Evaluation of liquefaction potential for a dam using combination of traditional and performance based seismic design methods

Hassan Ali & Ali Nasseri-Moghaddam
GHD Limited, Waterloo, Ontario, Canada

ABSTRACT
Liquefaction potential assessment of an earth dam is a multidisciplinary problem which involves correct evaluation of subsurface conditions of fill material and dam foundation, seismicity of the area, and evaluation method. Single scenario method for liquefaction assessment are useful however, the assessment techniques using performance based approach are a promising tool in correctly identifying the risk of dam due to liquefaction.

A case study is presented where a combination of deterministic, simple probabilistic and performance based methods are successfully used to identify the liquefaction potential of a dam. Preliminary geotechnical investigation showed presence of loose sand layers at the foundation of the dam. Considering the high hazard potential classification for the dam and for a return period of 1 in 10,000 years, the liquefaction analysis showed that loose sandy layer may liquefy in case of extreme seismic event. The initial analysis was carried out using a single scenario approach. Thus, a second phase of geotechnical investigation was completed with additional deep boreholes and CPT testing to identify the thickness of loose sandy layers. A detailed liquefaction assessment was completed using a combination of deterministic, probabilistic and performance based approaches. The results of the second phase showed that existing loose sandy soil layers are not wide spread and have limited thickness. For dams, simple deterministic method is not enough and probabilistic and performance based approach for liquefaction analysis show promising results.

RÉSUMÉ
L'évaluation du potentiel de liquéfaction d'un barrage en terre est un problème multidisciplinaire qui implique une évaluation correcte des conditions souterraines du matériau de remblai et de la fondation du barrage, de la sismicité de la zone et de la méthode d'évaluation. La méthode à scénario unique pour l'évaluation de la liquéfaction est utile, mais les techniques d'évaluation utilisant une approche basée sur les performances sont un outil prometteur pour identifier correctement le risque de barrage dû à la liquéfaction.

Une étude de cas est présentée dans laquelle une combinaison de méthodes déterministes, probabilistes simples et basées sur la performance est utilisée avec succès pour identifier le potentiel de liquéfaction d'un barrage. Une étude géotechnique préliminaire a montré la présence de couches de sable en vrac à la fondation du barrage. Compte tenu de la classification du potentiel de risque élevé pour le barrage et pour une période de retour de 1 sur 10 000 ans, l'analyse de liquéfaction a montré qu'une couche de sable lâche peut se liquéfier en cas d'événement sismique extrême. L'analyse a été réalisée en utilisant une approche à un seul scénario. Ainsi, une deuxième phase d'investigation géotechnique a été complétée avec des forages profonds supplémentaires et des tests CPT pour identifier l'épaisseur des couches de sable lâches. Une évaluation détaillée de la liquéfaction a été réalisée à l'aide d'une combinaison d'approches déterministes, probabilistes et basées sur la performance. Les résultats de l'étude de la deuxième phase ont montré que les couches de sol sableux lâches à très lâches existantes ne sont pas étendues et ont une épaisseur limitée. Pour les barrages, la méthode déterministe simple ne suffit pas et une approche probabiliste et basée sur les performances pour l'analyse de la liquéfaction doit être complétée.

1 INTRODUCTION

Assessment of the liquefaction potential of subsurface soils needs a detailed geotechnical investigation, laboratory and in-situ 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, proper techniques for evaluation of liquefaction potential is necessary for identifying possible remediation measures in case of ground shaking due to seismic events.

This paper presents a case study where a combination of deterministic, simple probabilistic and performance based methods are successfully used to identify the liquefaction potential of a dam. Preliminary geotechnical investigation showed presence of loose sand layers at the foundation of the dam. Considering the high hazard potential classification for the dam and for a return period of 1 in 10,000 years, the liquefaction analysis showed that loose sandy layer may liquefy in case of extreme seismic event. The initial analysis was carried out using a single scenario approach. Thus, a second phase of geotechnical investigation was completed with additional deep boreholes and CPT testing to identify the thickness of loose sandy layers. A detailed liquefaction assessment was completed using a combination of deterministic, probabilistic and performance based approaches. The results of the second phase showed that existing loose sandy soil layers are not wide spread and have limited thickness

1.1 Liquefaction Evaluation Methods

In the beginning the study of liquefaction was based on cyclic loading tests of reconstituted samples. These tests were very useful for defining the mechanics of liquefaction and giving insight into potential consequences but it was quick realized that such samples were not representative of field conditions and therefore could not be relied upon to assess the liquefaction potential in the field. Attention turned to the possibility of using in-situ penetration tests to assess the density and hence the resistance of soils in-situ to liquefaction. These studies have resulted in the development of liquefaction assessment charts based on SPT-N, CPT-qc and for soils that are difficult to penetrate, charts based on shear wave velocity, Vs. In the early days, site response analysis was not a viable option, so Seed and Idriss (1971) developed a simplified method for estimating the cycles of uniform stress representative of the actual shaking intensity of the earthquake. This approach, despite advances in computation capacity, is still very widely used (Liam et.al. 2014).

