Review of Professor Kenneth Torrance report regarding the Boundary Road Landfill Site Project, Ottawa

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By

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1. Introduction: I have been requested by Taggart Miller Environmental Services to provide my opinion on Prof. J.K. Torrance's report dated May 16, 2016 on the proposed Capital Region Resource Recovery Centre facility pertaining to geotechnical matters in relation to the Boundary Road Site and the project. In this report I provide my key comments and opinions on the main issues raised by Prof. Torrance. I also provide my own related conclusions.

2. Qualifications and Expertise: I graduated in 1973 at the Université du Québec à Montréal in Earth sciences. I then obtained my Master thesis in 1976 at the Department of Earth Sciences of the University of Waterloo, Ontario and the title of my thesis was *Quaternary geology of the Baie-des-Sables/Trois-Pistoles area with some emphasis on the Goldthwait Sea clays*. After two years in rock mechanics at the University of Alberta I went to the University of Sherbrook where I obtained my Ph. D. in Civil Engineering in 1982. The title of my thesis was on the *Origin of the structuration of sensitive clays in Eastern Canada*. Since 1981 I have been a professor at Laval University at the Department of Geology and Geological engineering. During my career I had the opportunity to work on the nature (including microstructure) and geotechnical characteristics of marine sediments from around the world, including England, Norway, Egypt, Singapore, Korea, Thailand, Japan and in the US. My main research has been and continues to be on understanding the various processes involved in the development of strength in clays and their evolution with time, along with their failure and post-failure behaviour. I had the opportunity to work on raised marine sediments, such as in the Champlain Sea clays, but also in many active sedimentary environments like the Saguenay Fjord, the St. Lawrence Estuary, the coastal areas in the Atlantic and Pacific oceans of North America and also in the Mediterranean and Adriatic Seas. My contributions to science and engineering in this field at the international level have been recognized by the K. Y. Low medal (2005), and the R.F. Legget Medal (2015) for my contribution to the geotechnical engineering profession in Canada. I also received the Schuster medal (2013) recognizing the contribution of my research on landslides, particularly for submarine landslides.

3. Site stratigraphy and subsurface conditions at the proposed landfill site

What follows is from my own review of the general geotechnical profile in Figure 1 below and CPT profiles (Appendix B, in Volume III) and pushed continuous soil core photographs in Appendix I (Volume III) of the EASR prepared by Golder Associates Ltd. (Golder) in support of the project. Golder carried out the subsurface investigation program and completed the geotechnical engineering for this project, as well as the related geological and hydrogeological assessments.

At the base of the soil profile there is a till deposit up to 8 or 9 metres thick overlain by a sandy discontinuous glaciofluvial deposit less than 1m thick likely marking the retreat of the glacial ice from the eastern Ontario area. As the ice retreated, the Champlain Sea quickly invaded the area depositing marine sediments over a thickness of about 30 m. The marine sediments here, according to the photographs, consist of grey banded clays (varve-like feature) with alternating grey layers of clay and silt with reddish colour bands near the top of the section. This would correspond to Unit II of Gadd (1985) who provides the following description of this Unit: '*In most sections it constitutes regularly laminated sediments with the appearance of typical varves (diatactic) comprising coarse to fine silt and silty clay*' (p. 5 of Gadd, 1985).

