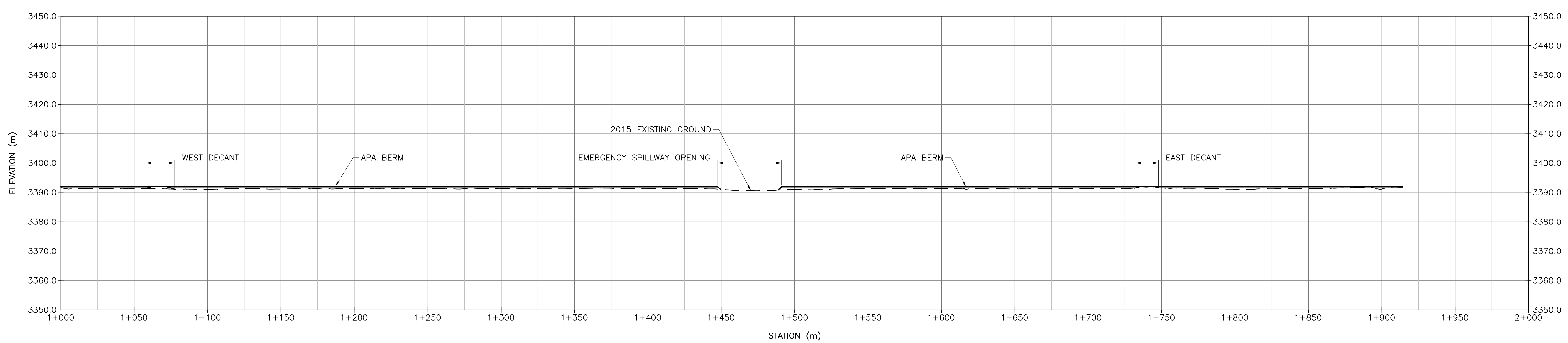

PROFILE – APA BERM

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SCALE VER. 1:750

NOTES:

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3. CONTOUR INTERVAL IS 1.0m AND IS BASED OFF SURVEY SURFACE TOPOGRAPHY SUPPLIED BY CLIENT ON MAY 22, 2015.
4. THIS SURVEY IS REFERENCED TO MINE DATUM (MASL + 3048 m).

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Project: MYRA FALLS MINE SITE
APA BERM STABILIZATION

Title: OVERALL PLAN AND PROFILE

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Date:	July 2016
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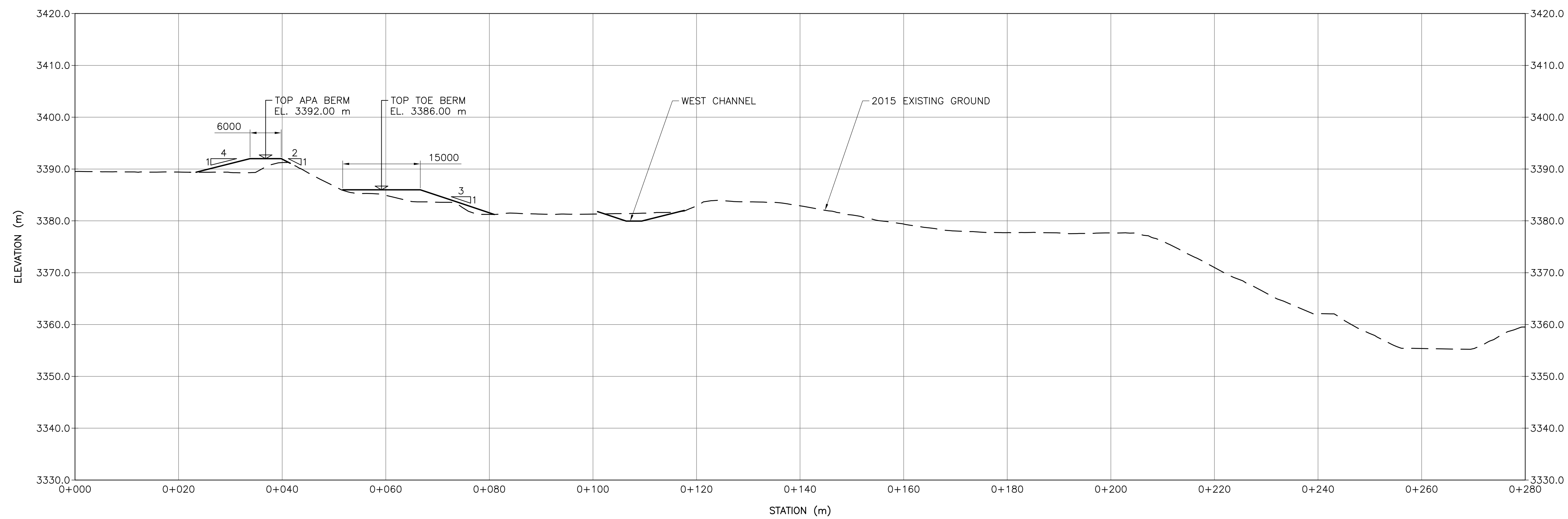

