Assessment of liquefaction potential at a site in Ottawa using SPT and shear wave velocities

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ABSTRACT
Liquefaction potential assessment of subsurface soil needs detail geotechnical investigations, and testing which is usually beyond the scope of conventional geotechnical investigations. Therefore, using simplified methods to assist with preliminary site assessments can be beneficial in advance of more elaborate field and laboratory works. Shear wave velocity measurement (V_s) is a promising tool for this purpose. However, the available analysis techniques are based on limited experiments and need further verification.

A case is presented where V_s measurement was successfully used to delineate a liquefiable soil layer at a Site. Low V_s values were observed within the upper 15 m of soil layers based on measurements using MASW method. Supplementary calculations showed the potential for liquefaction. Thus, a deep borehole was installed at the site with soil sampling at regular intervals using SPT method. Selected samples were subjected to laboratory tests. Detail calculations using SPT values showed that about 12 m of the subsurface soil is prone to liquefaction. Despite some discrepancies between the V_s and SPT calculations, V_s measurement showed to be a promising tool for preliminary liquefaction assessments.

RÉSUMÉ
L'évaluation du potentiel de liquéfaction des sols nécessite des investigations géotechniques plus élaborées et des essais qui excèdent les besoins des études géotechniques courantes. En conséquence, il peut s'avérer avantageux d'utiliser des méthodes simplifiées dans l'évaluation préliminaire de sites avant de procéder à des travaux de terrain et de laboratoire plus élaborés. Dans ce but, la mesure des vitesses des ondes de cisaillement (Vs) est une technique prometteuse. Cependant, les méthodes d'analyse disponibles s'appuient sur une expérience limitée et nécessitent d'être vérifiées d'avantage.

Une étude de cas est présentée où la mesure de Vs a été utilisée pour identifier une couche de sol liquéfiable. Basé sur des relevés avec la méthode MASW, de faibles valeurs de Vs ont été observées dans les premiers 15 m de sol. Des calculs additionnels ont confirmé le potentiel de liquéfaction. Ensuite on a réalisé un forage profond avec des essais SPT à intervalles réguliers pour obtenir les indices de pénétration standard (N) du sol. Des échantillons de sol représentatifs ont été soumis à des essais de laboratoire. Les calculs détaillés avec les indices N ont montré que les sols sont effectivement susceptibles à la liquéfaction sur près de 12 m. Malgré cet écart entre les résultats des calculs réalisés avec Vs et ceux avec les indices N, la mesure de Vs se présente comme un outil prometteur pour les évaluations préliminaires du potentiel de liquéfaction.

1 INTRODUCTION

Assessment of the liquefaction potential of subsurface soils needs a detail geotechnical investigation, laboratory testing and calculations which are usually beyond the scope of work of conventional geotechnical investigations. Specifically, in medium to low risk seismic areas with limited history of extreme seismic events detail subsurface investigations for this purpose often does not have financial justification. However, in heavily urbanized areas even if the probability of a major seismic event is low the risk of such event can be high with significant adverse consequences. Therefore, development of simplified methods for preliminary assessment of sites for liquefaction potential can be very beneficial in evaluating the need for more elaborate field and laboratory works and detail calculations.

Non-destructive testing (NDT) using shear wave velocity (V_s) is a promising tool for these preliminary evaluations. The measurements are relatively simple, fast can be carried out at several locations for proper site assessments, and are repeatable due to the non-destructive nature of these tests. However, the available V_s-based liquefaction analysis techniques are based on limited field observations and need further verification.

This paper presents a case that liquefiable soil layers were delineated based on shear wave velocity measurements using multi channel analysis of surface waves (MASW) method. Complementary borehole investigation, associated laboratory testing and SPT-based liquefaction analysis confirmed the existence of layers susceptible to liquefaction. However, some discrepancies are observed in the results obtained from the two different methods. Comparisons are made

between the different methods and potential reasons for discrepancies are investigated and discussed.

1.1 Liquefaction Assessment Methods

Traditionally, liquefaction is defined as the transformation of a granular material from a solid to a liquefied state as a consequence of increased pore-water pressure and reduced effective stress (Marcuson 1978). It was well recognized for long time that clean sands with few fines are susceptible to liquefaction. However, in the past two to three decades and following the observations during strong earthquakes in China, Turkey and Taiwan it is concluded that non-plastic to low plasticity fine grained materials - as well as granular material - may also be subjected to classical liquefaction. Boulanger and Idriss (2004) review the mechanisms of liquefaction of no to low plasticity fine grain materials subjected to cyclic loading. Taking into account the recent advances on this subject, the latter reference uses the term "liquefaction" to describe the onset of high excess pore water pressures and large shear strains during undrained cyclic loading of sand-like soils, while the term "cyclic failure" is used to describe the corresponding behaviour of clay-like soils.

