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Evaluation of deep liquefaction triggering using in-situ and laboratory test data

Évaluation du déclenchement de la liquéfaction en profondeur à l'aide de données d'essais in situ et en laboratoire

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ABSTRACT: Preliminary design of a marine facility requiring land reclamation with an area of 1.5 km by 0.7 km and placement of 10 m to 15 m of fill indicated the presence of seismically induced liquefiable soil to significant depths. A supplemental geotechnical marine investigation was completed using conventional in-situ testing and sampling tools with some modifications to enable collection of in-situ data and undisturbed samples to a maximum depth of 175 m below mudline. This paper will present a brief description of modifications required for this data collection, a summary of in-situ test data gathered at unusual significant depths, as well as a summary of a comprehensive advanced laboratory testing program. Finally, the paper will also present the site-specific Cyclic Resistance Ratio (CRR) relationship that was developed to supplement the Seed-Idriss Simplified Liquefaction method typically limited to about 15 m depth. Ultimately, the data collected at significant and unusual depths was used in conventional geotechnical analyses to mitigate the deep liquefaction risk.

RÉSUMÉ : La conception préliminaire d'une installation maritime nécessitant la remise en état d'une zone de 1,5 km sur 0,7 km et la mise en place de 10 à 15 m de remblai a révélé la présence d'un sol liquéfiable à des profondeurs importantes. Une étude géotechnique complémentaire a été réalisée à l'aide d'outils d'échantillonnage et d'essais in situ conventionnels, avec quelques modifications pour permettre la collecte de données in situ et d'échantillons de sol non remanié jusqu'à une profondeur maximale de 175 m. Cet article présentera une brève description des modifications requises, un résumé des données d'essai in situ recueillies à des profondeurs inhabituelles importantes ainsi qu'un résumé d'un programme complet d'essais avancés en laboratoire. Enfin, l'article présentera également la relation du rapport de résistance cyclique (CRR) spécifique au site qui a été développée pour compléter la méthode de liquéfaction simplifiée de Seed-Idriss généralement limitée à environ 15 m de profondeur. En fin de compte, les données recueillies à des profondeurs importantes et inhabituelles ont été utilisées dans des analyses géotechniques classiques pour atténuer le risque de liquéfaction en profondeur.

KEYWORDS: marine, liquefaction triggering, significant depth, DSS testing, CPT.

1 INTRODUCTION

Development of a multi-billion dollar marine project on the west coast of Canada included land reclamation of a 1.5 km by 0.7 km area. Prior to development, this area comprised mud flats with existing grades near El. -5 m (geodetic) for the majority of the site and as deep as about El. -12 m (geodetic) along the western edge. Land reclamation for the marine facility would require fill placement to almost El. 9 m (i.e. almost 15 m of fill placement for the majority of the site).

Preliminary geotechnical investigations for this development indicated geologically unconsolidated deltaic sand and silt deposits from the Holocene epoch (primarily cohesionless soil with occasional cohesive layers of limited thickness) extending to depths in excess of 80 m below the mud line, which was underlain by cohesive soil from the Holocene epoch (primarily low plasticity soil) that in turn was underlain by very dense till-like soils from the Pleistocene epoch. These soil conditions raised concerns about:

- seismically induced liquefaction of the Holocene silt/sand deposits to potentially depths of 50 m below the mud line;
- settlements associated with consolidation of the cohesive Holocene silt layers in the upper 80 m;
- settlements associated with consolidation of the cohesive Holocene deposit below 80 m depth.

Preliminary risk mitigation included potential densification of liquefiable soils to depths not previously completed in a marine setting, as well as preload surcharging. It was decided to implement a supplemental site investigation program to further evaluate the risks associated with these hazards (liquefaction and long-term settlements). The objectives of this supplemental investigation program were to:

Primary: Obtain undisturbed sampled of the Holocene sand deposit (generally 25 m to 50 m below mud line) for laboratory cyclic testing;

Secondary: Obtain undisturbed samples of the Holocene cohesive silt deposits (generally within the upper 40 to 80 m of the mud line) and the deeper Holocene cohesive soil (generally below 80 m depth of the mud line) for laboratory consolidation testing, and to determine cohesive soil thicknesses across the site;

Tertiary: Complete test holes in areas with limited existing test hole coverage.

