A case study: The seismic stability of an upstream-raised tailings impoundment (part i)

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ABSTRACT
This paper presents the results of piezocone (CPT-V_s) testing and liquefaction analysis using a simplified evaluation method for an upstream-raised tailings impoundment currently under construction. The impoundment is used to contain tailings from the processing of niobium ore at Mine Niobec near the city of Saguenay in the province of Quebec, Canada. The impoundment was constructed by building two parallel starter dykes 60 meters apart along the perimeter, placing and compacting coarse tailings between the two starter dykes, and then raising the coarse tailings in the upstream direction with the width of the compacted section decreasing with height. This creates a relatively dense shell of liquefaction-resistant coarse tailings around the perimeter of the impoundment. The site lies in the seismic zone that produced the 1988 Saguenay earthquake (magnitude 5.9). This zone is capable of producing earthquakes with magnitudes as great as 7.5.

RÉSUMÉ
Cet article présente les résultats d'essais au piezocône (CPT-V_s) et de l'analyse du potentiel de liquéfaction (méthode simplifiée d'évaluation) d'un parc à résidus présentement en construction par la méthode de rehaussement amont. Le parc à résidus est utilisé pour entreposer les résidus provenant du procédé d'extraction du niobium de Mine Niobec près de la ville de Saguenay dans la province de Québec, Canada. Le parc à résidus mesure 650 m par 1250 m et a été construit par l'aménagement de deux digues de départ parallèles en résidus grossiers espacées de 60 m le long du périmètre, par la mise en place de résidus grossiers compactés entre les deux digues de départ et enfin, un rehausssement par la méthode amont à l'aide de résidus grossiers pour former une digue périphérique compactée dont la largeur diminue avec la hauteur. Le site est situé dans la zone sismique qui a produit le tremblement de terre du Saguenay en 1988 (magnitude de 5.9) et qui pourrait éventuellement produire des tremblements de terre de magnitude supérieure à 7.5.

1 INTRODUCTION

1.1 Primary Design Aspects

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, has been in operation since August 2003. Tailings Impoundment No. 2 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. Tailings Impoundment No. 2 is the subject of this paper.

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 4:1 (horizontal: vertical). Fine tailings and unsegregated tailings slurries are placed upstream of the shell. 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 collecter 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. A plan view and a typical section of Tailings Impoundment No. 2 are shown on Figures 1 and 2, respectively.

Liquefaction resistance is an important issue since 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 purpose of the compacted shell of coarse tailings around the perimeter of the impoundment and the drainage system underlying the shell are to provide a zone of relatively dense, unsaturated, liquefaction-resistant material to increase the seismic stability of the impoundment and control seepage.

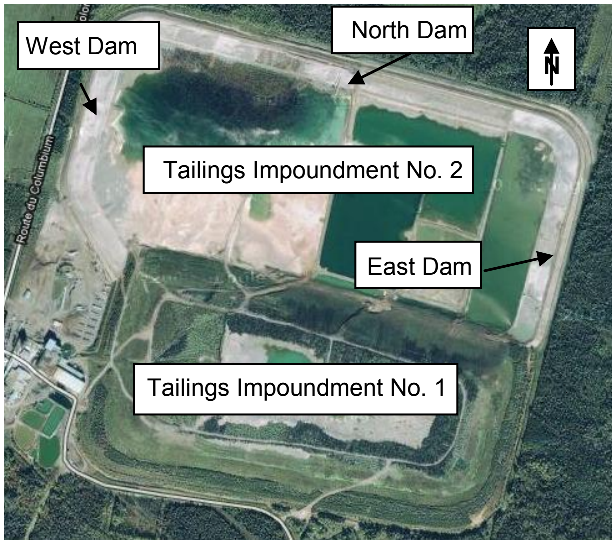

Figure 1 Plan view of Tailings Impoundment No. 2 (Not to Scale)

