A Case Study: Seismic Stability of an Upstream Tailings Impoundment – Empirical Methods and Input Ground Motions

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GEOQuébec 2015
Challenges from North to South
Des défis du Nord au Sud

ABSTRACT
This paper presents the implications of the selection and treatment of input ground motions for the seismic stability evaluation of an upstream-raised tailings impoundment currently under construction. This impoundment contains niobium tailings, considered susceptible to liquefaction under seismic loads. Therefore a shell of compacted coarse tailings was built along the perimeter and is subsequently raised in the upstream direction with a decreasing width with height. An extensive monitoring program of the compacted shell, including CPTu and SPT testing, has been implemented to evaluate the compaction methods implemented and for quality control. The site lies in the seismic zone that produced the 1988 Saguenay earthquake (magnitude 5.9) and is capable of producing earthquakes with magnitudes as great as 7.5. The different methodologies and results are compared and discussed in the light of the empirical liquefaction assessment method.

RÉSUMÉ
Ce document présente les implications de la sélection et du traitement des sollicitations sismiques pour l'évaluation de la stabilité sismique d'un parc à résidus en construction selon la méthode amont. Ce parc contient des résidus (niobium), considérés comme sensibles à la liquéfaction sous des charges sismiques. Par conséquent, une digue en résidus grossiers compactés a été construite le long du périmètre et est ensuite rehaussée vers l'amont avec une largeur diminuant avec la hauteur. Un vaste programme de contrôle de cette digue comprenant des essais CPTu et SPT, a été mis en place pour évaluer les méthodes de compactage mises en œuvre et à des fins de contrôle de qualité. Le site se trouve dans la zone sismique qui a produit le tremblement de terre du Saguenay en 1988 (magnitude 5.9) et pourrait produire des tremblements de terre avec des magnitudes atteignant 7,5. Les différentes méthodologies et les résultats sont comparés et discutés à la lumière de la méthode d'évaluation de liquéfaction empirique.

1 INTRODUCTION

Mine Niobec is a niobium mine located near Saguenay, Quebec. Tailings from ore processing are deposited in ring-type tailings impoundments. The first impoundment, Tailings Impoundment No. 1, operated from 1975 to 2005. A second impoundment, Tailings Impoundment No. 2 (TSF2), has been in operation since August 2003. TSF2 has plan dimensions of 650 m by 1250 m and will store approximately 30 M tonnes of tailings at its ultimate height of 30 m and is the subject of this paper. A plan view and the site is shown on Figure 1.

The key elements of the impoundment design are an exterior shell of compacted coarse tailings and an internal drainage system. The shell of compacted coarse tailings was initiated by the construction of two parallel starter dykes, spaced 60 m apart, along the perimeter of the impoundment. The starter dykes are composed of compacted coarse tailings and granular erosion protection was provided on the downstream face of the exterior starter dyke. The area between the starter dykes was then filled with compacted coarse tailings. The impoundment is being raised in the upstream direction with the width of the compacted shell decreasing with height. The downstream slope of the compacted shell is generally 4H:1V. Fine tailings and non-segregated tailings slurries are placed upstream of the shell. A plan view and a typical section of Tailings Impoundment is shown on Figure 2

The drainage system consists of finger drains (perforated pipes surrounded by sand) and French drains (sand only) that extend between the parallel starter dykes alternating at intervals of 23 m. The drains are connected to collector pipes at the upstream toe of the exterior starter dyke. The seepage is then conveyed through the exterior starter dyke using pipes to a collection ditch at the toe of the downstream slope of the impoundment. Decantation towers are used to manage the water inside of the impoundment and recirculate it to the concentrator.

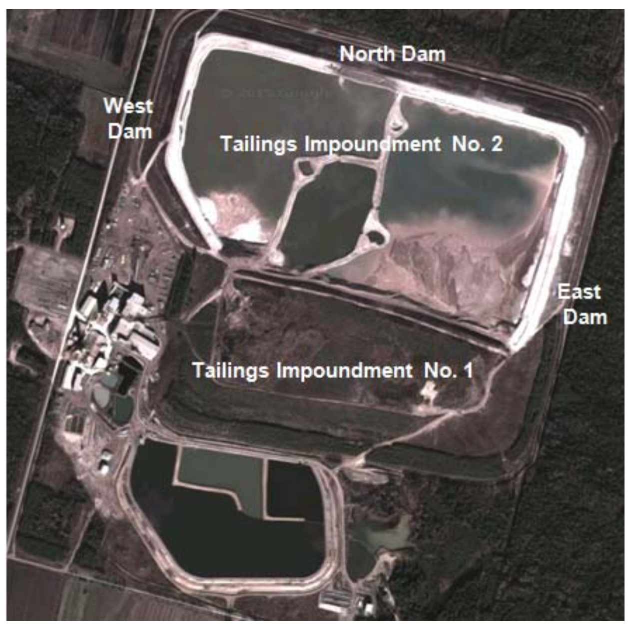


Figure 1. Plan view of Niobec tailings impoundment.