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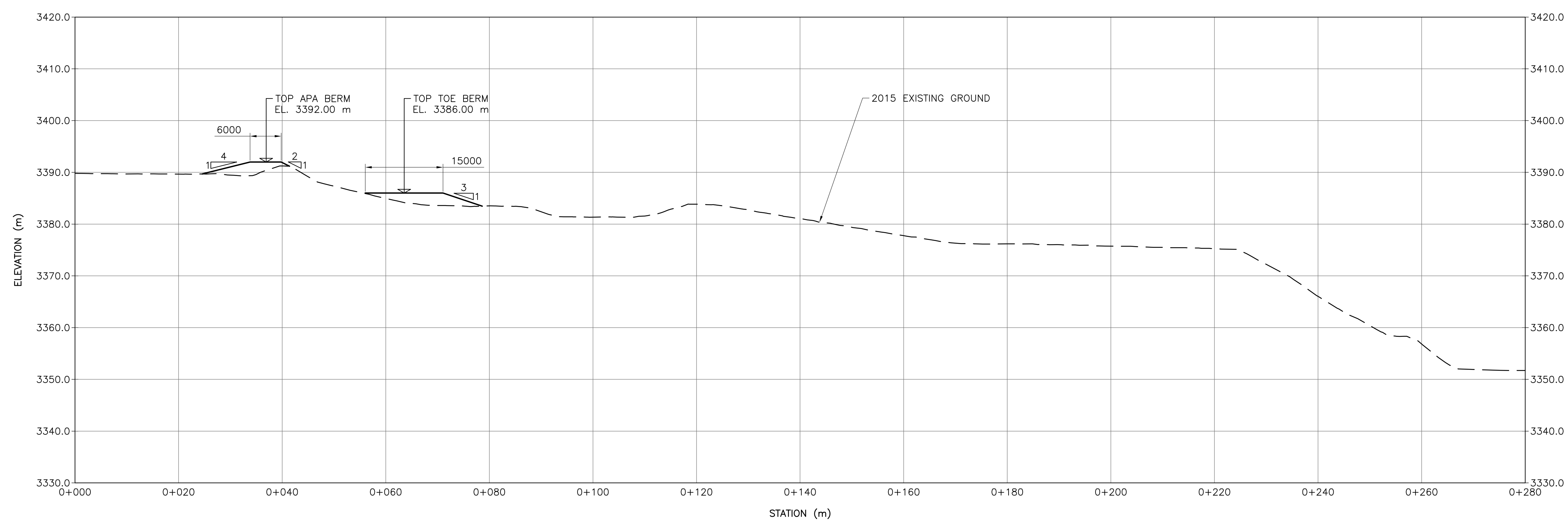

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Client: nyrstar

Designed By: DA
Drawn By: CMS
Checked By: DCH
Approved By: DA
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Project: MYRA FALLS MINE SITE
APA BERM STABILIZATION