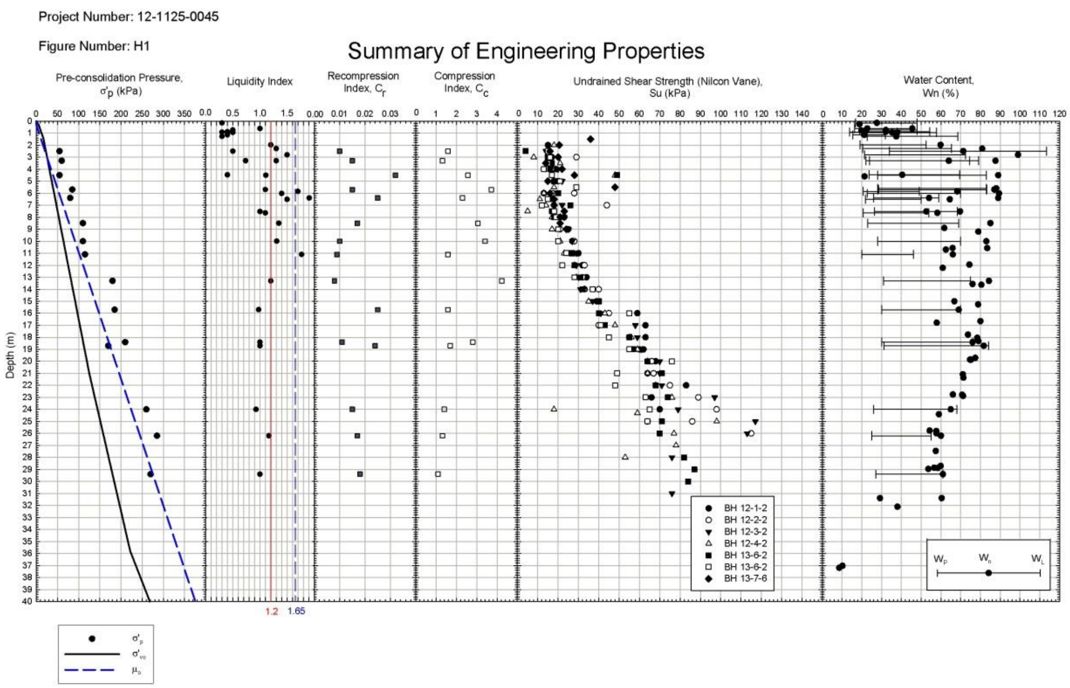

Figure 1. Modified geotechnical profile found in Appendix H, Figure H1 providing the actual data on the liquidity index and specific lines for values of 1.2 and 1.65.

North of the Boundary Road site, Aylsworth et al. (2003, p. 58) describe a similar deposit and mention that '*The marine sequence records an upward change from a deep water, high-salinity, marine environment to estuarine conditions (Gadd 1988). Massive to weakly stratified, gray* *marine clay at the bottom grades into a coarsening-upward prodelta*[1] *sequence of rhythmically bedded red and gray clay with thin silt bands and occasional silt or fine sand layers commonly less than 20 cm thick. Commonly known as Leda Clay, the marine sediments are geotechnically sensitive clayey silts and silty clays, mainly composed of non-clay minerals (glacial rock flour)*. Apart from some sand layers in the prodelta clays, Aylsworth et al. (2003) and Gadd (1985) do not mention the presence of sand layers in the underlying marine sediment. At the Boundary Road site, near the top of the profile, and at a depth of about 5 m, there is a continuous and less than 0.6 m thick silty layer that would be part of the prodelta sequence. According to the photographs, the bedded red and grey clay corresponding to a prodelta sequence, is found down to a depth of about 6 m.

From the 25 CPT profiles, the Leda clay profile below the prodelta sequence presents a very uniform stratigraphy with only very few very thin (less than 0.1 m thick) silty layers (I counted a maximum of 4 below), which are not continuous on the site. This corresponds with the analysis of Quigley et al. (1989) on some samples obtained from Geocon, which concluded that the samples they analysed were made of alternating layers of silt and clay with variations in water content from 35% (silty portions) to 90% (clay portions). In addition, according to Golder, no sand layers have been observed in the various continuous soil cores of the Leda clay sequence below the prodelta sediments on the Boundary Road site.

Opinion #1: The subsurface stratigraphy at the Boundary Road site is quite uniform throughout the area. Below the continuous silty layer at a depth of about 5 m, the rest of the Leda Clay profile is quite uniform (banded clay) with only a few (about 4) silty layers that are not continuous.

Opinion #2: Contrary to comment #3 of Prof. Torrance's report, and also mentioned elsewhere in his report, no evidence of sand layers was found in the data that would indicate that there are '*silt-dominated and sand-dominated layers within the 30 m of Leda clay*'. The Geocon work did not include nearly as many boreholes as completed by Golder, and did not include any CPT test profiles through the clay deposit as was carried out by Golder. It is my conclusion that the silty clay layers between a depth of 6 m and 30 m consist of alternating layers of silt and clays. If there are thin silty layers they are discontinuous as shown by the 25 CPT profiles and continuous soil sample photographs.

4. Leda clays and the presence of quick clays

In Québec the term "Leda clays" is not in use. The term used is simply 'sensitive clays', classified from low to extra-sensitive clays. Since the late 1970's the geotechnical community have developed a large body of knowledge on building on sensitive clays as illustrated by the book of Leroueil et al. (1990) related to embankments on soft clays that are highly compressible.

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[1] A prodelta can be defined as the furthest offshore portion of a delta, lying at the toe of the delta front, and characterized by a relatively slow rate of fine-grained deposition.

Postglacial sediments in Eastern Canada, as pointed out by Prof. Torrance, have a great diversity both in terms of index properties and sensitivity depending on their evolution since deposition and the degree to which they were deposited in salt water and subsequently leached. In Eastern Canada, postglacial marine sediments, as indicated by Prof. Torrance, have a similar mineralogy to the Norwegian clays but in my opinion and based on my work, they compare to the lower end of the spectrum of sensitive clays in Eastern Canada with regard to the plasticity index, i.e., Norwegian clays are generally coarser than Leda clays.