The marine testing program was completed in water depths of 5 m to 10 m and comprised:

- 15 boreholes completed to depths below mud line between 50 m and 175 m (boreholes included sonic core sampling, 76 Shelby tube samples and 17 electric vane shear tests).
- 1 downhole seismic test advanced to a depth of 161 m below mud line;
- 12 Cone Penetration Tests (CPTs) and 7 seismic SCPTs advanced to depths below mud line between 50 m and 145 m;
- Laboratory index testing (water content, grain size analyses, Atterberg Limits, specific gravity);
- Advanced laboratory testing (gamma ray radiography, static direct simple shear, cyclic direct simple shear, post-cyclic consolidation test, bender element velocity measurements, constant rate of strain consolidation, UU triaxial compression, CU triaxial compression).

The following sections present a summary of some of the in-situ and laboratory testing data obtained in the supplemental investigation program, and lastly provide a brief description of how some of this data was utilized in subsequent geotechnical analyses pertaining to liquefaction triggering. Conventional soil consolidation analyses were completed to evaluate the settlement risk, which was mitigated by preload surcharging (not discussed in this paper).

2 DATA COLLECTION

2.1 *Drilling and sampling*

The marine investigation program was executed from two cable connected floating barges (both approximately 12 m by 24 m). Geotechnical drilling was carried out using a Sonic drill rig mounted on a rubber tracked carrier with the drill rig modified to also enable mud rotary drilling.

The Sonic drill utilized a 100 mm ID inner core barrel and a 150 mm ID outer core barrel (also used 75 mm ID core barrel with 125 mm ID drill casing when infrequently encountered high frictional resistance of concern to the drilling contractor). A 200 mm OD casing was also frequently used to stabilize the boreholes, reduce frictional resistance on the drill casing and release drill casing when stuck.

The mud rotary drilling used either Sonic 100 mm ID inner barrel or 120 mm rotary drag bit.

Shallow boreholes (8) were advanced to about 50 m depth below the mudline using the Sonic drilling method. Primary purpose was to define soil index test parameters in this zone that were determined from laboratory testing completed on grab samples retrieved from the Sonic core barrel.

Deep boreholes (7) were advanced using the Sonic drilling method to about 3 m above the target depth at which point the drilling method was switched to the minimal disturbance mud-rotary drilling method. At the target depth, the mud rotary drill string was withdrawn to facilitate in-situ electric field vane testing (eVST) and/or hydraulic piston-deployed Shelby tube sampling (collection of silty sand samples between 25 m to 50 m below mudline, and collection of cohesive silty clay samples between 75 m and 125 m below mudline). The Shelby tube sampling was completed in compliance with ASTM D1587 using stainless steel tubes with 0.76 m length, 75 mm ID and sharpened cutting edges. Upon deployment of the hydraulic piston sampler, the tube was left in place at depth with the soil sample for a waiting period of at least 20 minutes prior to retrieval. At the surface upon sample retrieval, the tube ends were wax sealed and plastic capped prior to being placed upright in a padded box for transportation to the laboratory (all transportation activities were completed to minimize disturbance to the samples).

The 8 shallow and 7 deep boreholes were fairly evenly located within the proposed project site with an area of approximately 1.5 km by 0.7 km.

2.2 *In-situ testing*

Two sets of electric field vane tests (eVSTs) were completed subsequent to Shelby tube sampling (about 0.45 m and 0.75 m below the Shelby tube sampling depth with determination of peak, residual and remoulded shear strength at each depth). The field vane testing was completed using an up-hole electric motor and load cell system. The vane blade was coupled with a Nilcon-type vane road and friction slip coupler, which was attached to the end of the drill string (the slip coupler allows determination of internal friction of the system). The motor was operated by a system control box that included a real time visual display of the plot of soil shear strength as a function of vane rotation. The undrained shear strength was determined from the measured torque and the geometric characteristics of the vane used. The vane testing was completed at 6 of the 7 deep boreholes at a total of 17 different zones in compliance with ASTM D2573 except the testing was completed with a smaller vane at 7 zones with vane dimensions not in compliance with ASTM D2573 (a smaller vane was required due to system torque limitation associated with testing at significant depths).

Downhole seismic testing (DST) was completed to determine the average shear wave velocities of a soil column by measuring the interval travel time of shear waves travelling over the straight path distance between a seismic source and a seismic receiver. The shear wave source comprised a metal box placed on the seabed that contained a spring-loaded hammer-weight and an anvil (hammer horizontally striking the anvil). The DST receiver comprised geophones mounted on an internal block with two horizontal geophones aligned parallel and perpendicular to the seismic source (a built-in fluxgate compass and servo motor system controlled the orientation). The DST was generally completed in compliance with ASTM D7400-14.