Title: SECTIONS A AND B

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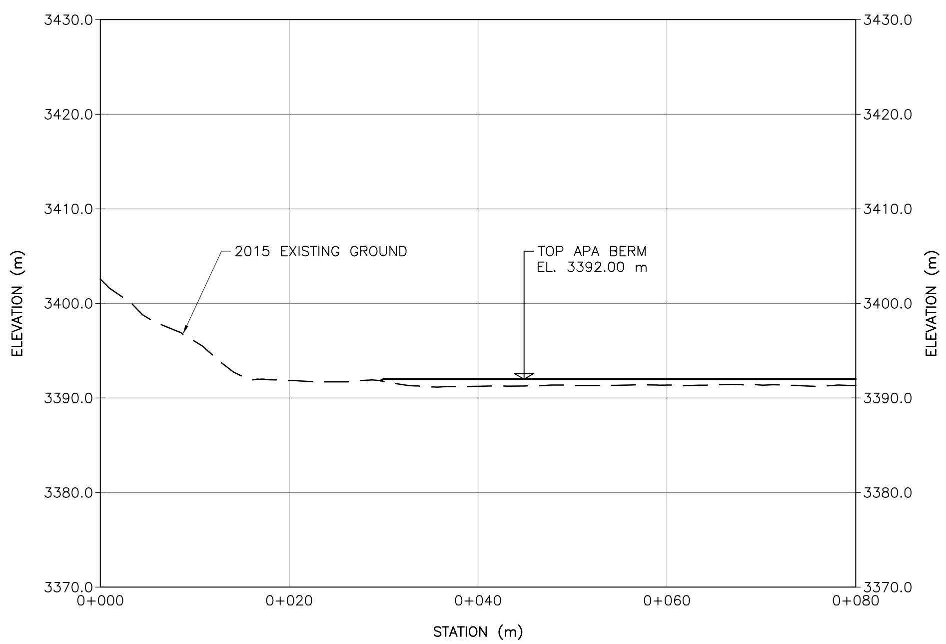
Sheet No.: 5 of 8



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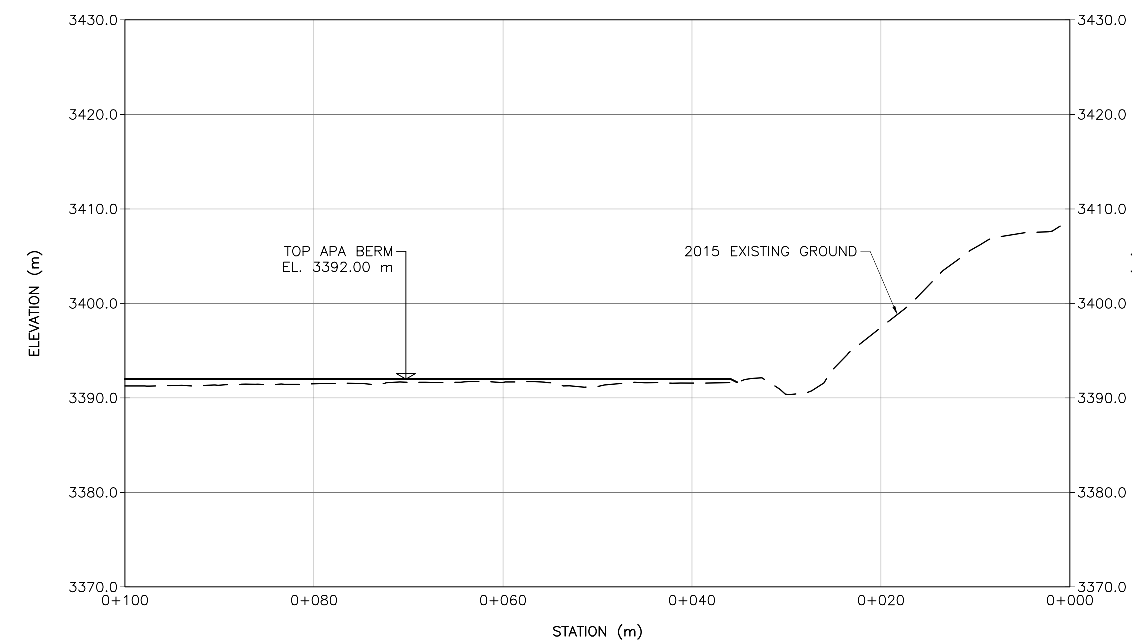