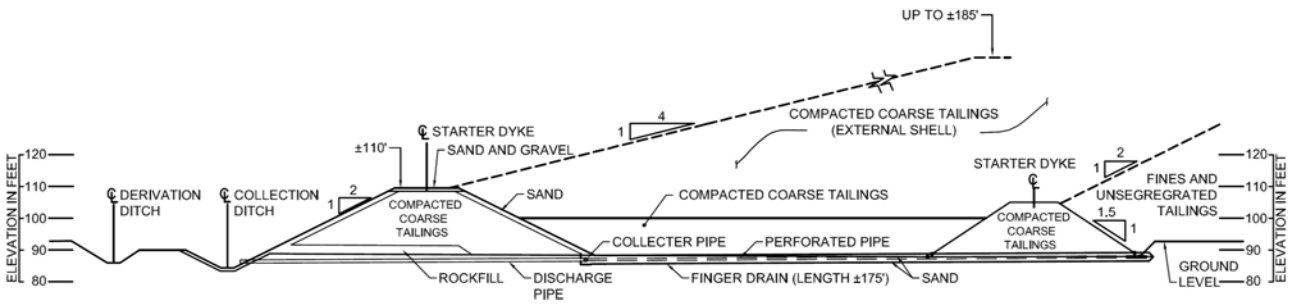


Figure 2. Typical Section of Tailings Impoundment No. 2 (Not to Scale)

The purpose of the exterior shell and the internal drainage system is to improve the static and seismic stability of the impoundment by creating a zone of high shear strength and liquefaction resistance, and inhibiting the development of seepage pressures in the downstream slope of the impoundment as shown on Figure 2.

1.1 Niobium Tailings properties

The primary physical characteristics of coarse tailings used to construct the compacted shell are presented in Table 1. Niobium coarse tailings are cohesionless and vary in gradation from silt- to sand-size particles and could thus be susceptible to liquefaction under seismic loads when loose and saturated.

Table 1. Coarse Tailings Properties

Properties	Units	Results
Dry Density	kN/m^3	15.4 - 23
Water content[1]	%	4 24
Standard Proctor	kN/m^3	16.8 – 17.6
Modified Proctor	kN/m^3	17.9 – 18.5
Relative Density (D_r) of grains	-	2.93 – 3.04
Permeability (in situ)	cm/s	3x10^{-4} to 7x10^{-4}
Permeability (laboratory)	cm/s	4x10^{-5} to 6x10^{-4}

2 COMPACTED EXTERNAL SHELL - IN SITU TESTS

2.1 CPTu and SPT Testing

An annual investigation program for the quality control of the compaction of the shell is in place since 2007 and was described in Lemieux et al (2011). It includes CPTu with shear wave measurement and SPT testing. Typical results (q_{c1N} and V_{S1}) are shown on Figure 3 for a cross section of the East Dam.

Generally, the compacted coarse tailings are characterized by q_{c1N} values greater than 160 and $(N_1)_{60}$ values greater than 35; the tailings at the base of the dam include layers of less compacted material and are characterized by q_{c1N} values between 75 and 130 associated with $(N_1)_{60}$ values between 16 and 25. The tailings deposited upstream of the compacted shell are characterized by q_{c1N} values between 50 and 80 associated with $(N_1)_{60}$ values between 4 and 25. Locally, the upstream border of the compacted shell shows values lower than q_{c1N} 160. It should be noted that no fines correction is applied to these normalized index, q_{c1N} and $N_{1(60)}$, as it was shown by James (2009) and others that this correction can be unconservative for tailings.

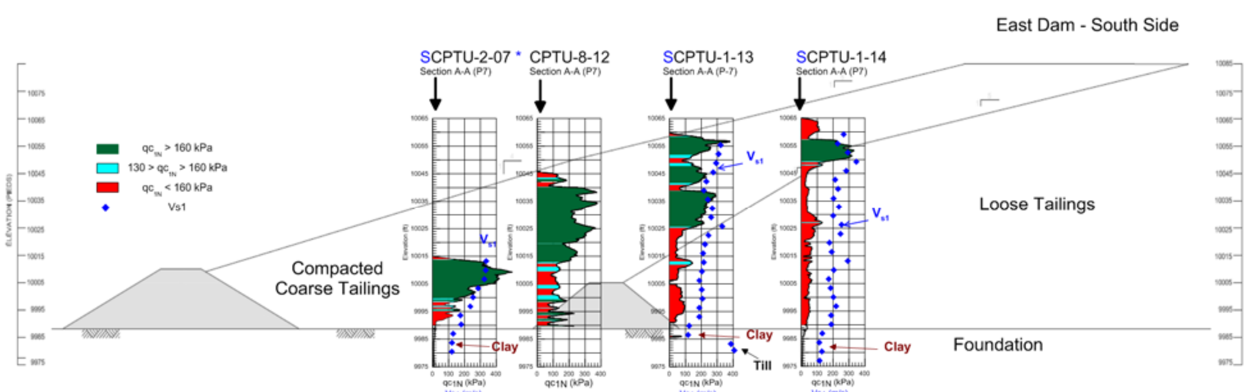


Figure 3. East Dam typical cross section with q_{c1n} and V_{S1} profiles from various site investigations.

3 LIQUEFACTION ASSESMENT

The Canadian Dam Association (CDA, 2013) recommends evaluation of the potential for liquefaction in stages beginning from the simple and conservative methods to more complex and precise methods. The Simplified method (Seed and Idriss, 1982; Youd et al., 2001, Idriss and Boulanger, 2008 and Seed, 2010) is widely accepted in practice and was the first step of our 2014 liquefaction evaluation.

3.1 Previous Studies – Quality control criteria

In 2010, based on 2007-2010 investigation results, qc1N from CPTu tests was selected as a compaction quality control criterion as indicated on Table 2. At that time, the design earthquake had a return period of 1:1,000 years. Analyses conducted with the Simplified method indicated that the CSR imposed by design earthquake would not exceed 0.28 and therefore tailings with corrected CPT tip resistance above 130 would not liquefy under the design earthquake, as illustrated on Figure 4.