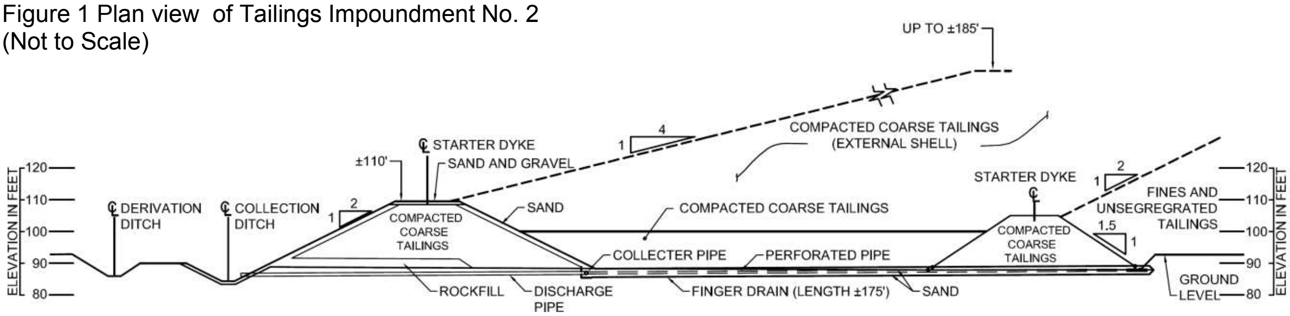

Figure 2 - Typical Section of Tailings Impoundment No. 2 (Not to Scale)

1.2 Methods of Tailings Deposition and Compaction

Initially, the coarse tailings forming the exterior shell, particularly in the west side of the impoundment, were deposited by spigotting between the French drains and finger drains. Compaction of the saturated tailings was accomplished using a 30-tonne backhoe with a load of waste rock in its bucket. After one year and visits to the Syncrude tailings site in Fort McMurray, Alberta and Highland Valley Copper Mine near Kamloops, British Columbia, the cell method was implemented. The cell method consists of placing coarse tailings slurry in a limited zone on the top of the compacted shell and using a bulldozer to spread and compact the tailings under a thin layer of water. The efficiency of this method is evident based on observations and in situ testing results. Figure 3 shows an example of compaction with this method.

From May to December, the shell of compacted coarse talings is raised and fine tailings slurry is placed upstream of the shell. During the Winter, from January to April, unsegregated tailings slurry is placed upstream of the compacted shell. The exterior shell is not raised during the Winter. Typical gradations of the fine and coarse tailings are shown on Figure 4.

Figure 3. Compaction of coarse tailings by the cell method.

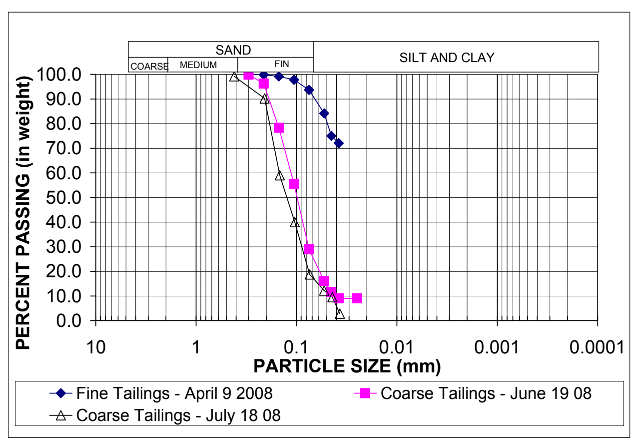

Figure 4 – Particle Size of Coarse and Fines Tailings.

2 IN SITU TESTING OF THE COMPACTED EXTERNAL SHELL

2.1 Compaction Criterion

The initial criterion adopted in the design to ensure sufficient liquefaction resistance of the compacted coarse tailings was a minimum density index, I_D, of 75% (Geocon, 2003). The corresponding dry density was 1682 kg/m³ based on the maximum dry density determined using the standard Proctor compaction test (ASTM D698).

2.2 Density Measurement by Nuclear-Density Gauge.

From 2003 through 2006, in situ tests were conducted using a nuclear-density gauge to measure the dry density of the compacted coarse tailings. Figure 5 shows the results of nuclear-density gauge measurements from 2003 to 2006. About 185 tests were carried out and 56 tests were below the minimum target dry density of 1682 kg/m³ (SLI, 2006).