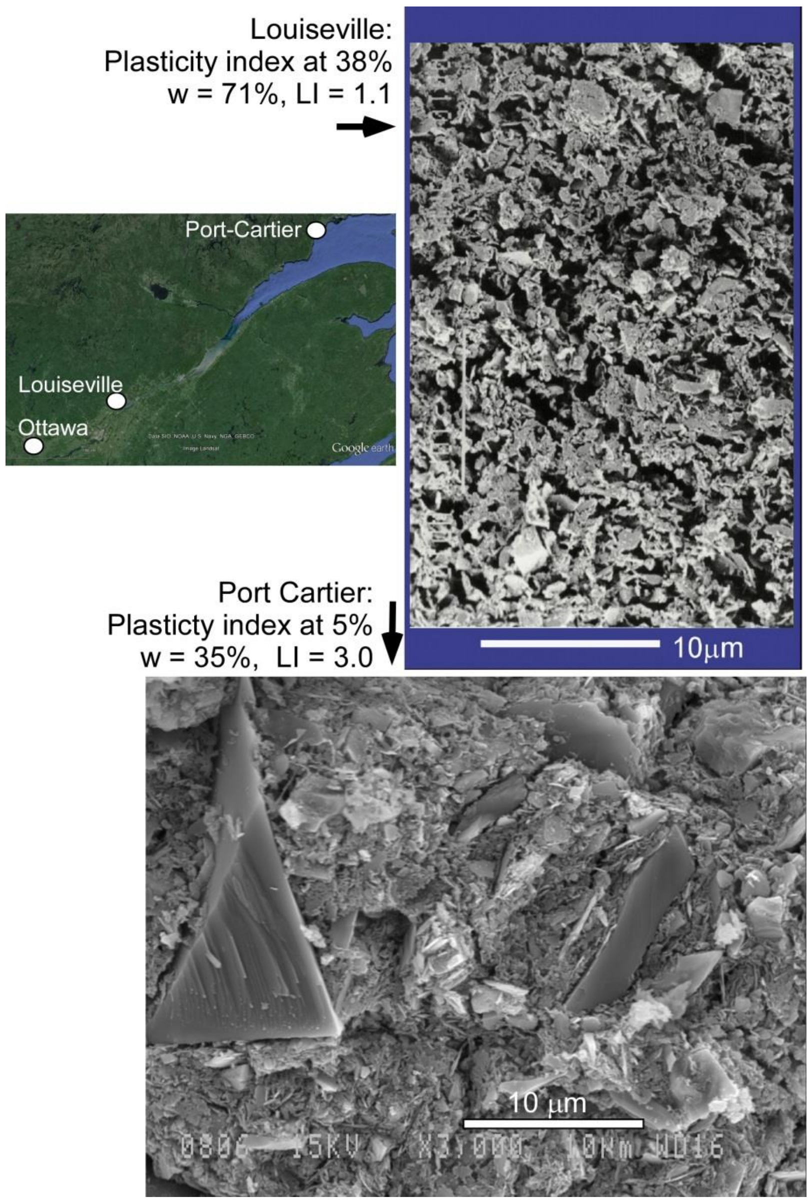

Figure 2. Microphotographs of marine sediments at Louiseville and Port-Cartier shown at about the same scale (white bar at 10 microns). Note the very great difference in the grain size of both samples, the Port-Cartier sample being much coarser (see the triangular quartz grain to the left) than the Louiseville sample. [w = water content; LI = liquidity index]

The microphotograph of a Norwegian clay presented in the report by Torrance is not typical of sensitive clays of the Champlain Sea. Figure 2 presents two microphotographs showing the large range of marine clays microstructure from Eastern Canada. The specimen from Louiseville has characteristics closer to that of the Leda clay at the CRRRC site with a plasticity index at 38%. The second example is from Port-Cartier (Goldthwait Sea sediment) and is similar to what is present in Norway with a very low plasticity index of 5% and a liquidity index of 3.0 corresponding to a remoulded shear strength of 0.07 kPa, which is at the lower limit of what can be measured we the Swedish fall cone.

In the region, which includes the Boundary Road site, following land emergence after the retreat of the Champlain Sea, studies indicate that the upper sections of the marine sediments were leached to a point that reduced the liquid limit of the sediment, which resulted in an increase in the liquidity index (LI) to values above 1.0, i.e., with a water content above the liquid limit. By that process and while the intact strength was kept almost constant, the result was an increase in their sensitivity and compressibility once the pre-consolidation pressure is exceeded. By inspection of the general geotechnical profile (Fig. 1) it can be seen that the leaching has been effective mostly down to a depth of about 16 m with a maximum liquidity index of about 1.9 at a depth of 7 m and a value around 1.0 at a depth greater than about 14 m. Quigley et al. (1989, Appendix III) provide some data on the salinity for few clay samples from one borehole drilled by Geocon at Boundary Road in 1987, which indicate that the salinity would be less than 2 g/L down to a depth between 5 and 8 m, and then regularly increases to a value of 12 g/L at a depth of about 18m. As a reference, the salinity of sea water is about 30 g/L but much less in estuarine environments (as low as 5 to 10 g/L as in the St. Lawrence Estuary).

Because of the presence of sensitive marine clays in Norway, the Norwegians, through the Norwegian Geotechnical Institute (NGI), were the early world leaders in researching the characteristics and engineering behaviour of sensitive marine clays. This has been of much assistance in understanding the sensitive marine clays in eastern Canada.