Cone Penetration Testing (CPT) and Seismic Cone Penetrating Testing (SCPT) were completed in compliance with ASTM D5778 and D7400. Typically, the CPT/SCPT probe (electronic piezocone) was advanced through BQ drilling rods embedded about 5 m into the seabed to provide lateral probe support. SCPTs were completed using the same equipment as CPTs except execution of SCPTs included utilizing the same seismic shear source as used for DST (see above). Shear wave testing associated with the SCPTs was completed in 1 m depth intervals. All the 7 SCPTs were paired with a borehole and located within 25 m to 30 m of each other, whereas only 2 of the 12 CPTs were paired with a borehole.

CPT/SCPT dissipation tests were completed at 5 m and 10 m depth intervals in the Holocene sand and cohesive silt units, respectively. The dissipation tests typically reached equilibrium in the Holocene sand unit and 50% dissipation (t_{50}) in the Holocene cohesive silt unit. Only infrequent dissipation tests were completed in the deep underlying Holocene cohesive soil unit due to the lower permeability of this deposit.

2.3 *Laboratory testing*

The objective of the laboratory testing program was to measure pertinent engineering parameters (i.e. advanced laboratory tests) at some locations and use these test results at other locations with similar soil conditions (as confirmed by CPT data and/or results from laboratory soil index tests). The advanced laboratory testing program comprised:

- Gamma ray radiography (76);
- Static direct simple shear (4);
- Cyclic direct simple shear (DSS) – stress controlled (50);
- Post-cyclic static direct simple shear (47);
- Post-cyclic consolidation (22);
- Bender element velocity (50);
- Consolidation – constant rate of strain (25);
- UU triaxial compression (7);
- CU triaxial compression (3)

The laboratory soil index testing program comprised:

- Water content (306);
- Grain size analysis (201);
- Fines content (168);
- Atterberg Limits (91);
- Specific gravity (25).

The laboratory tests were completed in compliance with the applicable ASTM procedure and/or laboratory equipment manufacturer's recommended test procedure.

3 RESULTS FROM SITE AND LABORATORY TESTING PROGRAMS

3.1 *Stratigraphy*

The results of the existing and supplemental investigations indicated soil conditions generally comprising:

- deltaic sand and silt deposits from the Holocene epoch to about El. -100 m and El. -150 m (geodetic) at the northeast and southwest corner of the site, respectively. The upper 30 m to 50 m of this deposit was typically sand with high silt content (at least silty) and occasionally had cohesive silt lenses (up to a few meters thick). The bottom of this deposit typically comprised sand with minor fines content (less than silty) and infrequent cohesive silt layers (up to a couple meters thick).
- underlain by a cohesive soil deposit (clayey silt) from the Holocene epoch, which was approximately 5 m and 50 m thick at the northeast and southwest corners, respectively.
- underlain by very dense till like soil (mainly sand and gravel) to maximum investigated depth of 175 m below mudline.

An example of a CPT and interpreted simplified stratigraphy is presented in Figure 1.

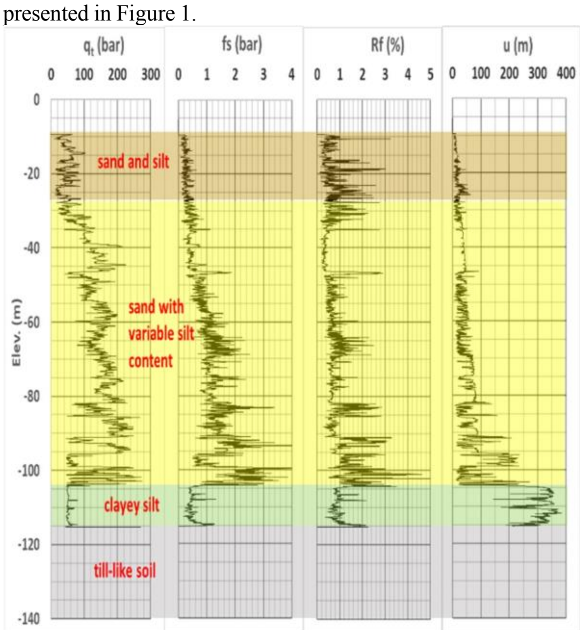

Figure 1 Example of CPT plot.

3.2 *Field vane testing (in-situ)*

Shear vane testing is an in-situ testing method used to determine the peak and remolded undrained shear strengths of cohesive soils. Two sets of electric Vane Shear Tests (eVSTs) were carried out at different depths in several boreholes following collection of Shelby tube samples.