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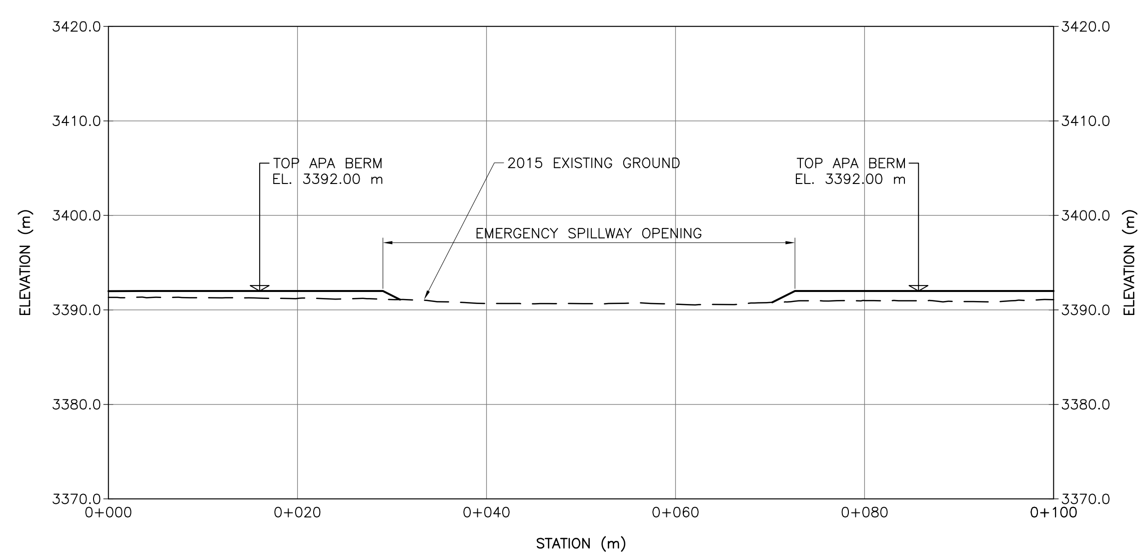

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NOTES:

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Project: MYRA FALLS MINE SITE
APA BERM STABILIZATION