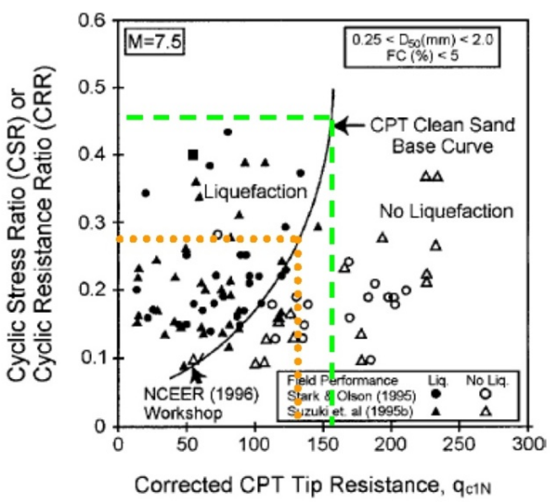


Figure 4. Criteria developed for the evaluation of the liquefaction resistance of the compacted coarse tailings (adapted from Youd et al, 2001)

Table 2. Liquefaction Criteria based on q_{c1n}

Criteria	Liquefaction Potential Assessment
qc1N > 160	Non-liquefiable tailings;
130 < qc1N < 160	Non-liquefiable tailings under earthquake of 1:1,000 years;
qc1N < 130	Tailings potentially liquefiable (more detailed analysis required).

On the typical cross section presented of Figure 3, the red zones within the profiles identify the potentially liquefiable zones within the tailings dam based on the criteria mentioned on Table 2. This criterion has been applied since then for the quality control of the compacted external shell and as reported in James et al (2011), non-linear dynamic response analyses have indicated that the risk of instability was extremely low for the 1:1,000 years seismic event.

3.2 2014-2015 Study

However, recently a new design earthquake with a return period of 1:2500 years was selected based on the revision of the dam classification based on the CDA Guidelines and Bulletins (2013 and 2014) and to meet the requirement of the MDDLCC Directive 019 (2012; even if TSF2 is not formally subjected to the 2012 revision). Therefore, a new study was undertaken to review the compaction criterion and assess the tailings liquefaction potential under the new seismic design.

Again, the liquefaction potential of the coarse tailings was assessed using the Simplified Seed-type methodology were the imposed Cyclic Stress Ratio (CSR) is compared to the Cyclic Resistance Ratio (CRR). This time, the simplified liquefaction assessment method was upgraded and, as proposed by Youd et al (2001), CSR was estimated by performing a sequence of One-dimensional equivalent-linear dynamic response analyses (CDA 2007 Level IV analysis).

3.3 Seismic Parameters

As recommended by the CDA (2013) Guidelines, the mean values of Spectral Acceleration (SA) and Peak Ground Acceleration (PGA) were used to develop the Uniform Hazard Spectrum (UHS). This task was carried out by Atkinson and Assatourians (2014) based on the seismologic data and methodology developed for the preparation of the 2015 NBCC Seismic Hazard Maps currently in preparation. These seismic models are the most up-to-date and they have been developed jointly by Prof. Atkinson and the Geological Survey of Canada Seismologists.

The computations results are presented in Figure 5 for probability levels between 1:1000 to 1:10,000 per annum probability for class A (Hard rock). For the 1:2500 p.a. UHS, the data were also deaggregated to identify the representative properties (magnitudes and distances) of the events contributing most to the hazard. Based on the deaggregation results, the two target scenarios of Table 3 were identified.

Table 3. Target Scenarios from Deaggregation

Magnitude (Mw)	Distance	Husid Duration[1]
6.5	32 km	11 s
7.0	50 – 70 km	20 s

[1] Husid duration is defined as the time interval between the points at which 5% and 95% of the energy in a ground motion have been delivered

3.4 Representative Input Motions

As recommended by CDA (2013), a multiple set of acceleration time histories was selected to represent the design earthquake ground motions, especially since simple scaling of natural earthquake records was used.

Simple uniform scaling has the advantage that the scaled time histories are natural, preserving the peaks and troughs in the response spectra of the recorded time histories (Stewart et al. 2001). However, the simple scaling has some setbacks as in eastern North America where there is a lack of records with appropriate frequency content, particularly for rock sites. Therefore, representative ground motions were selected from the local Saguenay 1988 (M5.9) earthquake and from North America earthquake databases as shown on Figure 6.

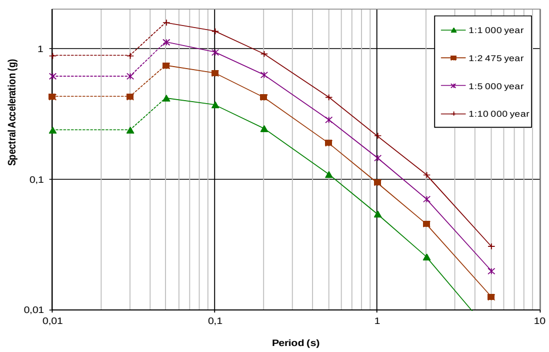


Figure 5. Mean-hazard horizontal-component UHS (5% damped) for NEHRP A Site Conditions (Hard Rock)