Four in-situ testing methods are commonly used for liquefaction assessment, which are 1) the Standard penetration Test (SPT) 2) the Cone penetration Test (CPT) 3) in-situ shear wave velocity measurement and 4) Becker penetration test (BPT). Analytical evaluation methods are also developed for each of the noted techniques. Laboratory tests for liquefaction analysis are not very well accepted in industry due to the difficulties in obtaining undisturbed and representative samples from non-cohesive materials.

The oldest and most widely used in-situ testing method for liquefaction assessment is the SPT method (Seed et. al. 2003). SPT field work is relatively simple and very well developed. Further, SPT based correlations for liquefaction assessments have been in use for relatively long time and are validated by numerous case histories. Various evaluation methods have been proposed for liquefaction assessment using SPT method such as the Seed and Idriss Simplified method (Seed et. al. 1984), Youd and Idriss (2001), and Cetin et. al. (2004).

V_s-based field and analytical works for liquefaction assessment are relatively new in compare to SPT-based methods (Andrus and Stokoe, 2000). These methods have considerable advantages over conventional SPT and CPT techniques. The field measurements are possible on hard to sample soils, can account for the variability of subsurface soil across a site, are repeatable and fast, and can be performed at a relatively low cost. From a theoretical viewpoint, shear wave velocity is a mechanical property of soil deposits, whereas SPT and CPT measure indices that are indirect representatives of these properties. On the other hand, V_s measurements are made at low strains therefore they are not very sensitive to cyclic/post-cyclic loading behaviour of subsurface soils, no samples are recovered during V_s measurements, and thin layers of loose/very loose soils may not be detected. Further, V_s-based analysis methods are based on limited case histories, adding to the involved uncertainties.

Generally, most of the available liquefaction analysis procedures follow the formats initially used in Seed-Idriss simplified method. In this format the earthquake loading on the soil is calculated using the cyclic stress ratio (CSR), which is then compared to the cyclic resistance ratio (CRR) calculated based on corrected local soil properties such as SPT or V_s measurements.

In this work, the SPT-based evaluations follow the Seed-Idriss simplified method as stated in Idriss and Boulanger (2008) and the V_s-based evaluations follow the procedures described in Andrus and Stokoe (2000). In these methods CSR is defined by the following equation:

$$CSR = 0.65\left(\frac{a_{\max}}{g}\right)\left(\frac{\sigma_v}{\sigma_v^{'}}\right)r_d \qquad [1]$$

Where $a_{\max}$ is peak horizontal ground surface acceleration, g is gravity acceleration, σ_v and σ'_v are the total and effective stresses at the specific depth and r_d is the depth correction factor. For brevity the equations for SPT-based CRR calculations are not provided here and the interested reader is referred to the above noted reference. For the V_s-based calculations the following equation is used for CRR calculations:

$$CRR = \left\{a\left(\frac{V_{S1}}{100}\right)^2 + b\left(\frac{1}{V_{S1}^* - V_{S1}} - \frac{1}{V_{S1}^*}\right)\right\}MSF \qquad [2]$$

Where MSF is magnitude scaling factor, a and b are curve fitting parameters, V_{S1} is shear wave velocity corrected for overburden pressure, and V^*_{S1} is the limiting upper value for V_{S1} and is defined as below:

$$V_{S1}^* = 215 \quad m/s \qquad for\ sands\ with\ FC \le 5\% \qquad \text{(a)}$$

$$V_{S1}^* = 215 - 0.5(FC - 5) \quad m/s \qquad for\ sands\ with\ 5\% \le FC \le 35\% \qquad \text{(b)} \qquad [3]$$

$$V_{S1}^* = 200 \quad m/s \qquad for\ sands\ with\ FC \ge 35\% \qquad \text{(c)}$$

Where FC is the average fines content (percent) by mass.

Finally, the Factor of Safety (FS) against liquefaction is defined by the following relationship:

$$FS = \frac{CRR}{CSR} \qquad [4]$$

A factor of safety less than 1 is indicative of potential for liquefaction triggering during an earthquake.

1.2 MASW Method

Multichannel analysis of surface waves (MASW) method is a seismic geophysical technique to estimate the soil

shear wave velocity profile. MASW was developed in late 1990's as an advancement of other surface wave techniques, which were in use since early 1960's (Park et. al. 1999, Nazarian 1984). This method has been used in various applications such as seismic site characterization according to building codes, dynamic soil parameter estimation, non-destructive evaluation of subsurface soil condition including hard-to-sample soil deposits, pre and post construction soil conditions, and detection of underground cavities (Xia 2006, Nasseri-Moghaddam 2007).