Title: SECTIONS C, D AND E

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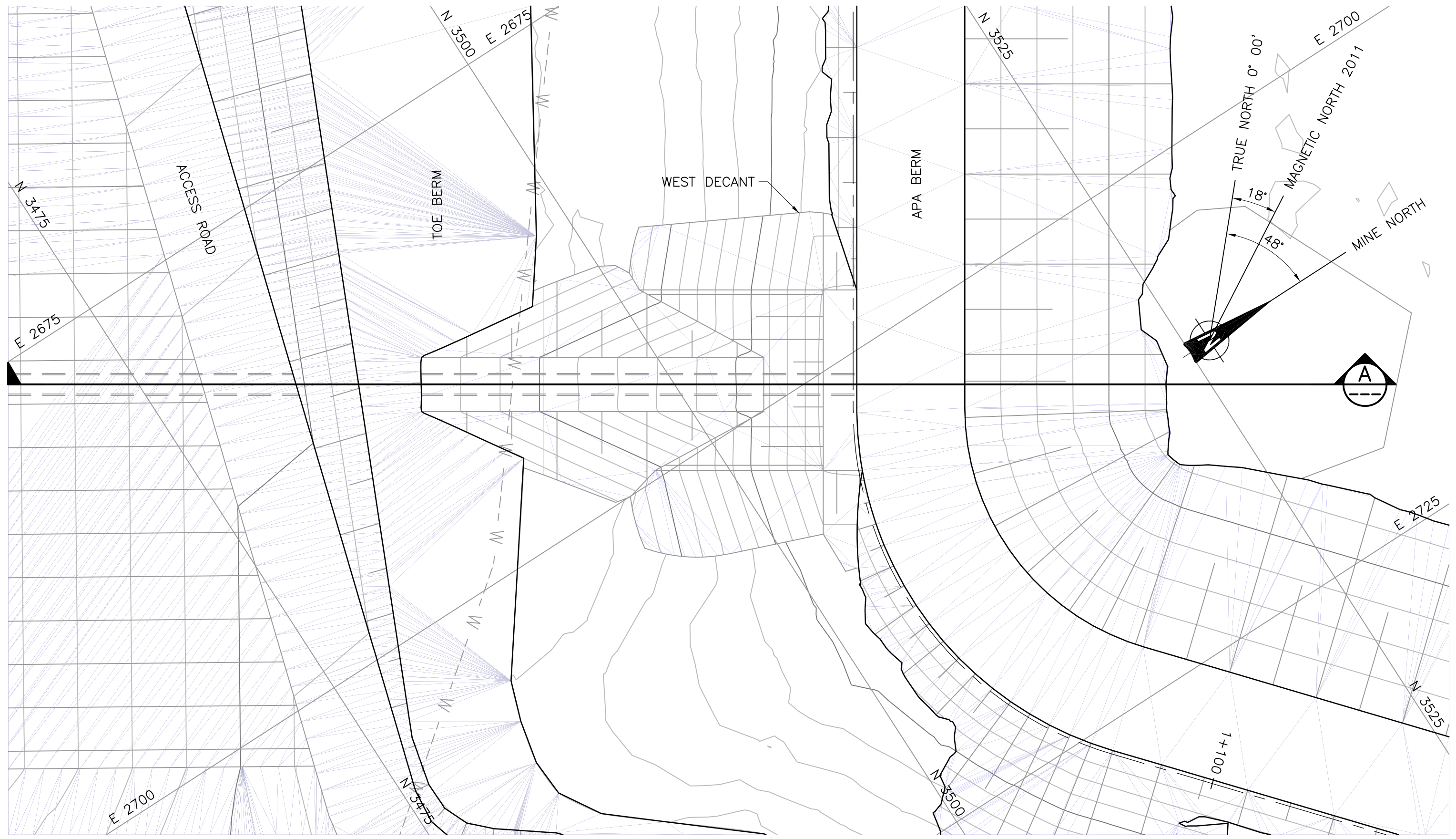