Initially, CPT data was used to derive peak undrained shear strengths ($S_{u,peak}$) using the N_{kt} method (Robertson, 2009). The N_{kt} factor was calibrated to achieve agreement with the undrained shear strength values determined from the eVSTs completed adjacent to the subject CPT (this resulted in a N_{kt} value of 12). Additionally, laboratory Unconsolidated Undrained (UU) triaxial testing was also completed to provide information about undrained shear strength. A comparison of $S_{u,peak}$ values determined from a CPT, eVST results from all deep borehole and all laboratory UU testing is presented in Fig. 2.

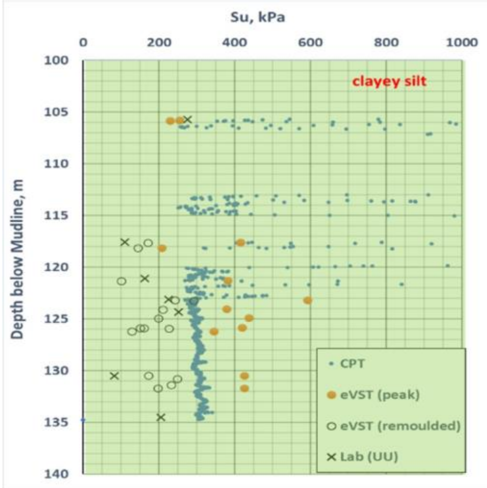

Figure 2 Comparison of Su determined from different methods

3.3 *Shear wave velocity (in-situ)*

Downhole seismic testing (DST) was completed at one location to determine shear wave velocity (Vs) of the very dense till-like soil. A SCPT was also completed adjacent to this DST that included determination of the shear wave velocity. A comparison of Vs determined from the DST and the nearby SCPT is presented in Figure 3 showing good agreement between the two methods.

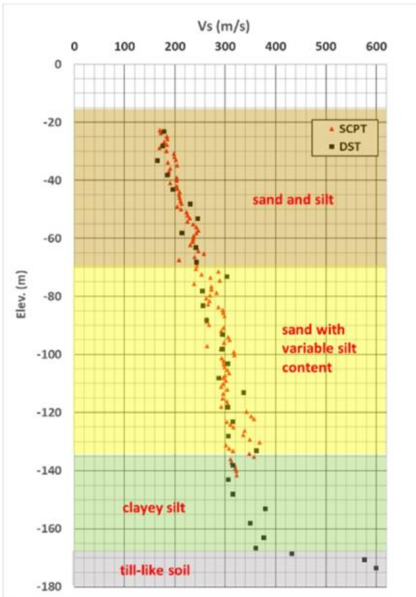

Figure 3 Comparison of Vs from DST and nearby SCPT.

3.4 *Gamma ray radiography (laboratory)*

Prior to assignment of advanced laboratory tests, all Shelby tube samples were subjected to gamma ray radiography completed in general compliance with ASTM D4452. An example is presented in Figure 4. The purpose of these scans was to visually identify the condition of the sample to select the best section of the Shelby tubes for specimen extraction and testing. Upon visually identifying the test section, the Shelby tubes were carefully cut and only the subject section was extracted to minimize sample disturbance (for cyclic DSS testing, the samples were extracted directly into the DSS ring assembly).

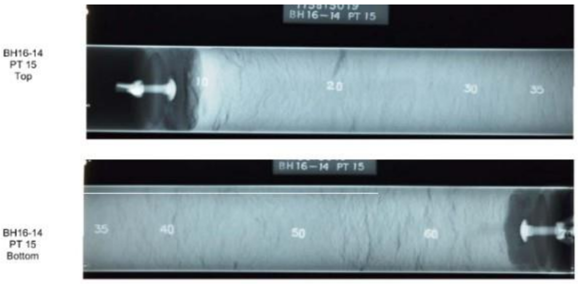

Figure 4 Example of gamma ray radiography of Shelby tube.

3.5 *Bender element testing (laboratory)*

Laboratory Bender Element testing was performed on all cyclic DSS test specimens at test holes where SCPTs and/or DSTs were completed. This testing was performed after the specimen had been consolidated and immediately prior to initiating the cyclic loading A voltage applied to special end platens used in DSS testing generated a shear wave, and the shear wave travel time between these two end platens were measured to determine the shear wave velocity over the 25 mm thick soil sample specimen. The purpose of Bender Element testing was to assist in the evaluation of sample disturbance by comparing in-situ shear wave velocity to that of the laboratory tested sample.

In-situ SCPT shear wave velocity was measured as an average over a 1.0 m depth interval, whereas the Bender Element testing was measured as the average over the 25 mm thick soil specimen. To enable comparison over a similar depth interval, an interpretive shear wave velocity method (Robertson, 2009) using CPT data was also completed. An example of these in-situ and laboratory measured shear wave velocities as well as the interpreted shear wave velocity is presented in Fig. 5.