To carry out an MASW test, several geophones are deployed along a line at certain distances from an impact source (Figure 1). The length of the geophone array (D) determines the deepest investigation depth that can be obtained from the measurements. The distance between the source and first receiver (offset) determines the contamination level of the signals. The source should produce enough energy over the desired test frequency range to allow for detection of Rayleigh waves above the background noise. A common source is a sledgehammer; or heavy drop weight hitting a base plate mounted at the ground surface. Generally, using a sledgehammer the maximum investigation depth is limited to about 15 m to 20 m below the existing ground surface (bgs), however using traffic noise or heavy drop weights the investigation depth can be increased to more than 30 m bgs.

Theoretically, the MASW test is based on the dispersive behaviour of Rayleigh wave (R-wave) in a layered media (Park, 1999, Rix, 2005). Dispersion of R-wave arises because different frequencies traverse the medium with different velocities. The latter is due to the fact that the penetration depth of R-wave is inversely proportional to its frequency. Thus, higher frequencies travel through shallower strata, and lower frequencies propagate mostly in the deeper layers. For practical purposes, the maximum depth of penetration can be considered to be equal to one to one third of the wavelength (KGS 2008, Stokoe 2008). Therefore each frequency carries the information associated to a specific depth of the medium that it is traversing. The recorded field responses (time domain signals) constitute the calculation basis for phase velocity-frequency curve (dispersion image) of the line. Subsequently, inversion of the constructed dispersion curve leads to the estimation of the shear wave velocity profile at the site.

2 INVESTIGATION METHODOLOGY

Due to the relatively small size and low importance of the proposed development, a conventional geotechnical investigation comprising shallow boreholes was planned and carried out to assist with the design and construction activities at a site in Ottawa, Ontario (Site). MASW method was used to measure shear wave velocities at the Site to assist with the seismic site class determination. Very low velocities were measured within the upper 15 m of the soil profile, indicating the potential for liquefiable soils at the Site. Therefore, a complementary deep borehole investigation and associated laboratory testing were devised and carried out to verify the subsurface soil condition at the Site.

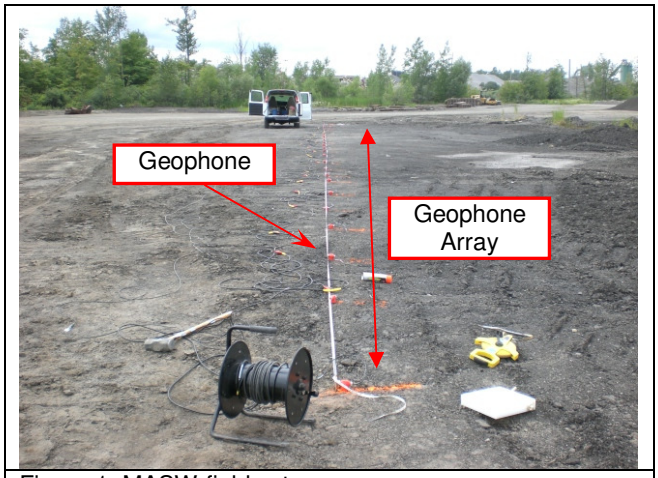

Figure 1. MASW field set up.

The borehole investigation consisted of two shallow boreholes (BH1 and BH2) to approximate depth of 7.0 m bgs and one deep borehole (BH3) to approximate depth of 29.5 m bgs. The boreholes were sampled at regular intervals using SPT. Undisturbed Shelby tube samples were also recovered at various depths from BH3. Piezometers were installed at the location of BH1 and BH2 to assess the groundwater depth at the Site. A total of 8 soil samples were subjected to gradation (sieve and hydrometer) and Atterberg limits tests. Further all samples were tested for moisture content.

MASW testing was carried out along two lines in the vicinity of the drilled boreholes. Each investigation line consisted of 24-4.5 Hz geophones which were set up at 1.5 m spacings (total array length of 34.5 m). Active vibrations were generated by hitting a 20lb sledge hammer to a rubber base-plate. Offset distances of 15 m and 7.5 m (10 and 5 geophone spacings, respectively) were used along each line. To evaluate the lateral variability of V_s profile each line was rolled 4 times at rolling distance of 1.5 m. A summary of the borehole data and shear wave velocity profile is provided in Figure 2.