1.1.1 Simplified Method

The simplified approach estimates average cyclic shear stress (CSR) caused by earthquake shaking using Equation 1,

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

Where a_{max} is the peak horizontal ground surface acceleration, g is the 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. The CSR is then compared to cyclic resistance ratio (CRR) calculated based on corrected local soil properties such as SPT or Vs measurements.

1.1.2 Magnitude Deaggregation Method

In this case, the magnitudes are collected in bins on 0.25M wide and the central magnitude value is assigned to the bin. The contributions of the bin magnitude to the site acceleration are sampled at various distances from the site. The contributions are given per mil in percent contributions to the site acceleration at the various magnitude-distance combinations are obtained. The total contributions per magnitude bin are obtained by summing the distance contributions horizontally. A 2-D magnitude deaggregation is developed showing the cumulative percent contributions per magnitude bin. The sum of the bin contributions is 100%. The factor of safety of the site at the code acceleration level is computed for each binned magnitude and then multiplied by the contribution of the magnitude to the site acceleration. The sum of all the contribution to the factor of safety gives the global factor of safety for the site.

1.1.3 Performance Based Seismic Design Approach

A performance based seismic design is accomplished by defining appropriate levels of design earthquake ground motions and corresponding acceptable levels of damage. The design earthquake motions include those from frequent events that are likely to occur within the life of the structure as well infrequent or rate events that typically involve very strong ground shaking. The acceptable level of damage is specified in terms of displacement to be experienced by the structure. Damage is categorized in terms of "Performance Categories", which are related to the effort required to restore the full functionality of the structure (Atukorala et.al. 2014).

1.1.4 Factor of Safety

Finally, the factor of safety (FS) against liquefaction is defined by the following relationship:

$$FS = \frac{CRR}{CST} \qquad [2]$$

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

2 PROJECT BACKGROUND AND OBJECTIVE

This section presents a review of the site geology, seismicity of the area and summary of previous investigations at the site.

2.1 Site Description and Geology

The Dam is located between physiographic regions of Southern Ontario known as Oak Ridges Moraine and the South Slope (Chapman & Putnam, 1984). The glacial till encountered at the dam site is known as Newmarket till which is predominantly a silty sand texture. It is common to see various layered deposits of silt, sand and clay till. The bedrock in the area is expected to be generally deep (minimum 80 m to 90 m below existing ground) and comprises shale of the Georgian Bay formation.

The Dam is a homogeneous earth fill dam more than 50 years old, which was constructed to provide flood protection. It is approximately 335 m long and about 7.6 m high. The Dam crest elevation is at 275.5 m Above Mean Sea Level (mAMSL). The upstream (reservoir side) of the dam is typically inclined at between 3.0 and 3.5 horizontal to 1.0 vertical and is protected with some riprap and sod. The downstream slope is inclined at between 3.0 horizontal to 1.0 vertical with majority of the slope vegetated with grass.

2.2 Seismicity at the Dam Site

In general, eastern Canada is located in a stable continental region and consequently has relatively low earthquake activity with an annual average of four seismic events with magnitude of four or more. The Dam site is located within the Southern Great Lakes Seismic zone according to Earthquakes Canada website (http://www.earthquakescanada.nrcan.gc.ca/zones). This zone is shown in on Figure 1 and is identified by low to moderate seismic activities compared to more active zones to the east of the area. In general the probability of major seismic events (seismic events with magnitudes of five or more) in the area is low. As a result, and to the authors best knowledge, there is no recorded seismically induced liquefaction in the general Southern Great Lakes zone and in particular in the vicinity of the Dam site. However, in specific cases the risk of seismic events can be considered high due to the consequences of failure of the structures as defined in Canadian Dam Association (CDA), CDA 2007/13. It is our understanding that the Dam has a high consequence category due to potential damage that may occur downstream of the dam as a result of a failure.