These results indicated that the cell method generally resulted in satisfactory compaction of the tailings. However, the gradation of tailings is intrinsically variable and the results of nuclear-density gauge testing are very sensitive to these variations. Additionaly, using nuclear-density gauges, it was not possible to measure any increase in the dry density with depth as additional compacted tailings were placed above.

2.3 Piezocone Testing

The design report (Geocon, 2003) recommended periodic verification of the liquefaction resistance of the compacted coarse tailings using piezocone testing (CPTu) and evaluation of the seismic stability of the impoundment using analytical methods. Therefore, piezocone testing has been conducted annually since 2007 to evaluate the degree of compaction and liquefaction resistance of the compacted coarse tailings shell.

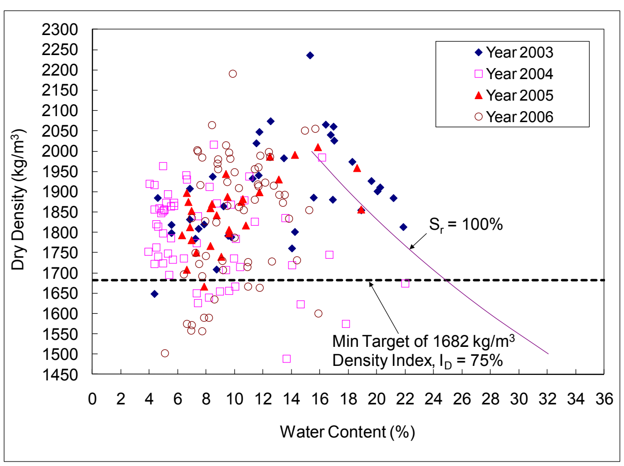

Figure 5 – Measured dry density of compacted coarse tailings of external shell (nuclear-density gauge) – Years 2003 to 2006

An initial geotechnical campaign was conducted in August 2007 and included piezocone tests with shear wave velocity measurements (CPTu-VS) that were calibrated using nearby SPT testing (Techmat, 2007). Additional campaigns were carried out in 2008 and 2009 (Qualitas, 2008 and 2009). The results of these tests are described below in the evaluation of the liquefaction resistance of the compacted coarse tailings.

3 LIQUEFACTION EVALUATION

3.1 Methodology

The Canadian Dam Association (CDA, 2007) recommends evaluation of the potential for liquefaction in stages beginning from the simple and conservative methods to more complex and precise methods, until an acceptable solution is found. The Simplified method of liquefaction evaluation (Seed and Idriss, 1982; Youd et al., 2001), widely accepted in practice, was the first step in the liquefaction evaluation for this project.

3.2 Summary of the Simplified method

To evaluate the liquefaction resistance by the Simplified method (Seed and Idriss 1982) as updated by Youd et al (2001), two parameters are necessary: 1) the seismic loading on the soil by the design earthquake, expressed as the cyclic stress ratio, CSR; and 2) the resistance of the soil to a given cyclic loading, expressed as the cyclic resistance ratio, CRR.

The CSR is estimated from the maximum horizontal acceleration at the ground surface (a_{max}), the initial stress state, and a stress reduction coefficient (rd) which is function of depth.

$$CSR = 0.65(a_{\max} / g)(\sigma_{vo} / \sigma_{vo}')r_d \quad \text{.....................................[1]}$$

where σ'_{vo} is the initial, effective vertical stress, σ_{vo} is the initial, total vertical stress, and g is the acceleration of gravity. The factor 0.65 is used to normalize the stress level based on observations by Seed and Idriss (1982).

The CRR is estimated based on corelations with in situ testing for a magnitude 7.5 seismic event, $CRR_{7.5}$. The in situ testing results must be normalized with respect to the testing equipment and methods, and fines content (percent of the soil by mass passing a 0.075 mm sieve). The correlation between the $CRR_{7.5}$ and the corrected piezocone (CPT) tip resistance recommended by Youd et al. (2001) is shown on Figure 6.