3 OBSERVATIONS AND ANALYSIS

3.1 Field and Laboratory Observations

The boreholes show a shallow layer of topsoil about 0.75 m thick, overlying a layer of sandy silt about 2.0 m thick, underlain by a clayey silt, trace sand deposit about 7.0 m thick, overlying a silty clay, trace sand deposit that was encountered to the termination depth of BH3. The field measured SPT 'N' values within the sandy silt layer (upper 3 m of the subsurface) resulted in values ranging from 3 to 12 blows per 0.3 m of penetration, indicating the very loose to compact nature of these deposits. Within the underlying clayey silt layer (approximately from 2.75 m bgs to 10.0 m bgs), SPT 'N' values between

0 to 3 blows per 0.3 m of penetration were measured indicating the generally very loose/very soft nature of this deposit.

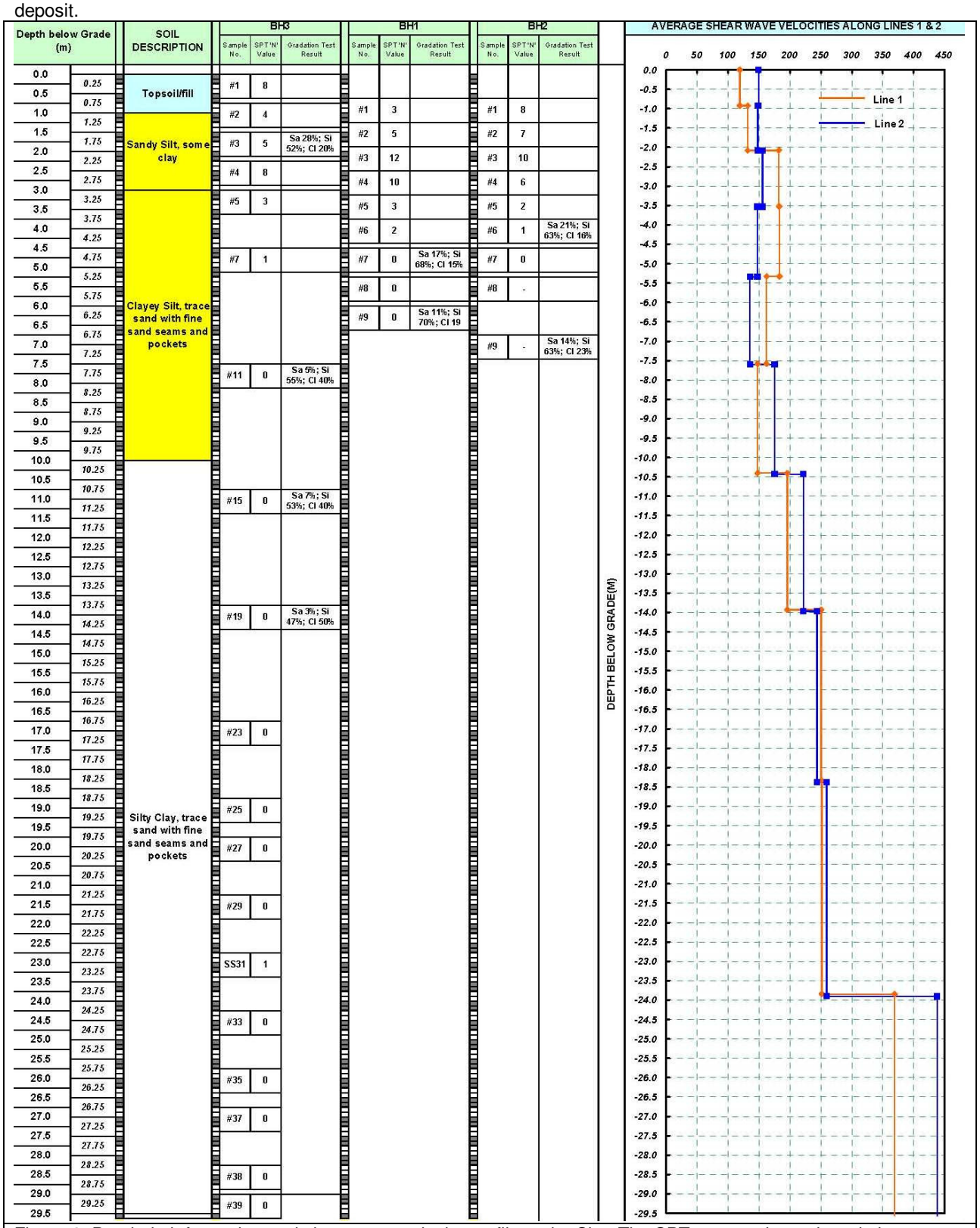

Figure 2. Borehole information and shear wave velocity profile at the Site. The SPT test results and gradation test results are shown at the corresponding depths.

The SPT 'N' values within the underlying silty clay deposit resulted in values from 0 to 1 blow per 0.3 m of penetration, indicating the very soft nature of this deposit.