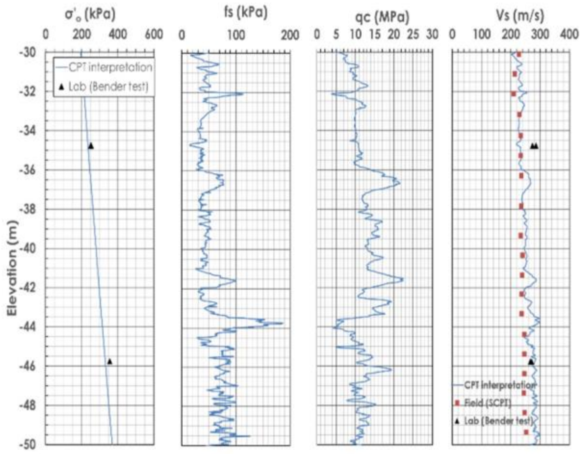

Figure 5 Example of shear wave profiles.

Review of the charts in Figure 5 indicates very good agreement between the CPT interpretive method (i.e. interpreting at every 25 mm depth interval) and the Bender Element tests (i.e. shear wave velocity of a 25 mm thick soil specimen). However, the measured in-situ shear wave velocity results (i.e. average over 1 m) are generally lower (roughly 5% to 38% with an average of 15%). This difference is considered to be mainly contributed by heterogeneity of the Holocene deposits (erratic CPT data also suggests heterogeneity). Overall, the difference is considered to be minimal in light of the numerous variables (i.e. SCPT horizontally offset from borehole location, thickness of tested soil, difference in measurement techniques, soil heterogeneity, etc.). Thus, it was concluded that this indicated relatively undisturbed soil samples used for the cyclic DSS testing.

3.6 *Cyclic DSS testing (laboratory)*

Unidirectional cyclic Direct Simple Shear (DSS) testing was completed on 50 specimens retrieved from 22 piston samples (2 to 4 specimens per sample) with the samples collected between 25 m and 50 m depth below mudline. Of those:

- 40 specimens from 19 samples were tested to determine number of cycles to liquefy (defined as ε = 3.75%) for different Cyclic Stress Ratio (CSR defined as τ_{cyc}/σ'_{vc}) values to represent earthquake induced stress conditions. Note, specimens from one sample were all tested under the same vertical consolidation stress (σ'_{vc}).
- whereof 8 specimens were tested with $\sigma'_{vo} = \sigma'_{vc}$ to simulate same existing and final grades;
- whereof 26 specimens were tested with $\sigma'_{vc} > \sigma'_{vo}$ to simulate areas requiring fill placement (10 tests with σ'_{vc} = 105% to 125% of σ'_{vo}, 10 tests with σ'_{vc} = 140% to 155% of σ'_{vo}, 6 tests with σ'_{vc} = 190% to 250% of σ'_{vo});
- whereof 6 specimens were tested with $\sigma'_{vc} < \sigma'_{vo}$ to simulate areas requiring dredging (σ'_{vc} = 55% to 65% of σ'_{vo}). Initially consolidated to σ'_{vo}, then unloaded to σ'_{vc}.

Grain size distribution of all 19 samples are shown in Figure 6 indicating sand with silt content between 5% and 65% and less than 10% clay content. Atterberg Limit testing indicated all samples were non-plastic.

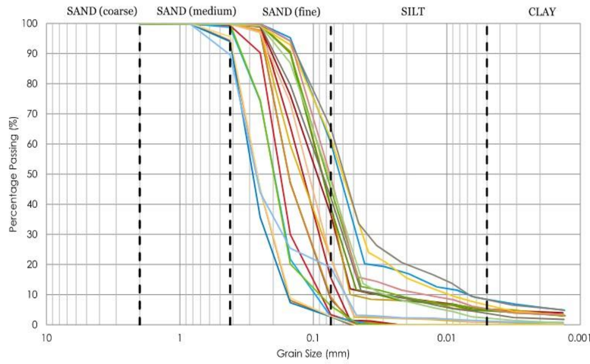

Figure 6 Grain size distribution of 19 samples used for cyclic DSS testing (all non-plastic).

Deaggregation of the seismic hazard included determination of the mean earthquake magnitude (Mw) for several return periods, which was used to define the number of equivalent uniform cycles at 65% of the peak stress (Idriss & Boulanger, 2008). A summary of this evaluation is presented in Table 1 indicating the range of interest of number of equivalent uniform cycles ranging from 8 to 11.