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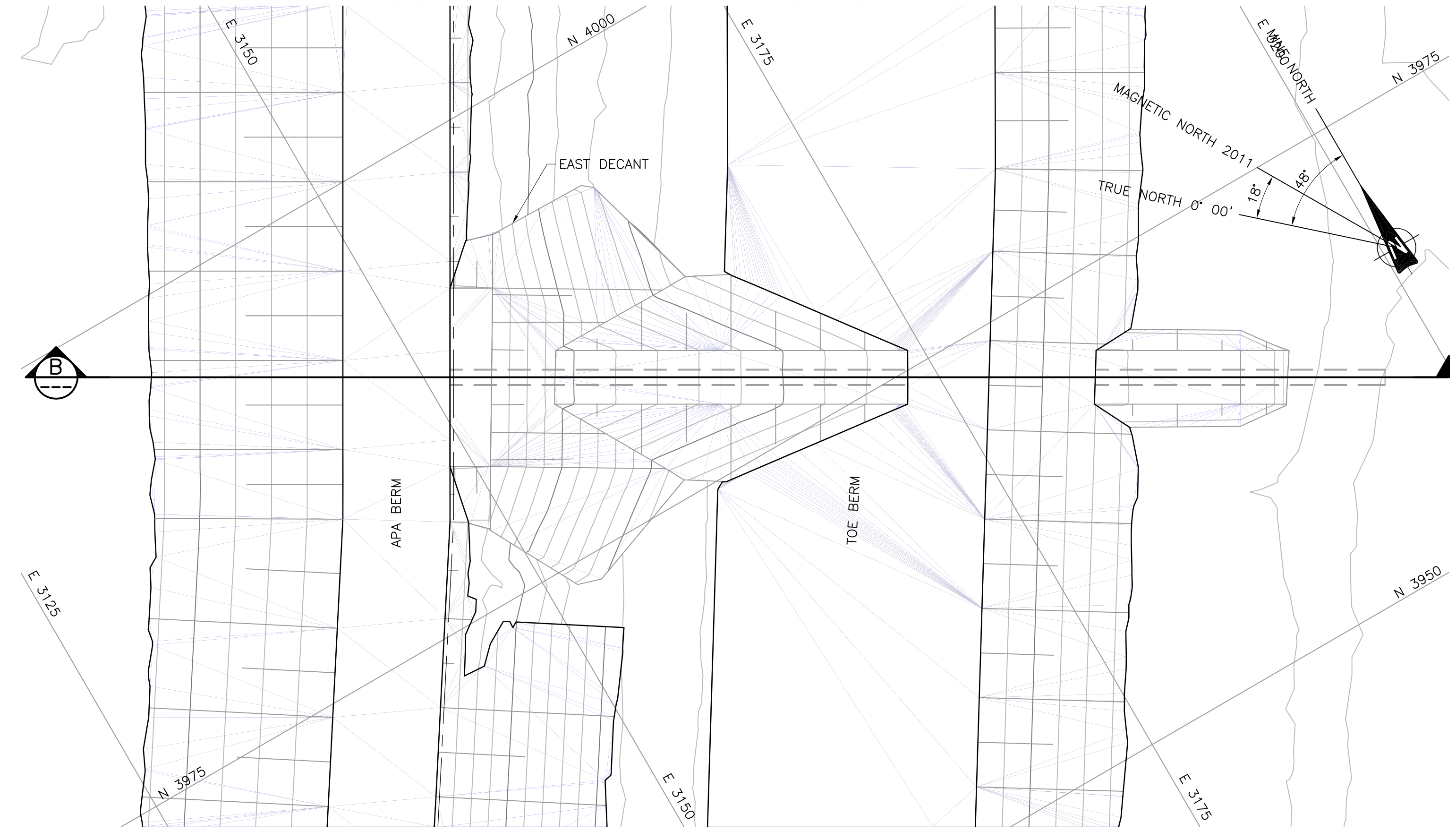