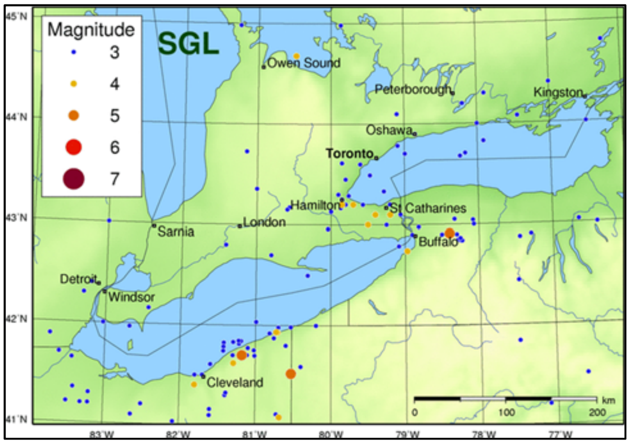

Figure 1: Seismic zone of Southern Ontario (http://www.earthquakescanada.nrcan.gc.ca/zones)

2.3 Previous Investigations at the Site

Previous studies, hereby referred to as Study-1 (Std1-2013) and Study-2 (Std2-2015) assigned a very high hazard potential classification to the Dam as per the requirements of Ministry of Natural Resources and Forestry's (MNRF) Lakes and Rivers Improvement Act (LRIA) and CDA Guidelines. During these studies, eight boreholes were installed on the dam crest and embankment slopes. These boreholes were advanced using hollow and solid stem augers and soil samples were recovered using standard split spoon (SPT) methods. Reportedly three of the boreholes encountered loose/very loose sandy layers at approximate elevations of 269.27 mAMSL, 267.7 mAMSL and 268.1 mAMSL, respectively. The thickness of this layer was identified to be about 0.61 m, 2.13 m and 3.1 m at the borehole locations, respectively. The geotechnical studies at the site provided the following comments regarding the potential for liquefaction at the Dam site:

- Std1-2013 concluded that the loose sandy layers may liquefy during extreme seismic events. However, they concluded that due to low probability of such extreme seismic events, the probability of liquefaction in the area is very low and therefore the potential for damage due to liquefaction is negligible. It is noted that this report recommended the use of seismic parameters that were specified in the old versions of the Building Code.
- Std2-2015 concluded that the applicable seismic event for the liquefaction analysis should have a return period of 1 in 10,000 years (0.0001 per annum). The applicable Hazard (peak ground acceleration-PGA) was calculated based on extrapolation of the hazards provided in the most recent version of the National Building Code of Canada (NBCC 2015), however details of the calculations are not provided. This analysis showed that this layer might liquefy in some areas during an extreme seismic event. The liquefaction analysis was carried out using a single scenario approach using Youd et. al. (2001) method. DDR 2016 concluded that if liquefaction occurs at the Site it may significantly impact the Dam functionality and it might result in overtopping. Therefore, they recommended a detailed liquefaction study be completed to verify the condition at the Dam site

Based on the review of the above two studies, a detailed evaluation of the seismic conditions were conducted at the Dam site and provided recommendations with respect to potential liquefaction of the loose sandy soils. Figure 2 shows the location of the boreholes, CPT, and MASW testing surveys.

3 GEOTECHNICAL INVESTIGATION AND INTERPRETATIONS

3.1 Field Investigation

A total of four boreholes were drilled along the crest of the dam as shown on Figure 2. The boreholes were advanced to approximate depths of 10.36 to 10.52 m below the existing ground (mbgs). Mud rotary technique was utilized for drilling activities to minimize the potential for basal heave and soil disturbance during the drilling work.

A total of seven cone penetration tests (CPT) and three seismic cone penetration tests (SCPT) were completed to the depths where the machine encountered refusal to advancement as shown on Figure 1. CPT17-01, CPT17-02C, CPT17-03 and CPT17-04 were advance on the downstream side of the dam. The objective of installation of these soundings was to assess whether potentially liquefiable layers exist in the general are or not. SCPT17-06, SCPT17-07 and SCPT17-08 were installed from the top of the crest of the dam.

A seismic multichannel analysis of surface waves (MASW) test was done on the dam crest and toe of the dam. Four MASW lines were done to measure the shear wave velocity within the upper 30+ meter at the Site as shown on Figure 2.

3.2 Subsurface Soil Stratigraphy

In general, the subsurface consists of fill deposit overlying native till deposits which were encountered to the termination depth of the boreholes. Figure 2 shows the profile of the subsurface stratigraphy at the Dam site. The profile also shows the stratigraphy from previous two studies (Std1-2013 and Std2-2015).

The fill materials were encountered in all boreholes. This layer was about 9.75 m thick and its bottom extended to elevations from 265.21 m AMSL to 265.71 m AMSL. The deposit consisted mainly of clayey silt to silt to some sand and gravel. SPT 'N' values measured within this deposit varied from 2 to 62 blows per 0.3 m of penetration indicating the very loose to dense to stiff conditions.