Table 1 Earthquake magnitude and equivalent uniform cycles

Return Period	Mean Magnitude (Mw)	No. of equivalent cycles
1:100 yrs	6.76	8
1:475 yrs	7.05	11
1:975 yrs	7.10	11
1:2475 yrs	7.11	11

The results of the 40 cyclic DSS tests completed on samples with different fines content (FC) are shown in Figures 7 to 9 for conditions with $\sigma'_{vc} = \sigma'_{vo}$, $\sigma'_{vc} > \sigma'_{vo}$, and $\sigma'_{vc} < \sigma'_{vo}$, respectively. Each figure shows the number of cycles required for liquefaction (NL) allowing determination of laboratory derived liquefaction resistance (i.e. Cyclic Resistance Ratio, CRR_{lab}) at a certain return period. In these figures, CSR and CRR are interchangeable.

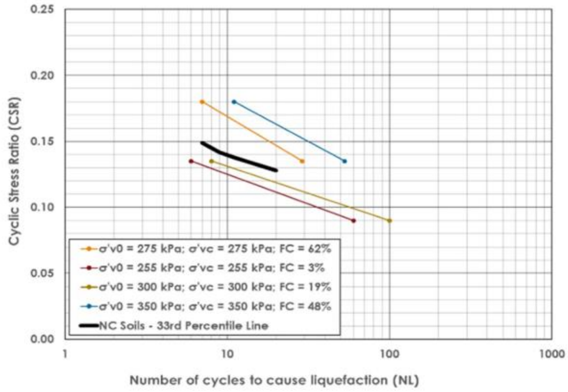

Figure 7 CSR vs number of cycles to liquefy ($\sigma'_{vc} = \sigma'_{vo}$)

For each sample (i.e. 2 to 4 specimens), the shown linear data sets were extrapolated in the semilogarithmic plots if needed to cover the range of interest between 8 and 11 cycles. Due to the significant scatter of the data shown in Figures 7 to 9, the 33rd percentile of the data was used to define CSR vs NL relationships.

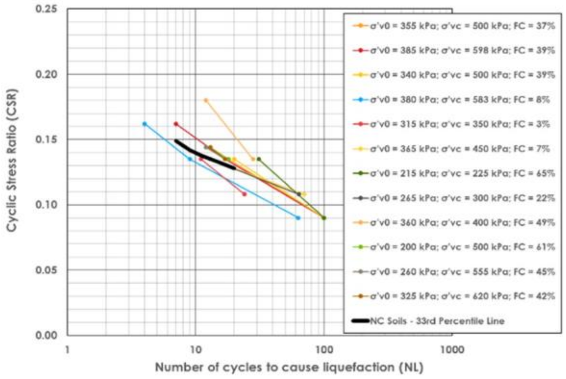

Figure 8 CSR vs number of cycles to liquefy ($\sigma'_{vc} > \sigma'_{vo}$)

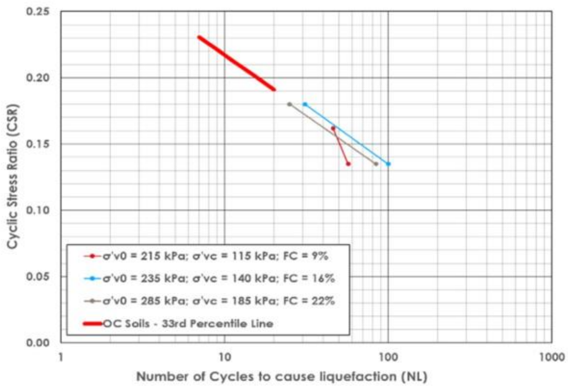

Figure 9 CSR vs number of cycles to liquefy ($\sigma'_{vc} < \sigma'_{vo}$)

For the cases with $\sigma'_{vc} = \sigma'_{vo}$ (i.e. simulating existing grades similar to final grades) and $\sigma'_{vc} > \sigma'_{vo}$ (i.e. fill placement), the specimens were prepared to be normally consolidated. Comparison of the 33rd percentile between these two cases show they are relatively similar. Thus, the laboratory cyclic DSS tests were compiled to represent CSR versus NL for normally consolidated and over-consolidated cases (see Figure 10).

Examination of the data did not indicate any other conclusive relationships (for example CSR as a function of fines content, relative density, vertical stress, etc.).