From the distribution of the liquidity index values, Prof. Torrance expounds in his report on the presence of quick clays at the Boundary Road site and the risks associated with them. In response, I would first like to note that the geotechnical soils conditions at the Boundary Road site are quite common in Eastern Canada. I also note that the definition of "quick clay" varies from jurisdiction to jurisdiction. In Norway (Torrance 2014, Kalsnes et al. 2014), they use remolded undrained shear strength less than 0.5 kPa (which is equivalent to a liquidity index of more than 1.65) and a sensitivity measured by the Swedish fall cone greater than 30. In Sweden (Persson 2014), a quick clay is defined by a sensitivity measured by the fall cone greater than 50 and a remolded undrained shear strength of less than 0.4 kPa (equivalent to a liquidity index of more than 1.75). In Canada, the Canadian Engineering Foundation manual does not provide a value for the undrained remolded shear strength for quick clays but only defines it as a soil with a sensitivity greater than 16 (not clear on how to measure it, but likely measured with the in situ vane as was used on the Boundary Road site by Golder). In Québec, as I have already noted, the term “quick clay” is not used as such, but Demers et al. (2014) would use the Norwegian definition if needed. In Québec, clay soil sensitivity is classified as the following: low (<10), average (10 to 40), high (40 to 70) and very high (greater than 70). In addition, in Quebec, extremely sensitive soils are those for which the undrained remolded shear strength cannot be measured by the fall cone (IL > 3.0). This is the case in places around the Canadian Shield, where sensitive clays can reach liquidity index values up to 7 or more, like at Saint-Jean-Vianney (Potvin et al. 2001).

Prof. Torrance uses a liquidity index of 1.2 when referring to the presence of sensitive clays and quick clays (p. 7). This value may come from what is use in Québec, but for another purpose. The liquidity index of more than 1.2 is used in Québec, in addition to other parameters such as the slope height, for classifying sensitive clayey soils which, if a landslide is initiated, can transform and retrogress into flow slides (Demers et al. 2014). I consider that the use, by Prof. Torrance, of a liquidity index of 1.2 to define quick clays, is inappropriate. Lowering the liquidity index threshold, as done by Prof. Torrance, results in an increase in the thickness of the quick clay layer as can be noted by inspection of Figure 1.

If the Norwegian criteria of 1.65 were considered, using the liquidity index values calculated from the data provided by Golder (provided on Figure H1 in Volume III Appendix H and modified as Figure 1), only 3 samples would meet this criteria and the thickness of clay classified as quick would be about 6 m. A thickness of less than two metres would meet this criteria if we use the Swedish criteria at a liquidity index of more than 1.75. If we use the Canadian Engineering Foundation Manual and look at the sensitivity (St) profile in Appendix E of the Golder Volume III report, only one point with a sensitivity of more than 16 would meet this criteria suggesting a quick layer thickness less than 2 m.

Opinion #3: In my opinion, the geotechnical characteristics of the soils at the Boundary Road site are quite common throughout the region. Many facilities and structures have been successfully established on such soils.

Opinion #4: I am of the opinion that comment #9 made by Prof. Torrance does not provide any precision on the actual thickness of the "quick clay" layer at the Boundary Road site, nor does it provide the definition he would use himself to locate it along the profile. I am also of the opinion that a liquidity index value of 1.2 is not appropriate here. If known and accepted definitions for quick clay from elsewhere are used, then the thickness of the quick clay layer reduces significantly from about 11 m (using 1.2) to about 6 m (for a LI of >1.65) and to less than 2 m when using St>16 or a LI of >1.75.

5. Use of sensitive soft silty clay geotechnical characteristics in the design of the landfill site

The analysis below will show that regardless of the definition for 'quick clay' behavior, the most important consideration is to ensure that the actual properties of the clays are properly taken into account in the design of the facility components.

The relevant geotechnical engineering considerations can be classified into two broad categories: (1) settlement and (2) slope stability. For settlement, the main parameters used are derived from consolidation (or oedometer) tests. With respect to slope stability (e.g., embankment stability) the strength parameters are used in drained or undrained conditions depending on the situation and are defined by either field or laboratory investigations or both (see also Leroueil et al. 1990).

5.1 Settlement of the soft Leda clays

The composite geotechnical profile in Figure 1 above shows that the water content in the silty clay formation decreases more or less regularly with depth at the Boundary Road site, likely largely as a result of consolidation since deposition. However, this could also have occurred due to changes in the grain size of the sediment. Although the Golder samples were taken with a Shelby tube, the quality is good so that the pre-consolidation pressure could be determined with sufficient accuracy. The oedometer test results plotted in Figure 1 show that the soils at the site can be considered as normally consolidated (NC), i.e., the pre-consolidation pressure is close to or slightly above the in situ effective stress (note here that other methods of sampling that further reduce the potential disturbance of the sample could yield greater values of pre-consolidation pressures). The value of the pre-consolidation pressure (see Figure 3 below) also indicates the stress (or applied load) above which settlement can become significant since the compression index (Cc) becomes much higher.