The factor of safety with respect to liquefaction is calculated as the ratio of the cyclic resistance ratio, CRR7.5 to the cyclic stress ratio, CSR.

$$FS=(CRR_{7.5}/CSR) \quad \text{..[2]}$$

However, correction factors must be applied to account for earthquake magnitudes other than 7.5, effective overburden stresses exceeding 100 kPa and the presence of initial static shear stresses (such as those below the slopes of dikes).

The correction for the magnitude is known as the magnitude scaling factor, MSF. MSF values recommended for general engineering practice are presented in Youd et al. (2001) and Aranago (1994) recommends specific values for the higher frequency earthquakes typical of North America east of the Rocky Mountains.

Based on laboratory research indicating that the liquefaction resistance increases with effective overburden stress, Youd et al. (2001) recommend a correction factor for effective overburden stresses exceeding 100 kPa, $K\sigma$, that varies with the density index of the soil. However, it should be noted that, research by Polito and Martin (2001) and James (2009) indicates that the liquefaction resistance of hydraulically deposited soils (such as slurry-deposited tailings) may be independent of effective overburden pressures as high as 400 kPa.

The magnitude of the initial static shear stress can have a significant effect on the liquefaction resistance. There are several initial static shear stress correction factors, $K\alpha$, presented in the literature. Youd et al. (2001) recommend these factors not be used by "non specialists in geotechnical earthquake engineering" or in routine engineering practice. Generally, the liquefaction resistance of dilative soils increases with increasing initial static shear stress and that of contractive soils decreases with increasing initial static shear stress (Youd et al., 2001).

Taking into account these correction factors, the factor of safety with respect to liquefaction may be calculated using:

$$FS=(CRR_{7.5}/CSR)\cdot MSF\cdot K_{\sigma}\cdot K_{\alpha} \quad [3]$$

The correlation presented in Figure 6 is for "clean sand" defined as sand without fines or where the penetration resistance has been corrected to eliminate the effect of the fines. However, research by Ulrich and Hughes (1994) indicates that this correction may not be applicable to tailings. Accordingly, the fines correction was not considered in the liquefaction evaluation of the tailings for this project.

The applicability of the Simplified method to tailings was demonstrated by James (2009). However, there are two important limitations of the Simplified method that are of concern for this project. Firstly, the method was developed for level ground conditions (no initial static shear stresses) and there is no consensus regarding its applicability in zones of relatively high initial static shear stresses, such as below the slopes of dykes. Secondly, the stress reduction coefficient and generic amplification factors (e.g. Finn and Wrightman, 2003) used in the analysis are not representative of the response of tailings impoundments to seismic loading (James, 2009).

3.3 Design Earthquake

The return period of the design earthquake was selected based on the requirements of the Québec Ministry of Natural Resources (MNR, 1997) and the recommendations of the Canadian Dam Association (1999, 2007). The return period used for liquefaction analyses was 1:1,000 years. The seismic parameters were provided by Earthquakes Canada (2007, 2008, 2009). These parameters are valid for conditions of firm soil, Class "C" (NBCC 2005). Based on shear wave velocity measurements during the 2007 campaign (Techmat, 2007), soil conditions underlying Impoundment No. 2 may be classified at the boundary between NBCC (2005) Classes "D" and "E". A generic seismic response was estimated using amplification factors proposed by Finn and Wightman (2003) on the basis of the stratigraphic data. Thus, the maximum horizontal acceleration at the ground surface (a_{max}) and the magnitude of the earthquake (M_W) estimated were respectively 0.37g and 7.0.