The gradation tests on the recovered samples from about 1.25 m to 10.75 m bgs resulted in more than 50% silt with various amounts of sands and clays. The sand content within the tested materials decreased with depth from about 28% at 1.75 mbgs to about 7% at 10.75 mbgs. These results are shown on Figure 2 at the corresponding depths. Atterberg limits tests were carried out on representative samples from BH3. Figure 3 shows the Atterberg test results overlain on plasticity chart. These results indicate that generally the samples up to about 11.0 mbgs have low plasticity and the sample from 14.0 mbgs showed medium plasticity characteristics.

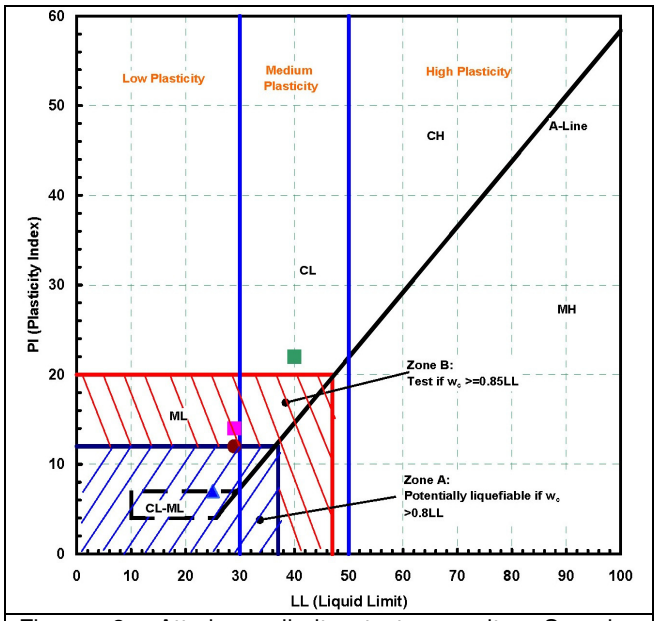

Figure 3. Atterberg limits tests results. Samples recovered from borehole BH3 at various depths, i.e. ▲ from 4.5 m, ● from 7.5 m, ■ from 11.0 m and ■ from 14.0 m.

The MASW data were analyzed using SurfSeis® version 2.0. Dispersion curves were generated and inversion analysis was carried out using a 10 layer soil model. Within the upper 10 m of subsurface very low shear wave velocities (lower than 200 m/s) were measured. Below this depth to about 24.0 m bgs measured velocities were in the range of 200 m/s to 300 m/s. The velocities measured below the depth of 24.0 m are considered unreliable due to the low resolution of dispersion curves. In general the very low shear wave velocities measured at this site indicate the very loose/soft nature of the subsurface soils. The average shear wave velocity within the upper 30 m of subsurface ($\bar{V}$) calculated as per the averaging scheme of NBCC 2005 are 186 m/s and 189 m/s along lines 1 and 2, respectively. These averages are marginally larger than the threshold for site class E. Considering these marginal values and the accuracy of the measurement methods a site class E is associated to the Site.

3.2 Liquefaction Analysis

The above noted field data were used to evaluate the liquefaction triggering potential at the Site. This assessment consisted of evaluation of the type of soil, V_s- and SPT-based calculations, comparison with historical data, and comparison of obtained factors of safety.

The gradation tests show that the soils at this Site contain significant fine content. Therefore, the guidelines provided in Seed et. al. (2003) for evaluating liquefiability of soils with significant fines is used in this assessment. This guideline divides the plasticity chart into three zones as defined below and shown in Figure 2:

- Zone A (blue hatch) defined by LL ≤ 37% and PI ≤12% and is indicative of soils susceptible to potential classic liquefaction.
- Zone B (red hatch) is indicative of soils that may be liquefiable and is defined by LL ≤ 47% and PI
- Zone C is the area outside zones A and B. The soil within this area is not susceptible to classical liquefaction, though it may be prone to cyclic softening (sensitivity)

The four tested samples are plotted on the graph shown on Figure 3. It is observed that the first sample (depth 4.5 mbgs) lies within Zone A, the second sample (depth 7.5 mbgs) lies on the boundary of Zones A and B, the third sample (depth 11.0 m) lies within Zone B and the fourth sample (depth 14.0 mbgs) lies within Zone C, thus three of the four tested samples have the potential to be subjected to classic liquefaction during a seismic event.

The V_s and SPT data (corrected for overburden pressures) are used to quantify the effect of liquefaction and the depth of the liquefiable soil layers. A magnitude 6.1 earthquake at a probability of exceedence of 2%/50 years is used for this evaluation based on deaggregation curves for Ottawa (Halchuk et. al. (2007). The peak ground acceleration (PGA) for Site Class C for a site in Ottawa is 0.42g (NBCC 2005). In this analysis a PGA of 0.5g (modified PGA for site class E) is used. A discussion on the adopted PGA value is provided in the subsequent sections.