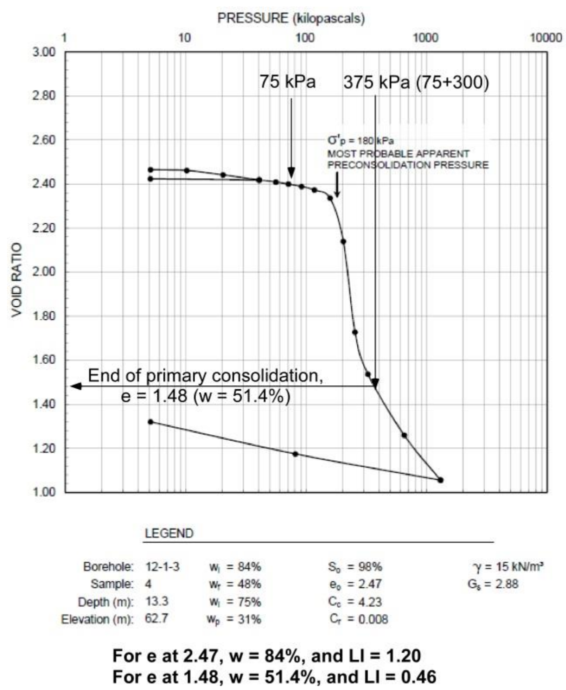

Figure 3. Estimation of changes in void ratio and water content at the end of primary consolidation for a sample located at a depth of 13.3 m.

In the portion of the silty clay where the liquidity index is above 1.0, the compressibility, measured with the oedometer tests, is also high (up to 4.2, Figure 1). This is a characteristic of soft sensitive clays (including quick clays) and this compressibility is often also positively related to the liquidity index in the case of leached marine clays (Leroueil et al. 1983).

If large settlements (up to 8 m) near the center of the landfill structure are predicted, this is because the soft clay (including quick clay) soils conditions were taken into account in the design. Interestingly and importantly, although not commented on by Prof. Torrance, with time, as consolidation of the foundation silty clay resulting from the increasing weight of the waste pile proceeds, water will be squeezed from the soils underlying the landfill component of the proposed facility. The result will be that over time, the liquidity index at the Boundary Road site will be reduced to values that will make the underlying soils much less sensitive and stronger than comparable soils in the general area.

A simple computation of the expected changes in the liquidity index of the soft sensitive clay due to the reduction of the water content (the liquid and plastic limits remaining constant) resulting from the consolidation under the landfill weight is shown in Figure 3. For that example, the oedometer test presenting a high compressibility of 4.2 is used. That sample has been taken at a depth of 13.3 m and in the natural pre-development condition is under an estimated effective stress of 75 kPa. At the end of construction of the landfill, the maximum increase in effective stress is estimated at about 300 kPa (Volume III, p. 73), which means that the stresses at 13.3 m would become about 375 kPa. If we use the oedometer test result obtained for that sample, i.e., a relationship between pressure and "e" the void ratio (or water content), we can see that the void ratio at the end of the primary consolidation could be about 1.48, which is equivalent to a water content of 51.4%. Replacing the value of the initial water content of 84% (corresponding to an initial void ratio of 2.47) by the value of 51.4% at the end of primary consolidation, the new value of the liquidity index decreases significantly from 1.20 to 0.46, i.e., well below the liquid limit of 75%. If a similar calculation is performed for a clay sample from 6.4 m (Figure F19, Appendix F of Volume III) that is within the upper zone where the clay is softer (and more compressible) and the LI is higher, the water content would reduce from 89% to 47% and the LI from 1.9 to 0.62, again well below the liquid limit.

It can be noted here that the water content of the soft sensitive clays will likely reach a value close to the liquid limit in a time much less than that needed to reach the primary consolidation (estimated to be about 100 years).

Opinion #5: The parameters used in Golder's analysis were those related to the presence of sensitive clays (including quick clays), i.e., high compressibility. Contrary to the assertions by Prof. Torrance (comment #8), the properties of the 'quick clays' at the Boundary Road site were not missed but rather very well integrated and taken into account in the design of the landfill to predict the settlement.

Opinion #6: The analysis of the compressibility as a function of loading indicates that as the construction of the landfill proceeds, the underlying Leda clay formation will undergo material consolidation (reduction in the water content). This will substantially reduce the liquidity index while strengthening the foundation soil. It can be expected that, after the completion of the primary consolidation, the foundation soils under the landfill will have increased in strength, become less sensitive, and any quick clay layer characteristics would likely disappear, this process being greater near the center of the landfill component. This a crucial and positive consideration for the long term behaviour of the landfill, which is not at all considered by Prof. Torrance.

5.2 Slope stability of the landfill

The strength properties are also well defined not only by the use of the in situ vane (for also estimating the sensitivity), but also of the many Cone Penetration Tests (CPT, Appendix B of Volume III) that provide an excellent description of the areal distribution of the soft clays at the site and also show how the strength varies with depth and over which thickness. For their initial slope design for the landfill configuration proposed at this site by the Regional Municipality of Ottawa-Carleton, Geocon. in 1987. proposed slope angles for a landfill slope height of 10 m with slopes at 7 horizontal (7H) to one vertical (1V, or 8.1°). In their report in support of the proposed CRRRC, Golder proposes 14H to 1V (or 4°) for the first portion up to a height of about 13 m and then a slope gradient of 20H to 1V (or 2.9°) for the remaining construction of the top portion of the landfill so as to maintain a sufficient factor of safety against failure in the underlying silty clay foundation. It is important to note here that these proposed slopes are very flat. The significant flatness of the slopes proposed by Golder directly results from taking into consideration the actual in situ strength of the soft silty clay for end of construction conditions. It must be underlined here that the stability calculations of Golder were undertaken using the full height of the landfill as if it was built at once using the strength parameters of the soft clay before construction, i.e., not taking any strength increase of the foundation resulting from the settlement process discussed above. This is quite conservative.