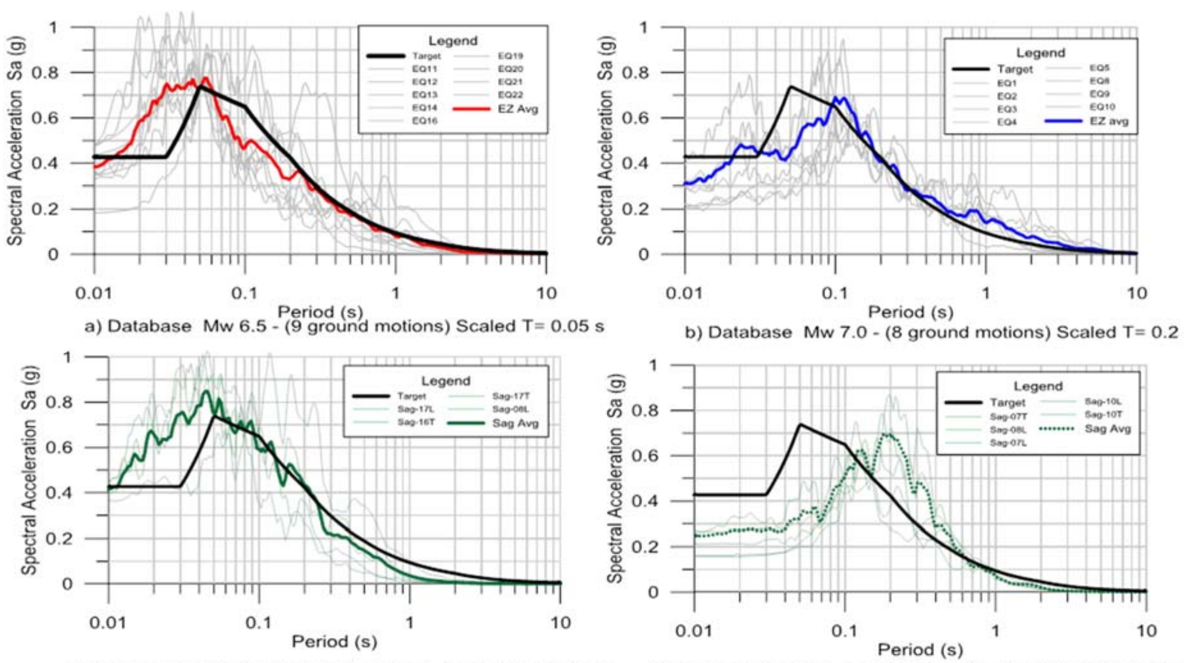
3.5 Database Ground Motions (Mw 6.5 and 7.0)

The databases were queried using the Spectral Matching module of the EZ-Frisk software (Fugro, 2011) to select input motions representative of the two target scenarios identified in Table 3. According to CDA 2007 the selection of records having appropriate magnitudes is important because magnitude strongly influences frequency content and duration of ground motion. It is desirable to use earthquake magnitudes within 0.5 magnitude units of the target magnitude. Selection of records having appropriate distances is also important especially for near-fault sites, because the characteristics of near-fault ground motions differ from those of other ground motions. However as stated by Atkinson (2014), faults mapped on the surface in eastern Canada were formed hundreds of millions of years ago, and may bear little relation to current seismic activity.

The Mw 6.5 ground motions represent relatively near seismic events and as illustrated on Figure 6a, they have been scaled to fit the high frequency (T=0,05 s) domain of the design spectrum. The Mw 7.0 ground motions represent more distant seismic events and have been scaled to fit the intermediate frequency (T=0,2 – 0,5 s) domain of the design spectrum (see Figure 6b).

3.6 Saguenay 1988 Ground Motions

The M5.9 Saguenay 1988 earthquake occurred in the Saguenay region where the site of Mine Niobec is located. The epicenter was about 60 km south of the mine site and therefore some records were included even if the magnitude is lower than the target magnitude of 6.5 and the distances of the recording stations are greater than the target distance of < 32 km. The records were selected out of the 21 available and scaled to fit different parts of the 1:2475 annual probability design spectrum at periods T=0.08 and T= 0.15 s ,as shown of Figure 6 (c and d). In general, the Saguenay recordings presented a better fitting when scaled at T=0.08 s, but not as well when scaled at longer periods.



a) Database Mw 6.5 - (9 ground motions) Scaled T= 0.05 s
b) Database Mw 7.0 - (8 ground motions) Scaled T= 0.2 s
c) Saguenay Mw 5.9 - (4 ground motions) Scaled (T = 0.08 s)
d) Saguenay Mw 5.9 - (5 ground motions) Scaled (T = 0.15 s)

Figure 6. Ground Motions selected to fit the target response spectrum for the 2:500 yr return period.

3.7 Selected Earthquake ground motions.

In general, the earthquakes selected from the databases were within their corresponding targeted distance; however, they presented higher durations than required. The Saguenay selections were within the prescribed duration, but the majority of them corresponded to longer distances. The original magnitude (Mw), distance (km) and duration (s) of the selected ground motions are illustrated on Figure 7.