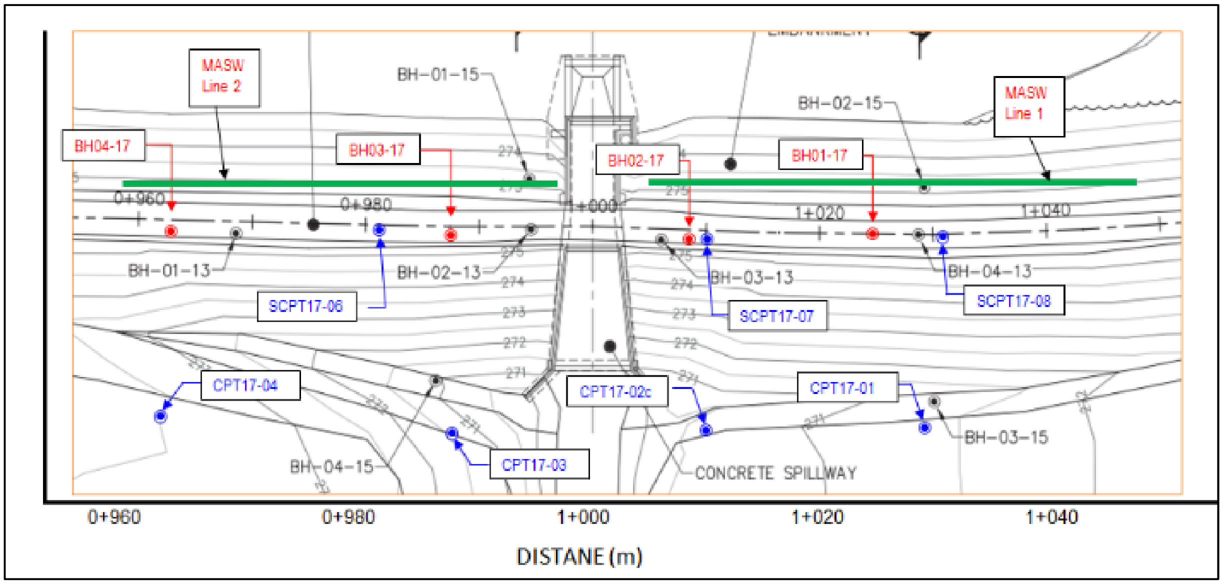

Figure 2: Borehole locations, CPT, SCPT, and MASW survey lines plan

Glacial till was encountered beneath the fill in all boreholes. The boreholes were terminated within the till material. The till is sandy silt to clayey silt in nature with some gravel. Moisture content tests carried out on samples from this deposit resulted in 10 percent moisture by weight indicating the moist conditions. SPT 'N' values measured within this deposit varied from 45 to over 50 blows per 0.3 m of penetration indicating the generally very dense/hard conditions of this deposit.

Figure 3 shows the profile of the soil stratigraphy showing the elevations and distance for all boreholes drilled at the Dam site.

3.3 CPT test interpretations

Four CPT soundings were completed at the downstream area of the dam (CPT17-01, CPT17-02C, CPT17-03 and CPT17-04). These soundings were terminated between 7.6 to 9.6 mbgs. Based on the CPT soundings the inferred soil layers behave more like low plasticity silt/fine grain material. A layer of material with sand behavior was observed interbedded between the silt like material. This layer is generally observed from 3 to 7 m bgs. The inferred undrained shear strength of the silt like material is in the range of 50 to 100 kPa. The inferred N60 values within the sand like material show values larger than 20 blows per 0.3 m of penetration. Based on these interpretations the existing material downstream of the dam mainly consist of silty material with stiff to very stiff consistency and the sand like material are in compact to dense state of compactness. As these soundings are installed downstream of the dam they are expected to be representative of the native material in the area which the dam is founded on. Based on these interpretations no widespread loose/soft soil layers are observed in the downstream of the dam.

Soundings SCPT06-17 to SCPT08-17 were advanced from the top of the crest of the dam. The interpreted layers are behaving more like sandy material throughout the depth of the soundings. The interpreted N60 values within the embankment (upper 5 m) and native material vary from 5 to 60 blows per 0.3 m of penetration, with the majority of the values larger than 10 blows per 0.3 m of penetration. Therefore, in general the material is in compact to very dense state of compactness.

Based on the interpretation of the CPT/SCPT data it is concluded that no widespread loose/soft soil zones exist in the area, however sporadic loose/soft zones may exist at some locations.

3.4 Shear Wave Velocity interpretations

Shear wave velocity measurements were completed by two methods, i.e. MASW and SCPT. MASW is completed along an investigation line, therefore it provides the average velocities of a volume of soil. SCPT is completed within a hole and therefore it represents local shear wave velocities within the hole. In general measured shear wave velocities less than 200 m/s are interpreted as loose/soft soil conditions.