In addition, it is proposed by Golder to install a series of piezometers to monitor changes in the pore pressures generated by the landfill construction in the soft clay foundation so as to adjust the rate of construction whenever necessary. This is a standard and accepted procedure to ensure the stability of the landfill infrastructure, which has been developed based on many years of construction experience by the geotechnical community on soft flays (including quick clays) in Eastern Canada.

Opinion #7: The stability analysis was carried out by Golder using actual in situ strength properties of the soft sensitive clays at the landfill site.

Opinion #8: With reference to Prof. Torrance's comment #9, I conclude that the analysis carried out by Golder for designing the landfill component of the CRRRC (for both settlement and stability) did use the full properties of the soft clays present at the Boundary Road site, so that nothing has been missed even if the term 'quick clay' was not used by Golder in their report. To me, the important element here is not the name of the sensitive silty clay or how to classify it but rather how to make sure that the geotechnical properties of the soils are taken into account for any design considerations. Golder's approach to the design was based on full consideration of the softness of the silty clays. In my opinion, nothing relevant has been missed by Golder and the design has been undertaken using a conservative approach to ensure the security of the facility.

6. Liquefaction

As indicated in the Canadian Engineering Foundation Manual (Canadian Geotechnical Society, 2013), liquefaction is defined *in terms of sudden reduction in the rigidity and shearing resistance of a soil under the effect of cyclic loading caused by an earthquake* (P. 92 in the French version). This is different from the transformation of a sensitive clay into a flowing material due to mechanical transformation, i.e., as a result of shearing of the clay as occurs initially during a landslide or as a result of bearing capacity failure. We have many landslides in sensitive leached marine clays and in marine sediments that were the result of earthquakes. In all cases, an initial slide is needed along a slope to initiate the retrogressive process during which back stepping of the various slices takes place and are remolded as they fall below the slope and then turn into a fluid like material. The report by Prof. Torrance use the term liquefaction for various situations without providing a clear definition.

Such a definition problem has been well described by Seed et al. (2003, p. 4): *Some of the confusion here is related to the definition of liquefaction. In this paper, the term "classic" cyclic liquefaction will refer to significant loss of strength and stiffness due to cyclic pore pressure generation, in contrast to "sensitivity" or loss of strength due to monotonic shearing and/or remolding as a result of larger, monotonic (unidirectional) shear displacements. By making these distinctions, we are able to separately discuss "classic" cyclically-induced liquefaction and the closely-related (but different) phenomenon of strain-softening or sensitivity*. According to Seed et al. (2003), soils within zone A (Fig. 4) are considered potentially susceptible to 'classic' cyclically induced liquefaction. Soils in zone B may be liquefiable. Soils outside zone A and B are not generally susceptible to 'classic' liquefaction but should be checked for potential sensitivity (loss of strength with remoulding or monotonic accumulation of shear deformation).

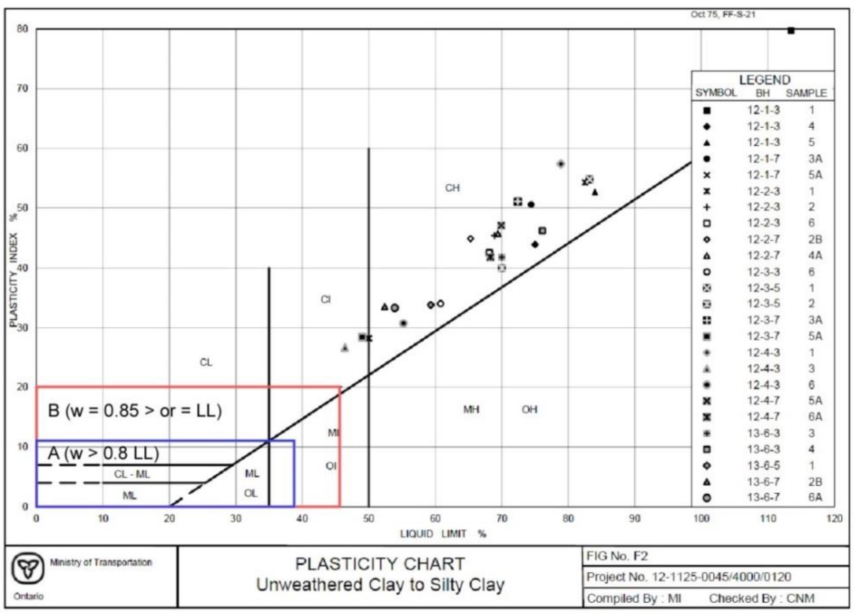

Figure 4. Type of soils which could be liquefiable showing that the soil at the landfill site are not susceptible to 'classic' liquefaction according to Seed et al. (2003). Zone A: below the blue line; Zone B: above the blue line and below the red line. In all cases, the water content has to be close to or above the liquid limit. The Leda clays tested at the Boundary Road site all fall outside regions A and B.