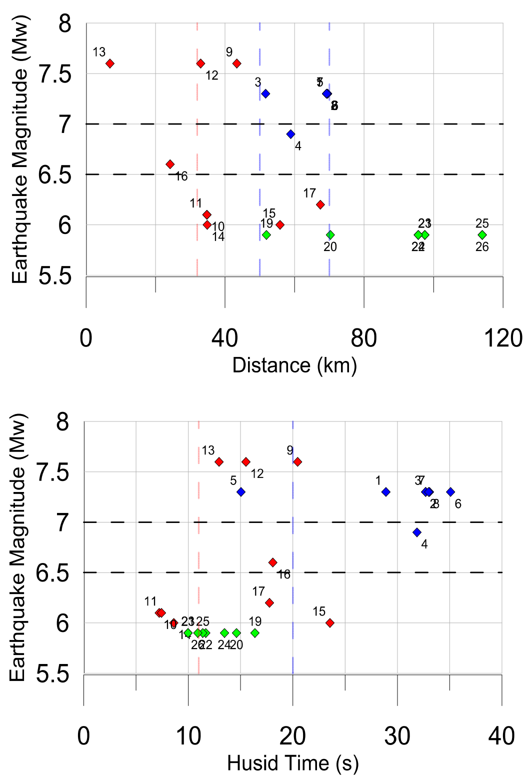


Figure 7. Ground motions selected in function of (a) distance (km) and (b) Husid duration (s).

4 1-D EQUIVALENT LINEAR ANALYSIS

In this CDA 2007 Level IV liquefaction assessment, one-dimensional (1D) equivalent-linear dynamic response analyses were then performed using the Proshake program developed by EduPro Civil Systems inc. (2003). The equivalent-linear model assumes that the shear modulus and damping ratio are functions of shear strain amplitude. Constant values of shear modulus and damping ratio are determined by iterations so that they become consistent with the level of strain induced in each layer. This model does not generate excess porewater pressure. These analyses provided the t_{max} vertical profile that was then used to estimate the CSR profiles imposed by the design earthquake. The following sections will describe the site conditions and the material properties used to perform the analyses.

4.1 Stratigraphy and Foundation properties.

There is a variable subsurface condition under the footprint of the tailings facility. The foundation varies from thick peat and clay deposits to glacial till foundation. These variations on the subsurface condition might have an effect on the seismic response and behavior of the overlying tailings dam section. Therefore two representative subsurface conditions were considered, 1) “Thick Clay” foundation and 2) “Glacial Till” foundation. Based on recent (2013-2014), but limited shear wave velocity Vs measurements, the normalized shear wave velocity V_{s1} was estimated in the order of 400 m/s for the till foundation and of 110 m/s for the clay deposit.

The shear modulus reduction curve and damping curve were generic curves: for the clay deposit, the curves developed by Vucetich and Dobry (1991) for a plasticity index Pi = 40 were used; and for the till, the gravel curves developed by Seed (1984) were selected.

The bedrock was encountered at depth of 0 to 30 m. A typical value of V_S of 2500 m/s was assigned to this stratum The properties of the bedrock were assigned to the bottom half space were the ground motions were applied.

4.2 Tailings properties - Shear Wave Profiles

Shear wave velocity profiles, V_{s1} were measured in 2007 at the compacted shell between the starter dykes and recently during the 2013 and 2014 campaigns at the dam crest. Figure 8 presents the typical Vs1 profiles estimated during the 2013 and 2014 site investigations. Representative values of normalized shear wave velocity, V_{s1}, were selected: in general, a value of V_{s1} 300 m/s was estimated in the compacted coarse tailings although the compacted shell at the lower slope had V_{s1} values near 350 m/s; in the medium compacted coarse tailings, Vs1 varied between 270 and 200 m/s with an average of 230 m/s; and in the spigotted loose tailings, it was between 150 and 200 m/s. This typical V_{s1} profile was used in the 1D analyses together with the average shear modulus reduction and damping curves for sand developed by Seed and Idriss (1970).

4.3 Representative Cross-Sections and 1D Vertical Profiles

As noted above, two cross-sections of TSF no 2 were selected to represent the subsurface conditions: 1) “Thick Clay” foundation and 2) “Glacial Till” foundation. For each representative section of the tailings dam, three 1D profiles were analyzed: actual (2014) crest (elevation 10065 ft), mid-slope and lower-slope profiles.

4.4 Water table within the tailings dam

The TSF2 is equipped with electric piezometers to monitor the water pressure in the tailings dams and, under the east dike, in the clay foundation. These piezometers are read periodically by Niobec to continuously monitor the ground water level within a certain alert threshold. Three piezometer series are located at the base of the downstream slope approximately, at the toe (Serie A), below mid-slope (Serie B) and below actual (2014) crest (Serie C). Figure 9 presents the piezometric levels measured since 2006 at Series C for various locations along the perimeter of TSF 2.

For the dynamic response analyses, the water table within the three 1-D profiles was estimated based on seepage analyses calibrated on the piezometric readings.

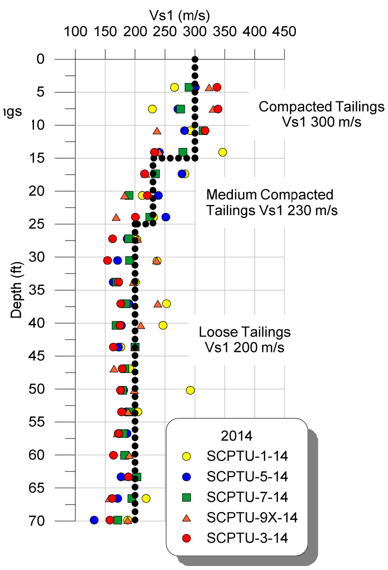


Figure 8. Normalized shear wave velocities profiles meausred at the actua crest of the TSF2.