Two MASW investigations were completed along the crest of the dam (Lines 1 and 2) and all the SCPT tests were completed from the crest of the dam. The variation of shear wave velocities vs depth obtained from these investigations are shown on Figure 2. Based on these data the shear wave velocities within the embankment vary from 230 m/s to over 500 m /s. The shear wave velocities within the native material (assumed below 7 m bgs) vary from 300 m/s to over 1000 m/s. These results show that in general the material is in compact condition at the investigated locations.

MASW investigation line 3 was completed at the downstream area of the Dam. The measured shear wave velocities vary from less than 300 m/s to about 800 m/s. These results show that the material is in compact condition.

MASW line 4 was completed along the reservoir bank in the proximity of the dam upstream area. The measured shear wave velocities are in the range of 300 m/s to over

800 m/s indicating the compact nature of the material. Within the depth of 5 m along this line shear wave velocities less than 200 m/s are recorded indicating the loose nature of this layer.

In general the measured shear wave velocities are in line with the borehole and CPT data. These data indicate that there are localized loose/soft soil layers in the area, however these layers are not wide spread. Figure 3 shows the Vs-profiles of the MASW and SCPT completed at the crest of the dam.

Based on this subsurface interpretation a detailed liquefaction evaluation was conducted. The results of this are presented next section.

4 LIQUEFACTION POTENTIAL EVALUATION

4.1 Single Scenario and Magnitude Deaggregation Method

The calculated seismic hazards for the site, as per National Building Code of Canada (NBCC 2015) provided by National Research Council Canada (NRCC) for different probabilities were obtained. These values are provided for firm ground (site Class C). Based on the measured shear wave velocities at the Dam site (MASW analysis), a site class C is associated to this Dam site. Therefore, the obtained hazard values from NBCC are not required to be modified.

Since a multidisciplinary approach was used in this study, In the first step, FS values using a single scenario earthquake, based on SPT'N' values were obtained for all boreholes using the approach presented in Equation 1. The PGA values were obtained from the Hazard Canada calculator for a return period of 2% in 50 years.

In the second step, FS values for liquefaction potential were obtained using the magnitude deaggregation method. The curves were calculated for distance between 0 to 500 km from the site and for magnitude from 4.5 to 8.0, respectively. NRCC provides seismic hazard values for events with return periods of up to 2475 years. To obtain the hazards for larger return periods a linear interpolation on a log-log scale is utilized. The hazard values utilized in this study are summarized in Table 1.

Table 1: Analysis Parameters for Different Earthquake Shaking Levels

EQ Level	Return Period	APE	PGA [g]	Mean [Mw]	Median [Mw]
EQL 1	1:476	0.00210	0.029	6.22	4.85
EQL 2	1:2476	0.00040	0.097	6.27	5.45
EQL 3	1:5000	0.00020	0.162	6.29	5.73
EQL 4	1:10000	0.00010	0.270	6.31	6.02

The National Building Code of Canada specifies that buildings and their components should be designed for seismic events with 2% probability of exceedance in 50 years (i.e. return period of 2475 years). However, the Code does not specifically comment on the appropriate seismic event for landfills, earth structures or dams. In accordance to CDA 2007/13 and OMNR (MNR 2011) guidelines dams with assigned consequence of Very High or more should be designed/checked for seismic events with return periods of 5000 to 10000 years. In this study we completed this assessment for four (4) different return periods as discussed below.

4.2 Performance Based Approach

In this approach the following Design Earthquake Ground Motions are considered:

- Earthquake Shaking Level 1 (EQL-1): 1:476-yr return period ground motions that are equivalent to having a 10% chance of exceedance in 50 years or 21% chance of exceedance per annum.
- Earthquake Shaking Level 2 (EQL-2): 1:2476-yr return period ground motions that are equivalent to having a 2% chance of exceedance in 50 years or 4% chance of exceedance per annum.
- Earthquake Shaking Level 3 (EQL-3): 1:5000-yr return period ground motions that are equivalent to having a 1% chance of exceedance in 50 years or 0.02% chance of exceedance in per annum.
- Earthquake Shaking Level 4 (EQL-4): 1:10000-yr return period ground motions that are equivalent to having a 0.5% chance of exceedance in 50 years or 0.01% chance of exceedance per annum.

5 LIQUEFACTION STUDY RESULTS

The liquefaction results were calculated for boreholes from this study as well as for previous studies. It is noted that due to the methodologies used in the previous drilling activities the results from these boreholes are less reliable as the SPT values may have been completed in the disturbed soil conditions due to basal heave/instability in the boreholes. The calculated factors of safety against liquefaction for each of the cases are shown at the corresponding depths in Figures 4 to 7. If the factor of safety is less than 1 (liquefiable condition) the FOS is shown in red. Further, in case that the soil is liquefiable the corresponding evaluated settlement (in mm) is shown below the FOS value in green.