In his report, Prof. Torrance (page 3) mentions marine clays that 'may liquefy and flow downslope because they can no longer support their own weight'; this is often related to high sedimentation rates as is the case in the Mississippi delta. But these types of slides are not caused by a 'classic' liquefaction process. For the case of the Chelsea slide, which took place in May 1973 (and not in 1976 as mentioned in Prof. Torrance's report), the fluidity of the remolded clay likely resulted from the transformation of the sensitive clay as a result of the initial bearing capacity failure.

In his analysis (page 4), Prof. Torrance, as testimony to support his hypothesis on the liquefaction of quick clays, refers to landslides that have been triggered by earthquakes in the Ottawa area. It is well known that earthquakes do trigger landslides; however, the transformation of the sliding mass into a fluid like behavior is a result of a post-failure mechanism, typical for highly retrogressive slides in sensitive clays (e.g. the St-Jean-Vianney slide in 1971). In all cases, these slides needed a nearby open slope for the first slide to take place.

Opinion #9: The Leda clays at the landfill site are not liquefiable in the 'classic' definition. Further, the local topography and environmental setting at the site (rather flat) do not provide the conditions for flow slides to develop.

7. Hazards associated with earthquakes

I would like here to concentrate on two elements of the discussion about potential hazards associated with earthquakes: (1) potential for deep foundation failure and the resulting surface deformation (as seen in the Lefaivre area by Ayslworth et al. (2003)) and (2) consideration of seismic risk**.**

7.1 Deep foundation failure

Aylsworth et al. (2003) reported large surface deformations related to deep failure within the Leda clays in the Lefaivre area about 50 km from the landfill site. Golder, in its analysis of the case, provides clear arguments, not commented on by Prof. Torrance, that the conditions at the CRRRC site do not compare with the area studied by Aylsworth et al (2003) i.e., (1) the absence of thicker clay deposits confined within a deep bedrock bowl (thereby reducing the site amplification effect) and (2) the absence of thick sand layers that could liquefy and generate surface displacement.

As indicated by Prof. Torrance (page 4) the fact that quick clays still exist today is a testimony that shaking does not always cause them to collapse. I would add to this that clayey soils in a region that has over time experienced many strong earthquakes may have become less prone to develop instability by a mechanism called seismic strengthening (which could be seen as a sort of natural dynamic compaction).

Golder has used a marker horizon about 5 to 6 m deep at the Boundary Road site to show that the surface has not been modified since the deposition of Leda clays (see Fig. 3-14 and 3-15 in Volume III). In addition, and as mentioned (p. 58 of Volume III), inspection of the photos of many continuous push soil cores taken of the banded Leda clays (Appendix I) at the Boundary Road site reveals no visible sediment deformation.

In addition, as I have noted above, the foundation strength under the landfill component of the CRRRC will significantly increase with time, so that the dynamic response of the soil at the landfill site to a strong earthquake, should one occur, will continue to improve.

Opinion #10: I fully agree with the analysis provided by Golder in relation to deep foundation failure and conclude that this is not an issue of concern in relation to the proposed CRRRC.

7.2 Seismic analysis

Volume III of the EASR report contains a section titled 'Seismic Assessment' (section 11.2). Details of the analysis are provided in Appendix Q (a Technical Memorandum on Seismic Stability Dynamic Analysis).

Prof. Torrance suggests in his report that the soft clay may present a risk during an earthquake (comment #8). On page 4 he also underlines the fact that earthquake triggered landslides should serve as a warning not to ignore the risk.

I agree that seismic risk must be taken into account and I consider that this is exactly what has been done by Golder. At the end of his report, Prof. Torrance insists on carrying out such analysis without providing any comments or arguments that would indicate that the approach used by Golder is incorrect.

As indicated in Volume III Section 11.2, Golder approached the analysis using the building code requirements and considering a mean earthquake magnitude ranging between M6 and M7 associated with distances between 25 km and 72 km. According to the building code, the probability of such an earthquake is one in 2,475 years. This indicates also that the probability that such an earthquake would take place before the end of construction is even less. Still, Golder's approach is conservative as I have already noted above, since the actual strength of the soft clay pre-development was used in the analysis without taking into account the increase with time in the shearing resistance of the clay foundation soil due to consolidation.

Opinion #11: I am of the opinion that the seismic analysis carried out by Golder used a conservative approach and corresponds to state of the art practice, and further that the response of the site to a strong earthquake, should one occur, will continually improve as the consolidation of the clay foundation soil takes place.