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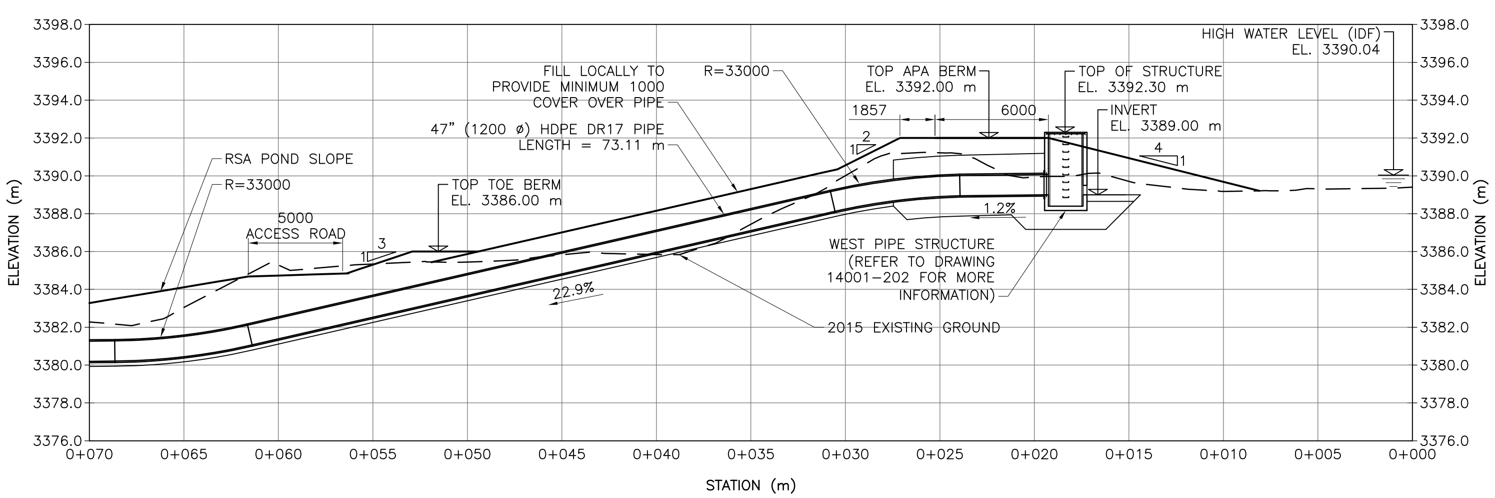

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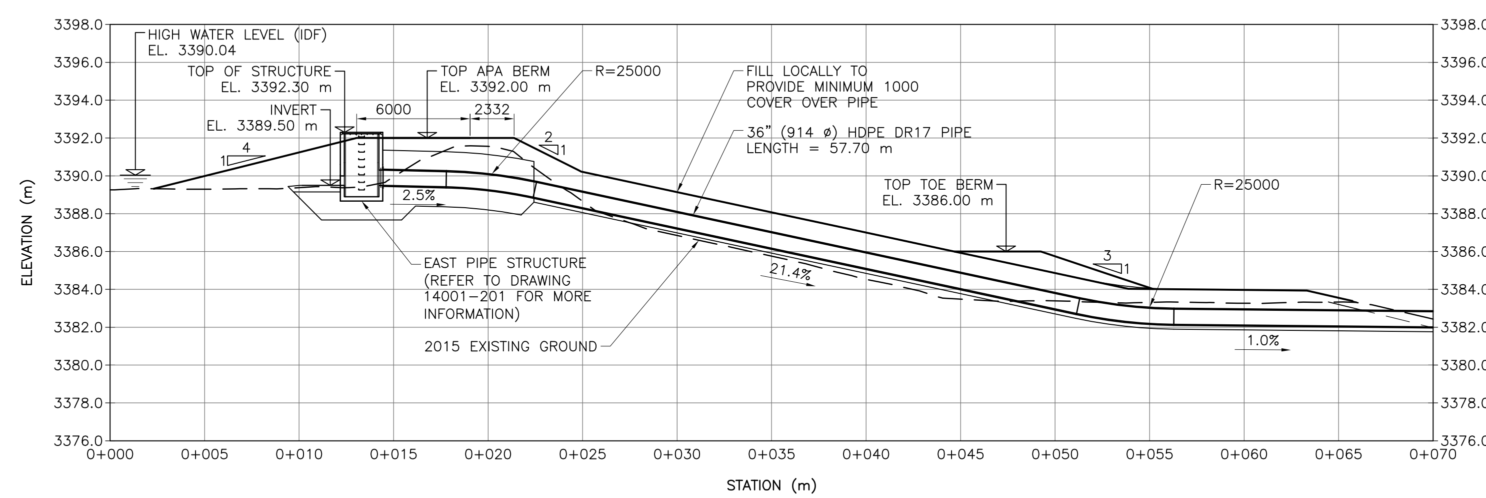

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Project: MYRA FALLS MINE SITE APA BERM STABILIZATION

Title: WEST AND EAST DECANT DETAILS 1 AND 2

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