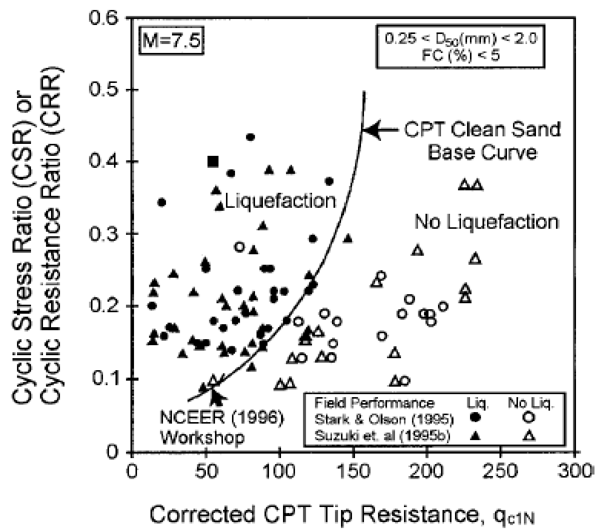

Figure 6 – Estimation of CRR based on corrected CPT tip resistance values (q_{c1N}) (from Youd et al. 2001).

3.4 Results of piezocone tests

Although several piezocone tests have been conducted on the compacted external dyke of Impoundment No. 2, it is important to note that to facilitate the understanding of the results of piezocone testing, only specific zones of the impoundment are referenced in this paper: CPTU-1 tests in the south zone of the east external dyke and CPTU-2 tests in the north zone of the east external dyke.

<u>2007</u>

In 2007, piezocone testing was conducted in the compacted shell of the impoundment. The uncorrected results of CPT Nos. CPTU-1-07 and CPTU-2-07 are presented on Figures 7 and 8. Analyses conducted using the Simplified method generally indicate that the coarse tailings compacted by the cell method will not liquefy in response to the the design earthquake (1:1,000 years). Calculated factors of safety of 5 or more indicate that the liquefaction resistance of the coarse tailings is greater than the upper limit of potentially liquefiable materials. CPTU-1-07 indicated that the liquefaction resistance of the upper 1.5 m (5 ft) of compacted tailings was relatively low possibly due to the low confinement. In 2008 and 2009, this zone of lower resistance disappeared with the addition and compaction of additional layers of material.

In tests CPTU-1-07 and CPTU-2-07 some zone of relatively low liquefaction resistance were encountered in the lower portion of the compacted coarse tailings corresponding to zones compacted before the implementation of the cell method. Theses zones have a thickness of about 0.3 to 0.6 m (1 to 2 feet) and could be susceptible to liquefaction or high excess porewater pressure development in the event of the design earthquake. Figures 9 and 10 show these zones that have factors of safety of about 1.0 or less.

Figure 9 presents the corrected CPTu tip resistance, q_{c1N}, $CRR_{7.5}$, $CSR_{7.5}$, and factor of safety with respect to liquefaction, FoS, for CPTU-1-07/08/09 and SPT F-1-07. It is interesting to observe that the mean values are about the same for CPT and SPT. However, the piezocone can detect relatively thin zones of less resistant material that cannot be detected by SPT. The SPT data corresponds to average resistance measured over a length of about 0.5 m (1.5 ft) must be compared to the average piezocone resistance over a similar height. Moreover, a perfect match is not expected since the tests are not performed at exactly the same location, and interpretation methods are empirical (see Figure 9).

2008 and 2009

The piezocone tests conducted in 2008 and 2009 in the compacted shell of the east dyke (CPTU-1-08, CPTU-1-09 and CPTU-2-09, see Figures 7 and 8 for uncorrected values) indicate that the coarse tailings compacted by the cell method will not liquefy due to the design earthquake (1: 1000 years). However, as mentioned for the 2007 results, some zones relatively low resistance were encountered, particularly at the base of the compacted shell (see Figures 9 and 10).

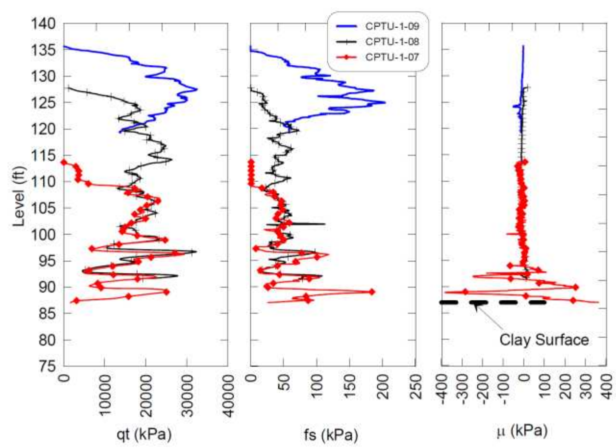

Figure 7 – Uncorrected Values of CPTU-1-07, CPTU-1-08 and CPTU-1-09 (East External Dyke, South Zone)