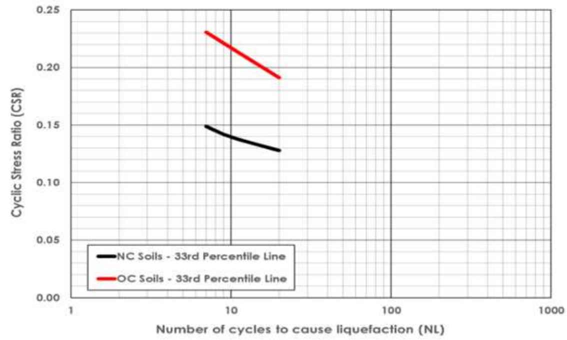

Figure 10 CSR vs number of cycles to liquefy (summary)

The remaining 10 cyclic DSS tests were completed as follows:

- 3 specimens from each of 2 samples were tested to evaluate the effects of vertical consolidation stress (σ'_{vc}). For σ'_{vc} = 250, 350 and 450 kPa and CSR of 0.15, the 6 cyclic DSS tests indicated 5 cycles to liquefy for all tests (i.e. σ'_{vc} had insignificant impact on number of cycles needed for specimen to liquefy).
- 4 specimens from 1 sample tested for static bias effects (i.e. evaluate effect in sloping ground conditions). Two specimens were consolidated to 500 kPa and then cyclic DSS tested without static bias at CSR of 0.13 and 0.14. Subsequently, the other two specimens were tested at CSR of 0.13 with static bias (α) of 0.2 and 0.3 (initially consolidated to 500 kPa prior to applying a horizontal shear stress for 24 hrs to induce static bias conditions). The resulting static shear stress correction factor (K_α = CRR_α/CRR) was 0.68 and 0.74 for α = 0.2 and 0.3, respectively. These values compare well with those presented in (Idriss & Boulanger, 2008).

4 LIQUEFACTION ANALYSES

4.1 *CPT-derived Cyclic Resistance Ratio (CRR) profiles*

The Seed-Idriss Simplified method (Idriss & Boulanger, 2008) was used to evaluate liquefaction triggering, but it was recognized that this semi-empirical method is only valid to about 15 m depth. Therefore, this method was supplemented by the results of the cyclic DSS testing. Specifically, liquefaction resistance (i.e. Cyclic Resistance Ratio, CRR) versus depth profiles were initially developed for each CPT using the following equation:

$CRR_{in\text{-}situ} = CRR \cdot MSF \cdot K\alpha \cdot K\sigma$ (Idriss & Boulanger, 2008) (1)

where

- CRR is the cyclic resistance ratio normalized to an earthquake of M = 7.5 and σ'_{vo} = 1 atm;
- MSF = earthquake magnitude scaling factor;
- Kα = static stress bias correction factor;
- Kσ = overburden correction factor.

Using methods outlined by Idriss & Boulanger (2008), the recorded CPT tip resistance (q_c) was corrected for confining stress using the correction factor C_N to achieve stress corrected values (q_{c1}), and then subsequently corrected for atmospheric pressure to obtain a dimensionless tip resistance (i.e. q_{c1N}). Subsequently, this dimensionless tip resistance was corrected for fines content (i.e. q_{c1Ncs}) using the method of (Robertson and Fear, 1998), which indicated the CPT derived 'apparent fines content' closely matched the laboratory testing determined fines content.

Since earthquake loading is best approximated using a two-direction simple shear loading, the CRR values derived from the unidirectional cyclic DSS tests were adjusted to relate to in-situ CRR values as follows:

$$CRR_{lab\text{-}corrected} = 0.9 \cdot CRR_{lab} \text{ (Idriss \& Boulander, 2008)} \quad (2)$$

The CPT derived CRR values for the site (i.e. $CRR_{in\text{-}situ}$) were scaled until the profile of $CRR_{in\text{-}situ}$ matched the CRR values from the laboratory testing ($CRR_{lab\text{-}corrected}$). This scaling procedure was completed for each earthquake return period. The scaling factor varied between 1.00 and 1.35 at the 7 combined SCPT/borehole locations (the scaling factor was similar for different earthquake return periods at these locations). Subsequently, the $CRR_{lab\text{-}corrected}$ values representing existing conditions were modified to represent final grades (CRR_{final}), if needed. For areas requiring fill placement, this resulted in negligible change (i.e. ΔCRR ~ 0.01). For areas requiring dredging, this resulted in an increase as to the difference between normally and over-consolidated conditions (i.e.CRR_{OC}/CRR_{NC} ~ 1.55, see Figure 10). An example of a location requiring fill placement is shown in Figure 11 for a return period of 100 years. This procedure was repeated for other earthquake return periods and indicated negligible differences (i.e. ΔCRR ~ 0.01).