Figures 4 and 5 show the factors of safety against liquefaction calculated for loading condition EQL-3. Figures 4 and 5 show the FOS for earthquake magnitudes of 5.73 and 6.29 corresponding to median and mean earthquake magnitudes for seismic events with a return period of 5000 years (Table 2). As per these results under loading condition EQ-3 all factors of safety are larger than 1 and liquefaction will not occur. Based on the same analysis it is concluded that under loading conditions EQL-1 and EQL-2 (seismic events with smaller hazards) liquefaction will not occur at the site.

Figures 6 and 7 show the FOS against liquefaction for loading condition EQL-4 (seismic event with return period of 10000 years). Figure 6 corresponds to earthquake magnitude of 6.02 and Figure 7 corresponds to earthquake magnitude of 6.31 (corresponding to median and mean magnitudes – Table 2). According to these results there is a potential for soil liquefaction at some of the boreholes.

The calculated settlements range from 25 mm to 82 mm. It is noted that the calculated settlements within the boreholes completed by GHD (BH01-17, BH02-17 and BH03-17) vary from 28 mm to 37 mm and no liquefaction occurs in borehole BH04-17. The main reason that larger settlements are calculated for boreholes from previous studies is that they reported thicker liquefiable soil layers, mainly because they used larger sampling rates (lower vertical resolution compared to GHD study). Therefore, it is our conclusions that settlements calculated at the boreholes installed during this study are more representative of the site conditions. In conclusion we envision that in case of occurrence of extreme seismic events (return periods of 10000 years) maximum settlements in the range of 40 mm may occur. This settlement is not expected to interfere with the Dam performance and therefore is within the acceptable range for loading condition EQL-4. Further, as the liquefiable soil unit does not seem to be continuous and widespread across the site the potential for massive instabilities in the area is very low.

6 SUMMARY AND CONCLUSIONS

A case study is presented where single scenario approach for evaluation of liquefaction was combined with magnitude deaggregation method and performance based design approach for a dam site in southern Ontario.

The initial geotechnical investigation showed the presence of loose sandy layer but the borehole drilling program was not properly completed to show the thickness of that layer. In this study, the geotechnical program was re-evaluated with deep boreholes to properly characterize the loose sandy layer and it was concluded that the loose sand layer is a shallow layer and it is scattered under the foundation dam.

A detailed liquefaction assessment was conducted at the site. The results of this study show that there is no historical evidence that ground shaking with sufficient energy and cycles occurred within 500 km of the Dam. Also, there is no historical evidence of liquefaction in the general area of the Dam. The use of performance based seismic design (PBSD) compared to traditional method with result in more realistic outcome. The liquefaction analysis was completed for three levels of performance and four levels of ground shaking (EQL-1 to EQL-4). The ground shaking parameters were calculated based on extrapolation of the hazard values provided by NRCC with AEP of 1/2475, 1/5000 and 1/10000 years.

Based on the results of the liquefaction analysis, the liquefaction may only occur at sporadic locations during an extreme seismic event (EQL-4). The settlements calculated for the liquefied soils are all within the acceptable range for the defined performance level.

7 REFERENCES

Atukorala U., Hawson H., Mylleville B., Williams R. 2014. Seismic Design Guidelines for Dikes 2nd Edition. Ministry of Forests, Lands, and Natural Resource Operations Flood Safety Section. Golder Associates.

Chapman, L.J. and D.F. Putnam. 1984. Physiography of Southern Ontario. 1973. Special Volume No.2. Toronto, Ontario: Ontario Geological Survey. 270 p.

Finn W.D.Liam., Wightman, A., and Sherstobitoff, J. 2014. Evaluation and mitigation of liquefaction hazard for engineering practice. EERI Short Course – Utah Chapter.

Idriss I.M., and Boulanger R.W., 2008. Soil liquefaction during earthquakes. Monograph MNO-12, Earthquake Engineering Research Institute, Oakland, CA, 261 pp.

Youd, T., Idriss, I., Andrus, R., Arango, I., Castro, G., Christian, J., Dobry, R., Finn, W.D.L., Harder, L., Jr., Hynes, M., Ishihara, K., Koester, J., Liao, S., Marcuson, W., III, Martin, G., Mitchell, J., Moriwaki, Y., Power, M., Robertson, P., Seed, R., and Stokoe, K.H. (2001). Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils. ASCE Journal of Geotechnical and Geoenvironmental Engineering, 127(10): 817–833.