Figure 4 shows the *FS* values - obtained from V_s- and SPT-based liquefaction analysis - plotted against depth below ground surface. For plotting purposes the non-liquefiable soil layers (due to soil and/or ground conditions) are assigned an FS value of 2.

The V_s-based factors of safety along the first MASW investigation line (blue diamond) show the potential for liquefaction at about 2 mbgs, and between 6 to 12 mbgs. These data do not show liquefaction potential at 4 m, and below 12.5 m bgs. The results from second MASW investigation line (pink diamond) show the potential for liquefaction from about 4 mbgs to 12 mbgs. Based on these data there is no liquefaction potential above 4

mbgs or below 12 mbgs. The SPT-based factors of safety (red triangles) show potential for liquefaction from about 1 mbgs to 12 mbgs.

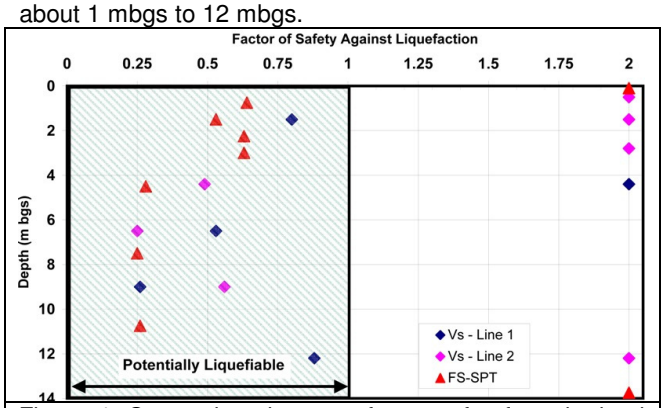

Figure 4. Comparison between factors of safety obtained from V_s-based and SPT-based liquefaction analysis.

Overall the V_s-based factors of safety showed potential for liquefaction from about 2 mbgs to 12 m bgs which is in good agreement with the corresponding values calculated based on the SPT data. However, discrepancies are also observed between the evaluated factors of safety. These discrepancies are mainly observed for the soil layers above 4 mbgs. Specifically, the results obtained from the two MASW lines do not completely match with each other and also they do not agree with the SPT-based evaluations. A discussion on the possible sources for discrepancies is provided in the subsequent sections.

A comparison is made between the data obtained in this study and published case histories. Figures 5 and 6 show results from this comparison for V_s-based and SPT-based case histories, respectively. In each figure the published case histories are shown in the background in black. The solid black dots in each figure show the cases where liquefaction were observed at the investigated site, and the hollow dots show cases where liquefaction were not observed during a seismic event. The trend lines (solid or dashed black lines in each figure) show the best fits that separate the liquefied cases from non-liquefied cases. Thus, the points plotted to the left of the trend lines are indicative of the liquefaction potential.

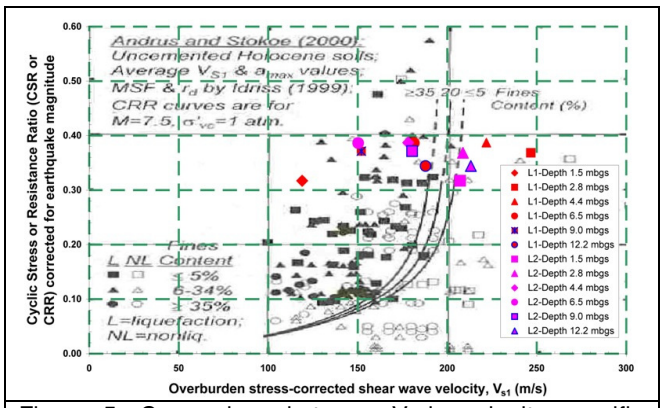

Figure 5. Comparison between Vs-based site specific data and published case histories (Andrus and Stokoe, 2000).

Figure 5 compares the V_s-based analysis results with compiled case histories from Andrus and Stokoe (2000). In this figure the horizontal axis shows the stress-corrected shear wave velocity and the vertical axis shows the corrected CSR or CRR values. The red and pink dots are the CSR-V_s pairs corresponding to investigation Lines 1 and 2, respectively. It is observed that the points between about 4 m to 12 are plotted to the left of the trend lines, indicating the liquefaction potential of these layers. The proximity of the data obtained from this investigation and the historical data (black solid dots) indicate the liquefaction potential at the investigated Site and strengthens the conclusion that soils between about 2.5 m to 12.0 m are potentially liquefiable.