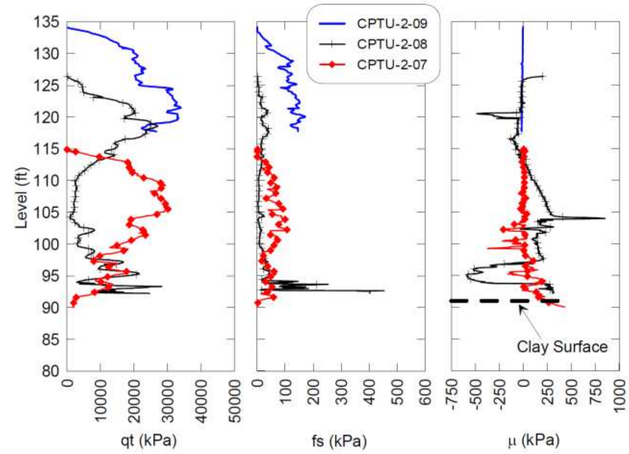

Figure 8 – Uncorrected Values of CPTU-2-07, CPTU-2-08 and CPTU-2-09 (East External Dyke, North Zone)

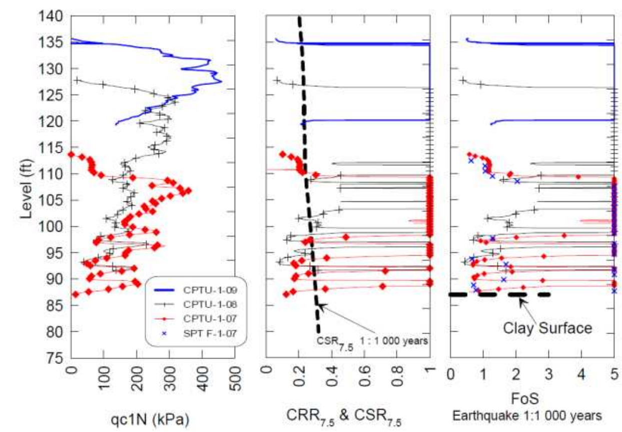

Figure 9 – Corrected Values with Results of Simplified Method (East External Dyke, South Zone)

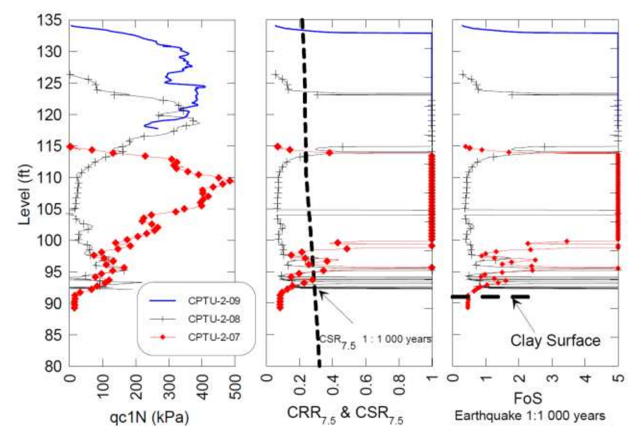

Figure 10 – Corrected q_{c1N} values with results of Simplified method, (East External Dyke, North Zone)

3.5 Criterion for quality control of compacted tailings

The analyses indicate that the criteria for quality control of compacted tailings can be developed on the basis of piezocone testing.

Indeed, Figure 11 presents the relationship between liquefaction resistance and corrected CPT tip resistance, qc1N. In this figure, the qc1N above about 160 (green dashed line), the materials are not liquefiable and can resist CRR7.5 values greater than 0.45. It has been shown that an acceptable level of liquefaction resistance can be achieved using the cell method to construct the external shell of the dykes.