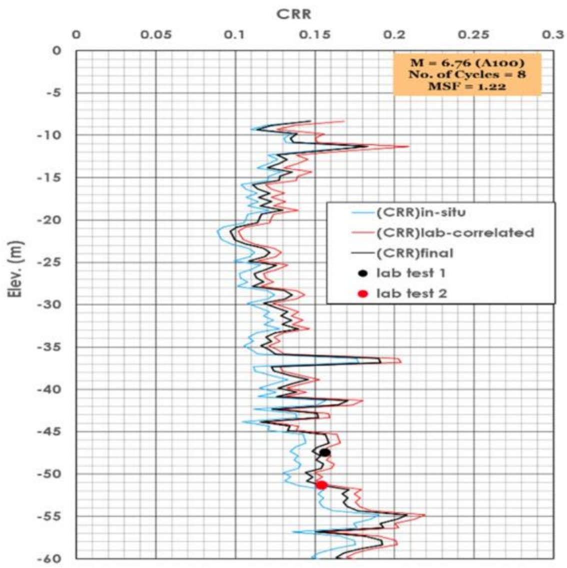

Figure 11 CPT derived CRR profiles for 100 year return period.

4.2 *Determination of CSR and liquefaction evaluation*

The Cyclic Stress Ratio (CSR) induced by the earthquake was determined by completing site specific one-dimensional equivalent-linear total stress dynamic site analyses. For each of the earthquake return periods (i.e. 100, 475, 1000, 2475 yrs), a suite of 11 linearly scaled ground motions was utilized to derive a CSR profile versus depth for each of the 11 ground motions. Ultimately, the average of these 11 profiles was used to represent the stresses induced by an earthquake with a certain return period (details pertaining to determination of CSR are beyond the objective of this paper). This average CSR profile was compared to the CRR profile derived at each location. An example of this comparison at a borehole/CPT location is shown on Figure 12 for existing conditions and final conditions requiring fill placement.

For areas with static stress bias (i.e. sloping ground), two-dimensional numerical modelling was completed to determine CSR value (details pertaining to determination of CSR in areas with static stress bias are beyond the objective of this paper).

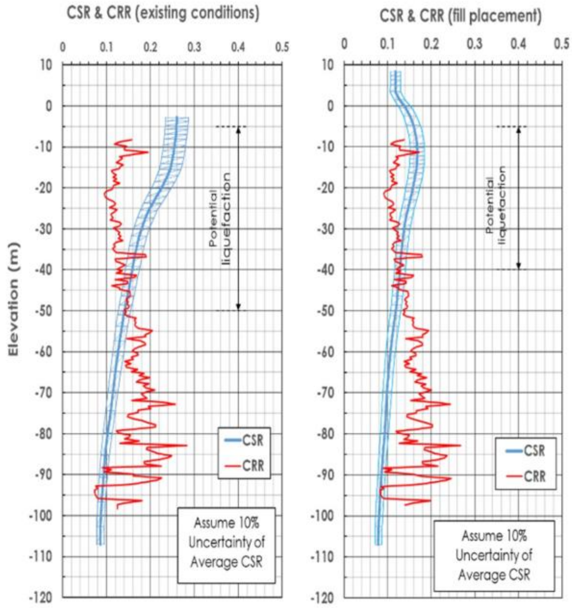

Figure 12 Liquefaction potential at a borehole/CPT location for earthquake with 1,000 yrs return period.

Ultimately, the liquefaction triggering analyses indicated the following upon completion of the required site grading (i.e. fill placement and dredging):

- **Fill placement areas:** High risk of liquefaction extending to depths in the range of El. -30 m to El.-40 m for 1,000 yrs return period (the potential liquefaction depth extending about 10 m deeper for 2,475 yrs return period);
- **Dredging areas:** High risk of liquefaction extending to depths in the range of El. -40 m for 1,000 yrs return period (the potential liquefaction depth extending a few meters deeper for 2,475 yrs return period);
- **In front of fill slopes:** High risk of liquefaction extending to depths in the range of El. -40 m to El.-50 m for 1,000 yrs return period (the potential liquefaction depth extending about 5 m to 10 m deeper for 2,475 yrs return period)

6 CONCLUSIONS

Challenging soil conditions resulted in concerns pertaining to mitigation measures needed to address deep liquefaction at the site as well as settlements due to cohesive soil extending to significant depths (settlement hazard not discussed in this paper). A supplemental site investigation retrieved undisturbed soil samples from significant depths (up to 125 m) to enable determination of soil parameters relevant to these two hazards.

Laboratory DSS test results suggested that extrapolating the Seed-Idriss Simplified CPT method (Idriss & Boulanger, 2008) beyond the method's depth limitation of 15 m to 20 m resulted in reasonably to slightly underestimated CRR values at depths in the 30 m to 50 m range at this site.

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