Figure 6 shows a similar comparison between the SPT-based analysis results and compiled case histories (Idriss and Boulanger 2008). The horizontal axis shows the corrected SPT 'N' values ($N1_{60}$) and the vertical axis shows the corrected CSR or CRR values.. This comparison shows that all the points are plotted to the left of the trend curves, indicating the liquefaction potential from about 0.75 mbgs to more than 10 mbgs.

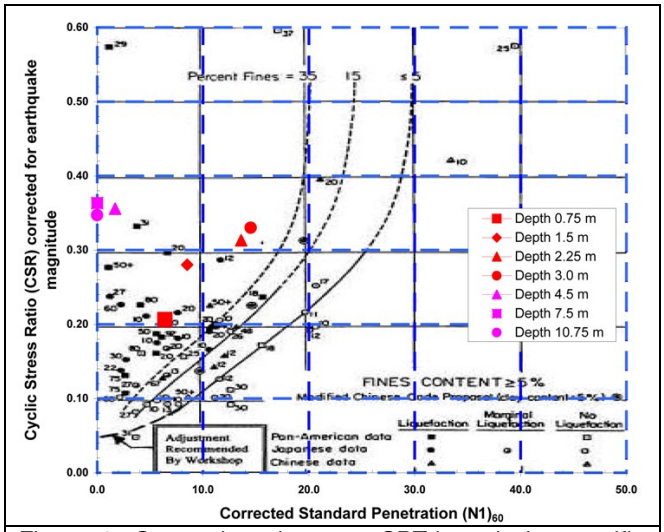

Figure 6. Comparison between SPT-based site specific data and published case histories (Idriss and Boulanger, 2008).

4 DISCUSSIONS

The presented data indicate that both the V_s-based and SPT-based analyzes show the potential for liquefaction at the investigated Site. However, discrepancies are observed in the results, i.e. different thickness of liquefiable layers as well as different factors of safety against liquefaction. Following discussions are provided to explain the possible sources of discrepancies.

This study is based on ground motions with probabilities of 2% in 50 years (1 in 2475 years) corrected for local soil conditions. Neither NBCC 2005 nor Canadian Foundation Manual (CFM 2006) has

explicit comment on the selection of earthquake magnitude and corresponding peak ground acceleration for liquefaction analysis. The procedures followed in this study for choosing the site specific PGA, are in accordance with industry recommendations (Task Force Report, 2007). However, considering the differences of the soil profiles, seismicity and liquefaction potential of eastern and western Canada the used PGA may be conservative. To overcome this problem, Adams and Halchuk (2007) suggest using ground motions with probabilities of 1 in ~1400-1500 years for eastern Canada events. This means that for liquefaction analysis a reduced PGA can be used. Considering the historical evidence of liquefaction in the area, this approach provides more realistic results; however following this procedure the significance of the provided PGA in the code can be questioned. Rather than using a different probability, a more prudent approach is to modify the magnitude scaling factor (MSF) in equation [2] (or the equivalent for SPT-based evaluation) and specific MSF functions be developed for different regions to account for the differences in the frequency content and magnitude of earthquakes and liquefaction potential. This approach will avoid the confusions due to the use of different probabilities and can provide a uniform basis for liquefaction analysis among practitioners. More research work on this topic and the consequences of different approaches should be undertaken.

One of the contributing factors in the observed discrepancies is the differences in the field measurement methods. Theoretically, two main differences exist between the V_s and SPT measurements as follows:

- Borehole investigation is a destructive technique and provides localized subsurface information. On the other hand, MASW uses the data obtained from multiple geophones to generate the V_s profile along an investigation line; therefore it provides average properties within a soil deposit. Comparing the SPT 'N' values measured within the upper 4 m of the subsurface soil at the location of boreholes BH1, BH2 and BH3 (Figure 2) it is observed that some variabilities is expected across the site. Due to the averaging nature of V_s measurements using MASW, these measurements may be reflective of this variability. Thus, the observed discrepancies between the Vs- and SPT-based analyzes at depths above 4 mbgs could be a realistic indicative of the variabilites at the Site. Also, the variability of the soil conditions as observed by V_s versus SPT measurements could be the reason for the difference in obtained safety factors.
- SPT measures soil mechanical properties at large strains whereas shear wave velocities are measured at low strains. Therefore, shear wave velocities are not as sensitive to the cyclic behaviour of saturated soil during liquefaction (large strain) as SPT measurement is.

It is noted that the extents of the discrepancies that should be expected from the above reasons are not very well understood.