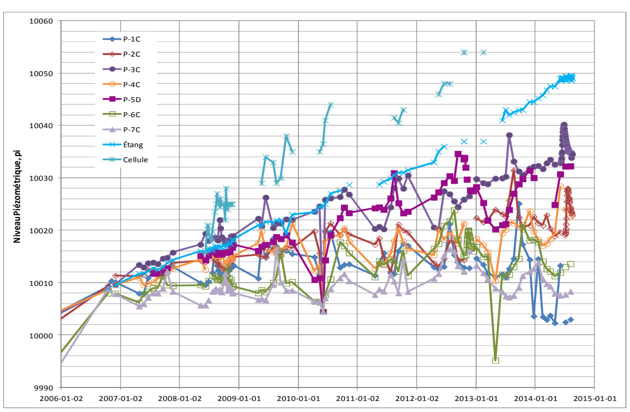


Figure 9. Ground water levels from piezometers (Series C) installed at the downstream slope near the actual crest.

5 RESULTS OF SITE RESPONSE ANALYSES

The results of the 1D equivalent-linear analyses will be illustrated for the 1-D profile corresponding to the actual 2014 crest (EL 10065 ft). The approximate location of this profile is sketched on Figure 10(a). Two cases with till (Figure 10) and clay (Figure 11) foundations will be compared.

Figures 10 and 11 present typical q_{c1n} and V_{s1} profiles for the tailings and the foundation, and the corresponding V_S profile. The q_{c1n} profile was used to calculate the normalized cyclic resistance ratio $CRR_{7.5}$ using the Youd et al (2001) and Idriss and Boulanger (2008) methods for comparison, see plot (d). The normalized cyclic stress ratio ($CSR_{7.5}$) for each scaled signal is also presented on the same figure. The CSR profiles were normalized using the magnitude scaling factor MSF recommended by Seed & Idriss (1982). Note that for presentation, a constant CRR of 0.5 was plotted for all q_{c1n} > 160, see Figure 4.

As described on the figures legend, the blue lines correspond to the group target Mw 7.0; the red lines for Mw 6.5 and the green lines to the Saguenay earthquake signals. The thicker lines represent the signals that produced the highest $CSR_{7.5}$ profiles for each group. For example, on Figure 10(d) for the till foundation case, the maximum $CSR_{7.5}$ observed was at the crest with an upper bound value of 0.3. At the transition zone CSR values below 0.2 were observed.

The factor of safety profiles presented on plot (e) was calculated based on the highest CSR for each group and the typical CRR profile. It is worth to note that the FS_{LIQ} of the loose tailings is less than 1, which means they are potentially liquefiable when saturated and since it is near 0.5, it could readily liquefy during the seismic loading.

On the transition zone inside the compacted shell with a q_{c1n} value of 130, it was found that the FSliq ~1 when compared to the upper bound of the CSR profiles. This value is marginal but this zone is above the water table and might not undergo a development of excess pore water pressure.

On the upper compacted shell, a q_{c1n} value of 230 was selected as a lower bound given that in general this zone has higher values. The FS_{LIQ} >1 for this zone only indicates that the strength of this area is beyond the charts available. However, given its high density and low confine stresses, this zone would undergo dilative behavior and at most would experience cyclic mobility with limited displacement.

For the 1-D profile with till foundation, an amplification of the acceleration profile was observed through the till foundation and damping through the loose tailings deposit following a slight amplification towards the crest, see Figure 10(g). In general the response spectrum was amplified between the period T=0.5 -1.0 s as shown on figure 10(h).

For the case with clay foundation there was a notorious damping of the acceleration through the clay deposit, with a resulting $CSR_{7.5}$ profile with values below 0.2. The response spectrum was amplified between the period T=0.8 -1.5 s.

On figure 11(e) the factor of safety against cyclic softening for the Idriss and Boulanger (2008) method is presented for the three group of signals analyzed. The factor of safety was calculated based on the higher peak shear stress profile for each group. The FS against cyclic softening of the cleat for the group of Mw 7.0 was the lowest and it was approximately 1.2.