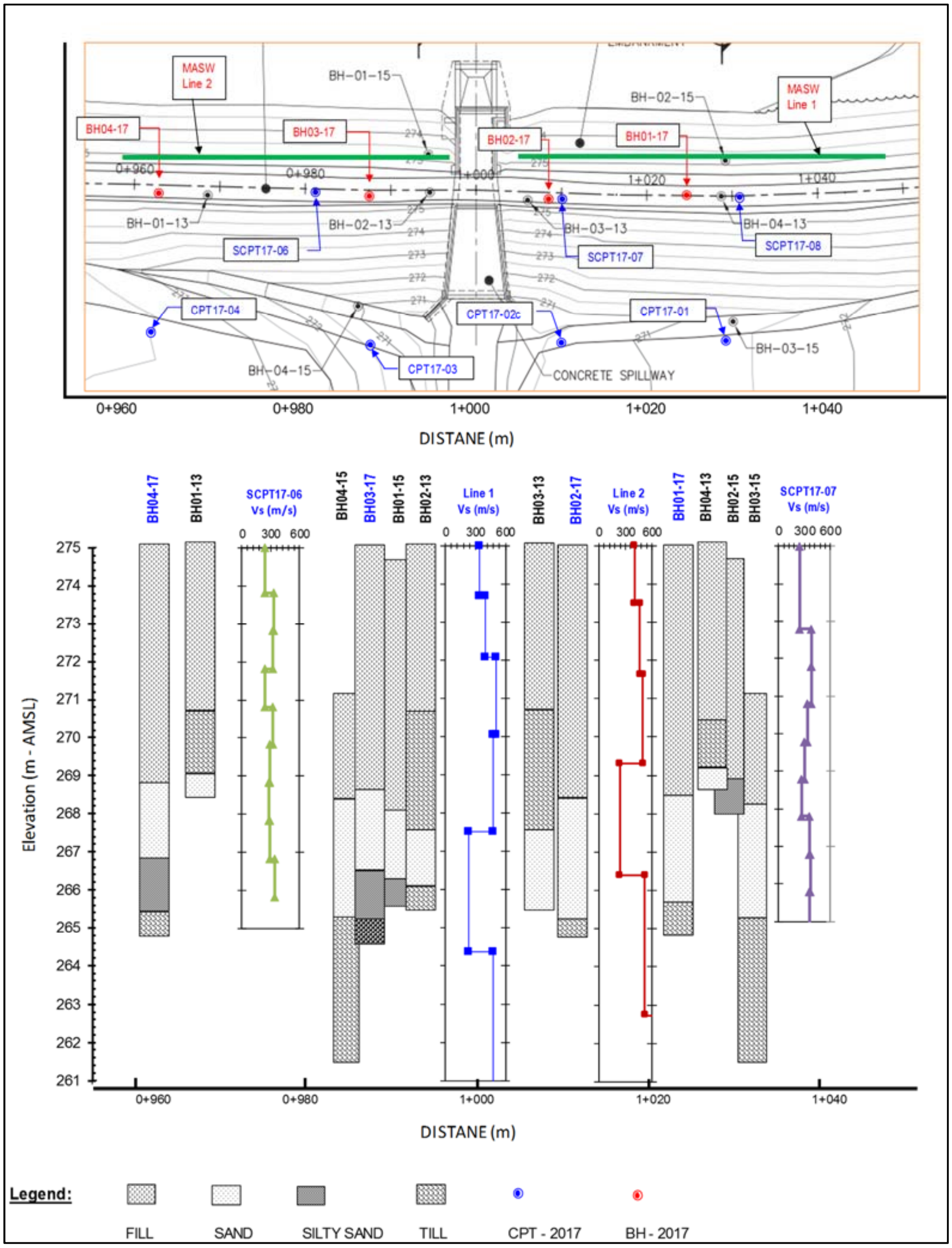

Figure 3: Soil Stratigraphy section showing subsurface soil, SCPT and MASW Vs-profiles.

.