Moreover, analyses conducted with the Simplified method indicate that the CSR imposed by the 1:1,000-year earthquake should not exceed 0.28 (orange dashed line on Figure 11) and therefore tailings with corrected CPT tip resistance above 130 will not liquefy under the design earthquake. However, the development of excess porewater pressure can be expected in zones where the tailings are saturated.

The following classification of the compacted coarse tailings based on these relationships was developed by the authors for this site:

- q_{c1N} > 160 : Non-liquefiable tailings;
- 130 < q_{c1N} < 160 : Non-liquefiable tailings under earthquake of 1:1,000 years;
- q_{c1N} < 130 : Tailings potentially liquefiable (more detailed analysis required).

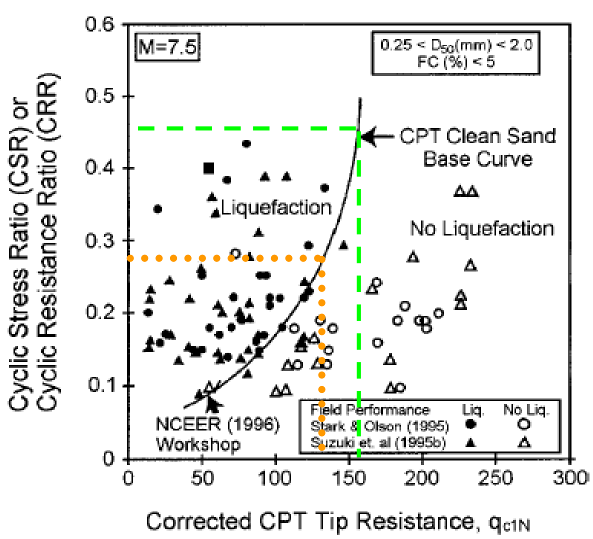

Figure 11 – Criteria developed for evaluation of the liquefaction resistance of the compacted coarse tailings.

4 DISCUSSION AND CONCLUSIONS

In the first years of the construction of Tailings Impoundment No. 2 (2003 to 2007), in-situ density measurements (nuclear-density gauge and sand cone) were conducted to evaluate the method and results of compaction of the exterior shell of coarse tailings. The results indicated that it is not possible to develop a reliable criterion for evaluationg the liquefaction resistance of the tailings because of significant variations in the test results related to their sensitivity to intrinsic variation in the gradation of the tailings.

Therefore, since 2007, piezocone testing (CPTu) has been used to evaluate the compaction method and liquefaction resistance of the coarse tailings. Piezocone testing has shown that the cell method performs well and generally creates non-liquefiable tailings. However, the liquefaction resistance of some relatively thin seams at the base of the compacted coarse tailings may not be adequate with respect to the design earthquake (1:1,000 years).

Criteria were developed based on the relationship between the liquefaction resistance and the corrected CPT tip resistance of the compacted coarse tailings. Values of corrected CPT tip resistance (qc1N) above 160 are considered non-liquefiable. Values below 130 are considered to represent zones of potential liquefaction or high excess porewater pressure development. Values between 130 and 160 are considered resistant to liquefaction with respect to the design earthquake but may allow for the development of significant excess porewater pressures.

There are a number of uncertainties associated with the use of the simplified method to evaluate the liquefaction resistance of the tailings. These include: a) the effect of the initial static shear stresses induced by the geometry (slope) of the impoundment; b) the ability of the relatively permeable coarse tailings (hydraulic conductivity, k>10-4 cm/s) and the installed drain system to dissipate excess porewater pressures generated during earthquake loading; and c) the actual cyclic loading on the external shell due to the site response of the impoundment which will differ significantly from that of the assumed level ground conditions.

The ultimate goal of the liquefaction analysis is to provide a basis for evaluation of the seismic stability of the impoundment. Given the uncertainties in the liquefaction analysis and the limitations of the pseudo-static method of seismic stability analysis when there is a potential for excess porewater pressure generation and liquefaction, it was decided to use two-dimensional, dynamic numerical modeling to evaluate both the potential for liquefaction and the seismic stability of the impoundment under the 1:1,000-year earthquake. A description and the results of the numerical analysis are presented in a companion paper.

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