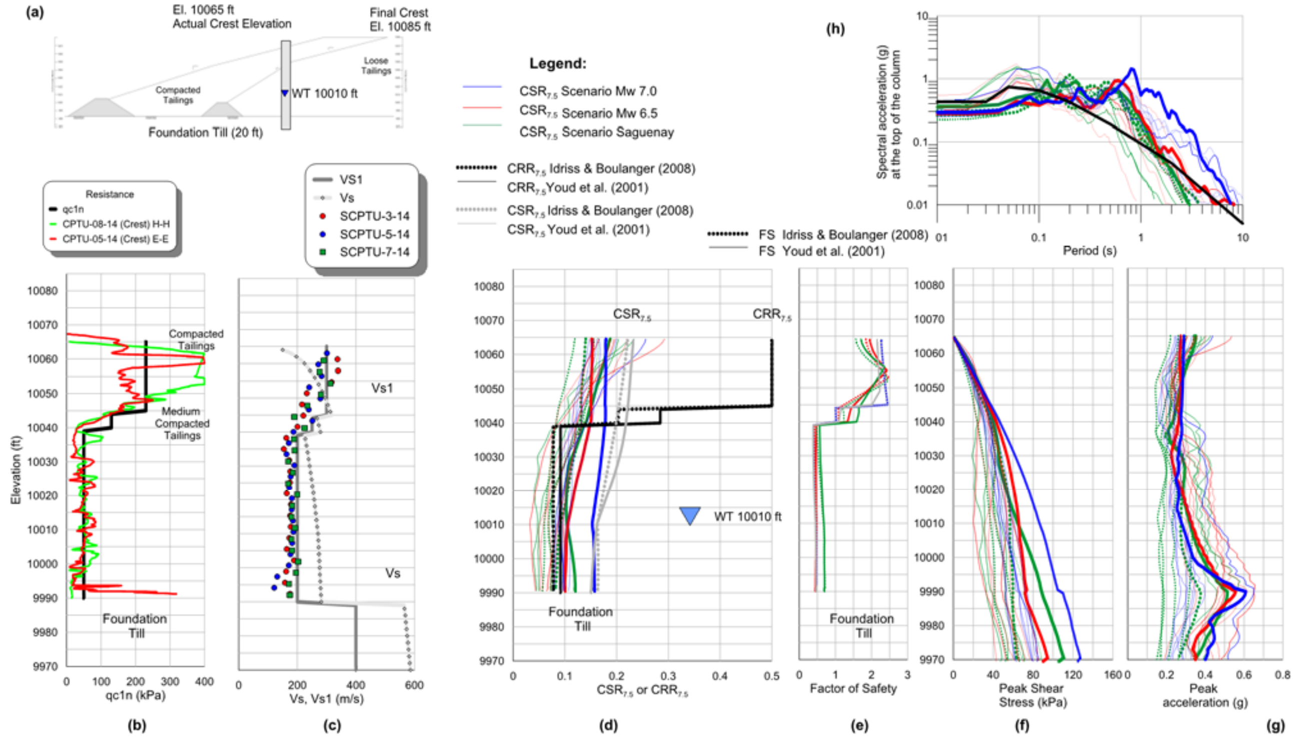


Figure 10. Site response analysis for the 1-D profile at the actual crest (El. 10065 ft) and for the till foundation.

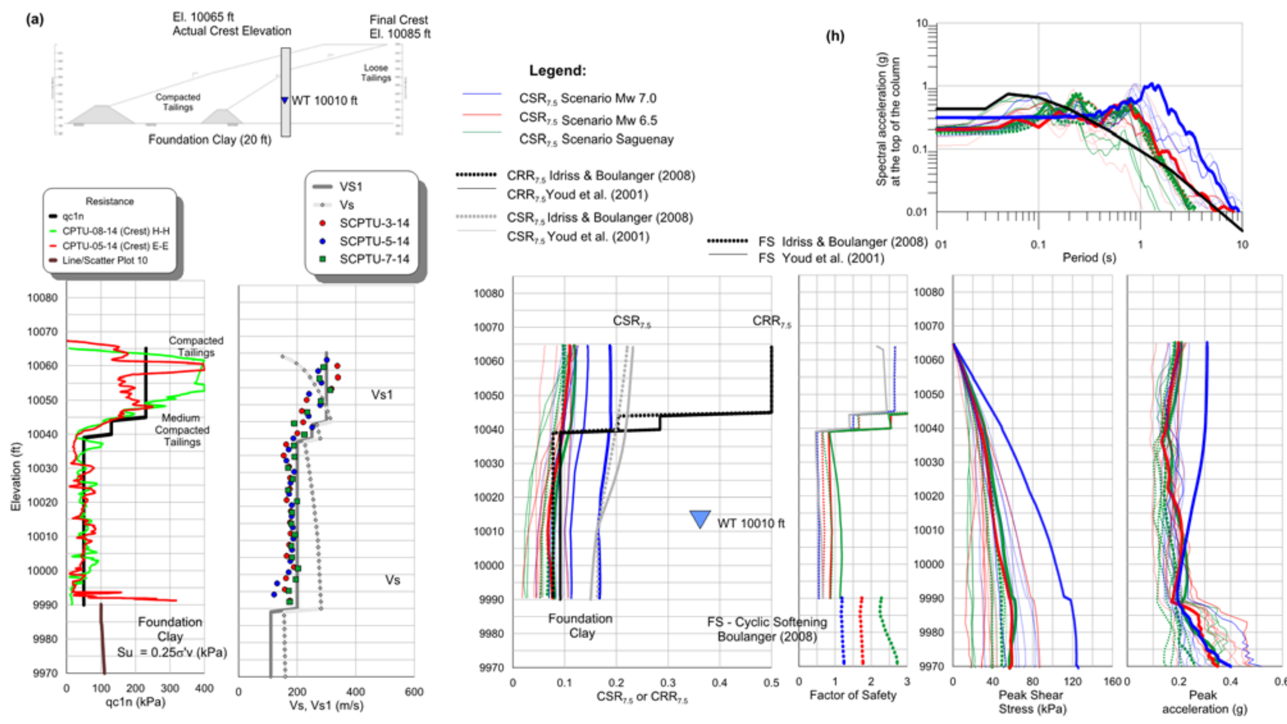


Figure 11. Site response analysis for the 1-D profile at the actual crest (El. 10065 ft) and for the clay foundation.

6 SIMPLIFIED R_D METHOD

For comparison, $CSR_{7.5}$ profiles calculated using the so-called simplified r_d method were also shown on figures 10 and 11 using Youd et al (2001) and Idriss and Boulanger (2008) methodologies. The maximum horizontal acceleration at the ground surface can be estimated using amplification factors proposed by Atkinson and Assatourians (2014) for Class C (Vs30 of 450 m/s) and by Boore and Atkinson (2008) for Class D (Vs30 of 250 m/s). Differences between the two profiles are due to the different methodology for calculating the stress reduction coefficient, rd, which accounts for flexibility of the soil (Youd et al 2001); and on the magnitude scale factors (MSF) used to normalize the CSR_M calculated to $CSR_{7.5}$.