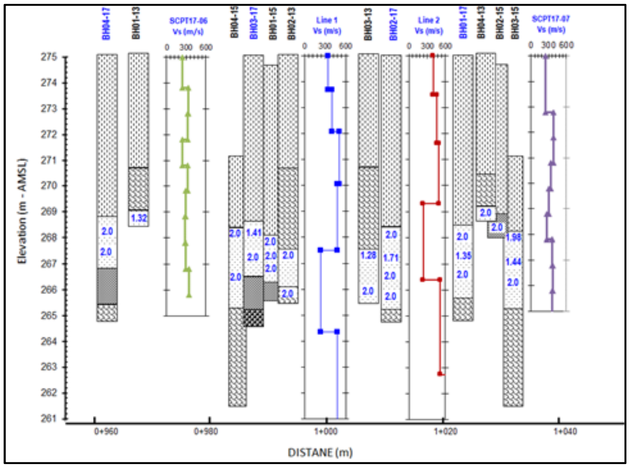

Figure 4: EQL-3 Factor of safety against liquefaction for Magnitude of EQ – 5.73

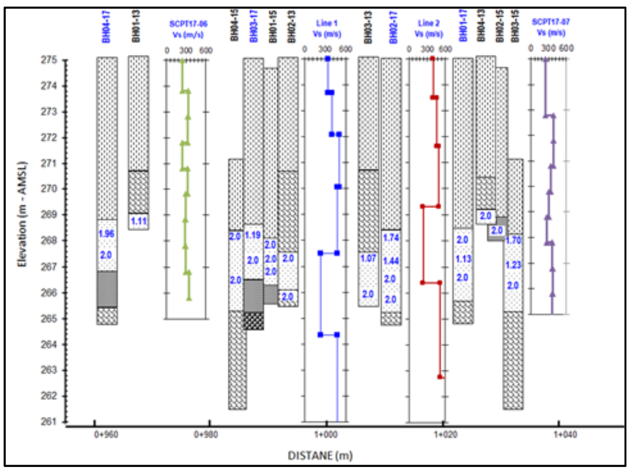

Figure 5: EQL-3 Factor of safety against liquefaction for Magnitude of EQ – 6.29

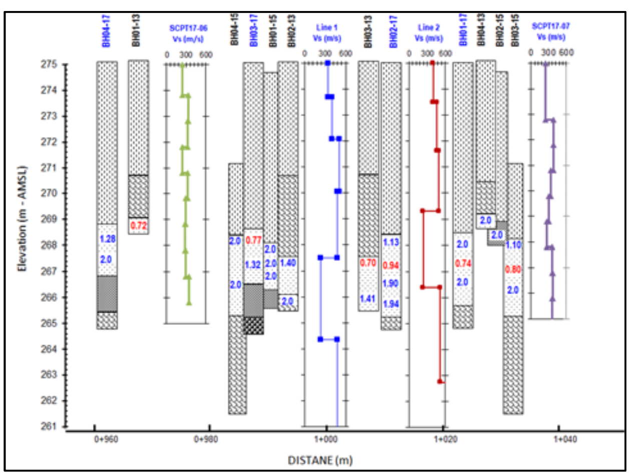

Figure 6: EQL-4 Factor of safety against liquefaction for Magnitude of EQ – 6.01

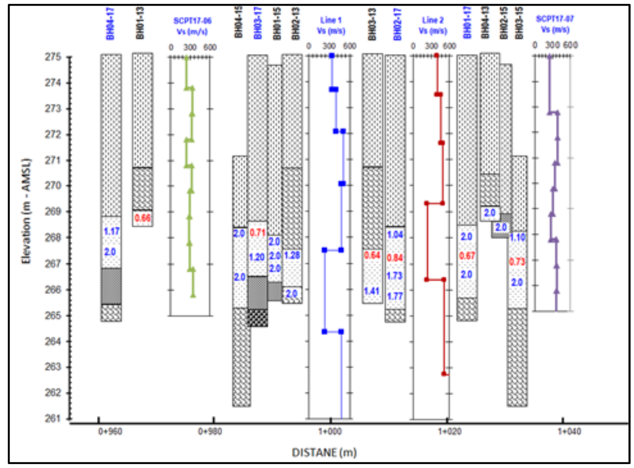

Figure 7: EQL-4 Factor of safety against liquefaction for Magnitude of EQ – 6.31

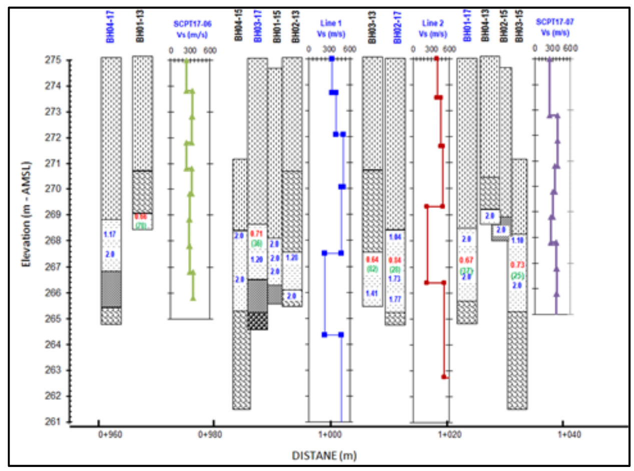

Figure 8: EQL-4 Settlement values for loose sandy layer for Magnitude of EQ – 6.31. Blue values shows FS >1, Red values shows FS<1 and green values show Settlement of layer for FS<1 in mm.