A second reason that can be mentioned for the observed discrepancies is the assumption for limiting shear wave velocity, i.e. V^*_{S1} in equation [2]. Due to dilative behaviour of dense soils, it is commonly believed that soils with large SPT or V_s values will not liquefy. However, there is not a consensus on the threshold of soil density that distinguishes between liquefiable/non-liquefiable soils. The analysis presented herein is based on the assumption made in Andrus and Stokoe (2000) that for clean sands (fines ≤ 5%) liquefaction is not possible above corrected SPT 'N' ($N1_{60}$) value of 30. They also estimated that this value for $N1_{60}$ is equivalent to a shear wave velocity of 210 m/s. Based on this estimation they provided the relationships in equation [3] for the limiting shear wave velocities distinguishing liquefiable/non-liquefiable soil layers, i.e. sands with fine contents larger than 35% will not liquefy if their V_{s1} is larger than 200 m/s.

Review of the results presented herein shows that the discrepancy between the V_s- and SPT-based factors of safety between depths of about 2 to 4 mbgs is a result of the applied noted limitation on shear wave velocity. This implies that the limiting values of equation [3] may not be applicable to the type of soils encountered in this study. Comparisons between measured V_s and corresponding SPT values at various sites in the general Ottawa area show that usually for SPT values of 30 measured shear wave velocities are in the range of 300 m/s (authors experience – not published yet). Therefore, a detail study on the applicability of the V_s-SPT correlations and upper limiting SPT and V_s values for distinguishing liquefiable/non-liquefiable soils is required. To be representative these studies should be carried out specifically on soils generally found in eastern Canada.

To compare the V_S-SPT relations Andrus et. al. (2004) developed graphs that correlate the corrected SPT 'N' values to the corrected V_{s1} values that result in the same CRR. This graph is shown in the background of Figure 7. In this figure both V_s- and SPT based analyzes provide similar predictions if they are plotted on the black solid curve. The points plotted above the curve indicate that the SPT-based analysis provide more conservative result than the V_s-based analysis, and the points that plot below the curve indicate that the V_s-based analysis provide a more conservative result. The coloured dots on the graph show the results obtained from this study. It is observed that almost all the points are plotted above the trend line, indicating the more conservative results from the SPT-based analysis. This observation also confirms that the general correlations between V_s and SPT values used in this study may not be valid for the soil conditions encountered at this Site.

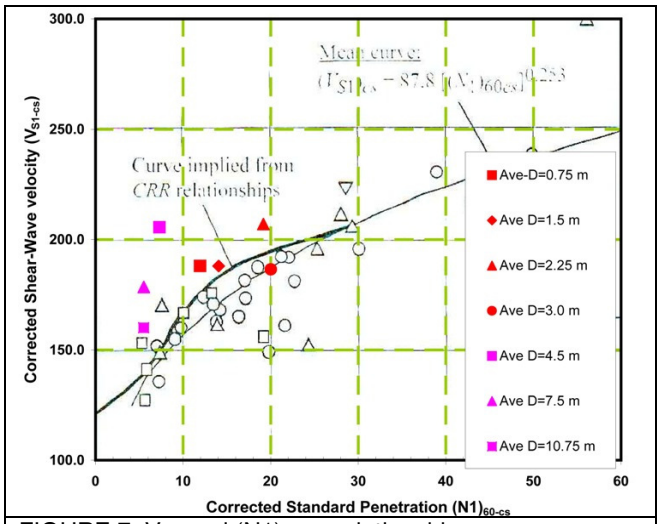

FIGURE 7. V_{S1} and $(N1)_{60\text{-}CS}$ relationships

5 CONCLUSIONS

A case is presented where borehole investigations were combined with shear wave velocity measurements using MASW for liquefaction assessment at a Site in Ottawa. The V_s-analysis showed potential for liquefaction within 2 to 12 m of ground surface, which was compatible with the SPT-based analysis.

This study shows that liquefaction assessment using shear wave velocities is a promising tool that can be used in preliminary investigations and site assessments. Due to the more reliable data bases, SPT-based evaluations should verify the results. However, it is noted that V_s measurements can take into account the variability of soil conditions across the sites. This variability can have a significant effect on the extents of the damages resulted from a liquefaction incident. Therefore, it is prudent to consider the V_s-based evaluations as complementary to the SPT-based evaluations.

The choice of the size of the seismic events that is required to be considered in liquefaction assessments in eastern Canada is a matter of dispute among the practitioners. The use of recommended PGAs in NBCC 2005 results in assessments that are not very much compatible with historical data. Therefore, it is recommended that specific magnitude scaling factors be developed for different regions to account for the variability's in seismic events and liquefaction potentials.

This study shows that the evaluations based on SPT-data provided a more conservative result than the ones based on shear wave velocities. Suggestions are made to modify the existing evaluation methods to consider the local soil characteristics in developing SPT-V_s correlations.

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