In general, the simplified methods produced higher cyclic stress ratio profiles given that the stress reduction coefficient, r_d, does not capture the damping produced by the loose tailings deposit and the clay foundation for example.

Cetin (2004) proposed a new methodology to calculate rd in function of the stiffness, Vs, of the soil profile and other seismic parameters. Figure 12 presents a comparison between the Youd et al (2001), Boulanger (2008) and Seed (2010) methods which includes the methodology proposed by Cetin (2004) for the calculation of r_d based on a Vs_{12m} profile of the upper 12 m of the soil strata; and Moss (2006) for the estimation of CRR_M based on a probability of occurrence. For this case, a probability of 0.15 was used on the calculations. The three methods, Youd et al (2001), Idriss and Boulanger (2008) and Seed (2010) were applied to the analysis of CPTU-6-14, considered typical for the tailings profile below the actual crest elevation.

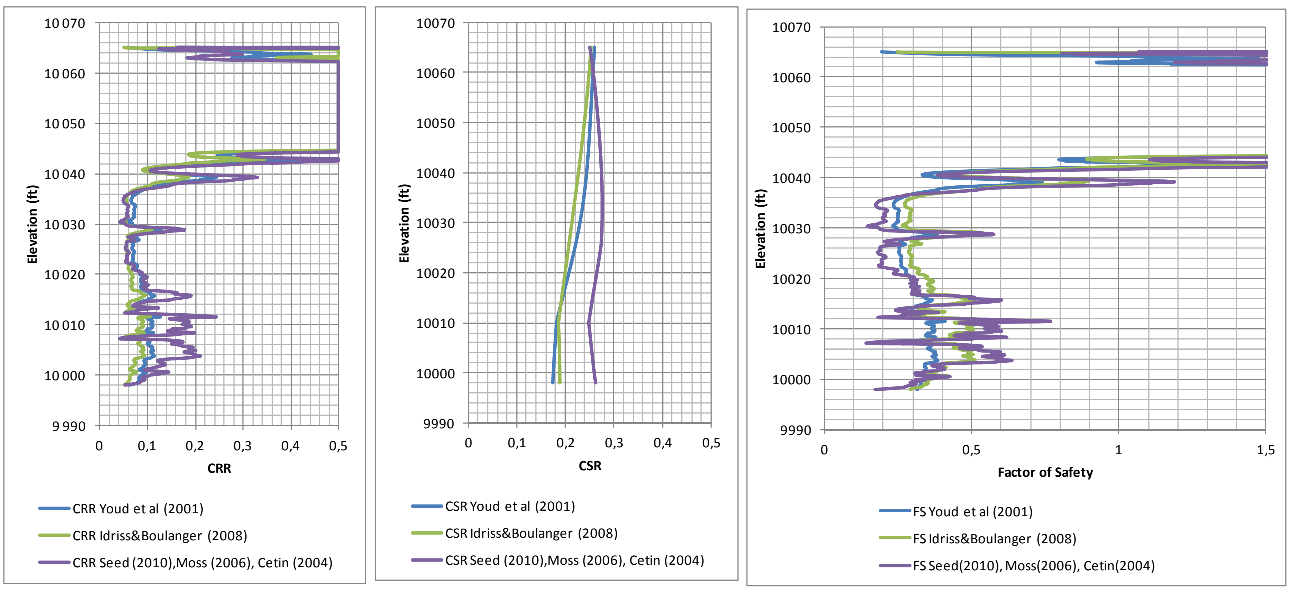


Figure 12. Youd et al 2001, Idriss &Boulanger (2008) and Seed (2010) methods for SCPTU-6-14 - Comparison.

7 DISCUSSION AND CONCLUSIONS

Based on the site response analysis, it was found that: earthquake signals that were scaled to match the response spectrum at longer periods produced the most significant peak shear stress profiles and therefore higher cyclic shear stress ratio (CSR) as calculated by the simplified method.

The presence of the clay foundation with lower VS profiles might produce damping and reduce the peak stress imposed to the column of soil simulated.

The simplified methods were found to produce CSR profiles located at the upper bound of the CSR profiles produced by the site response analysis. These methods are considered conservative. Youd et al (2001) and Idriss and Boulanger (2008) methods were compared to the Seed (2010) method which includes probability of exceedance, the stiffness of the soil profile to estimate the stress reduction coefficient and relevant seismic parameters (a_{max}, Mw, etc). The three methods led comparable results for the case analysed, as showed in the previous section.

The analysis of a group of signals and the comparison among different methods offers a broad understanding of the seismic response of any structure. The implementation of 1-D sites response analysis offers the opportunity to quickly assess several earthquake responses and to complement the semi-empirical methods available to assess the liquefaction potential. It is relevant to point out the limitations of the simplified method, which is valid for up to 15 m in depth and for flat terrain, and its applicability to tailings has been studied only in a limited manner by James (2009) and others.

As potentially liquefiable zones are identified through site exploration and analytical solutions, post-earthquake stability analyses will be carried out to assess the stability requirement of the TSF2, including the study of the clay deposit seismic response.

ACKNOWLEDGEMENTS

The authors acknowledge and appreciate the permission granted by Niobec inc. for the publication of this case study and for their assistance during the project.

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