8. Leachate effects on the strength of the clay

Prof. Torrance (page 7) recognized that in the leaching experiments carried out by Quigley et al. (1989) some gain in strength took place. Quigley et al. (1989, p. 31) also noted that the resulting decrease in the void ratio introduced a decrease in the hydraulic conductivity 'k' that may compensate for any increase in permeability due to physio-chemical factors. The time required for any leachate contaminant migration in the Leda clay formation will be quite long, so that consolidation settlement of the foundation will largely have taken place and would in my view effectively counterbalance any adverse effect of leachate on the hydraulic conductivity of the clay over time. From the work of Lapierre et al. (1990) on the Louiseville clay, we can expect a one magnitude reduction in the hydraulic conductivity for the expected changes in the void ratio of the foundation at the end of primary consolidation.

Opinion #12: I am of the opinion that taking into consideration the expected large settlement at the end of the primary consolidation period would effectively compensates for any physio-chemical impact of leachate on the permeability of the Leda clay at the site over time.

8. Conclusions

Following are my principal conclusions on my analysis of Prof. Torrance's report:

1. A major point missing in the report by Prof. Torrance is any consideration of the evolution of the geotechnical characteristics of the foundation soil (including quick clays) with time until the end of primary consolidation.
2. The major points raised by Prof. Torrance regarding the presence of quick clays and their response to seismic shaking were very well considered by Golder in the design of the landfill infrastructure.
3. I consider that Golder has taken into account the full characteristics of the soft Leda clay (including quick clays) in all aspects of the design of the landfill site and used a very conservative design approach by not considering the gain in strength of the foundation as a result of the construction of the landfill site;
4. I consider that the proposed monitoring program will ensure the safe progression of the landfill operations;
5. The expected settlement will ensure a significant reduction in the water content of the compressible (i.e., soft clays) part of the foundation to a point where there would be no more soft sensitive clay underneath most of the landfill site sometime before the end of primary consolidation, i.e. once the water content has been reduced to the liquid limit or less than 100 years after the beginning of the construction.
6. The expected settlement and the corresponding reduction in the hydraulic conductivity of the soft clays and the fact that the leachate will accumulate towards the center of the landfill structure will ensure that leachate will be contained and collected within the landfill component of the proposed CRRRC.

As a concluding statement, in my opinion, the main points raised by Prof. Torrance were well understood and appropriately considered by Golder. It is also my opinion that none of the main points in Prof. Torrance's report as discussed above raise any geotechnical or related concerns not adequately addressed in the evaluation and design of the proposed facility.

References

Aylsworth, J.M. and Lawrence, D.E., 2003. Earthquake-induced landsliding east of Ottawa; a contribution to the Ottawa Valley landslide project. *In* Geohazards 2003, Edmonton, pp.: 57-64.

Canadian Geotechnical society, 2013. Manuel canadien d'ingénierie des foundations. 4th Edition, ByTech Publishers Ltd, 476p.

Demers, D., Robitaille, D., Locat, P., and Potvin, J., 2014. Inventory of large landslides in sensitive caly in the Province of Québec, Canada: Preliminary analysis. *In:* J.-S. L'Heureux et al. (eds), Landslides in Sensitive Clays: from Geosciences to Risk Management. Advances in Natural and Technological Hazards Research 36, pp.: 77-89.

Gadd, N.R., 1985. Lithofacies of Leda Clay in the Ottawa Basin of the Champlain Sea. Geological survey of Canada Paper 85-21, 46 p.

Gadd, N.R. (editor). 1988. The late Quaternary development of the Champlain Sea basin. Geological Association of Canada, Special Paper 35.

Kalsnes, B., Gjelsvik, V., Jostad, H.P., Lacasse, S., and Nadim, F., 2014. Risk assessment for quick clay slides- the Norwegian practice. *In:* J.-S. L'Heureux et al. (eds), Landslides in Sensitive Clays: from Geosciences to Risk Management. Advances in Natural and Technological Hazards Research 36, pp.: 355-367.

Leroueil, S., Magnan, J.-P., and Tavenas, F., 1990. Embankments of soft clays. Ellis Horwood Publisher, 360 p.

Lapierre, C., Leroueil, S., and Locat, J., 1990. Mercury intrusion and permeability of Louiseville clay. Canadian Geotechnical Journal, 27: 761-773.

Leroueil, S., Tavenas, F., and Le Bihan, J.-P., 1983. Propriétés caractéristiques des argiles de l'est du Canada. Canadian Geotechnical Journal, 20: 681-705.

Persson, M.A., 2014. Three-dimensional quick-clay modeling of the Gothenburg Region, Sweden. *In*: J.-S. L'Heureux et al. (eds), Landslides in Sensitive Clays: from Geosciences to Risk Management. Advances in Natural and Technological Hazards Research 36, pp.: 39-50.

Potvin, J., Pellerin, F., Demers, D., Robitaille, D., La rochelle, P., and Chagnon, J.-Y., 2001. Revue et investigation complémentaire du site du glissement de Saint-jean Vianney. *In* Proceedings of the Canadian Geotechnical Conference, Calgary, pp.: 792-800.

Quigley, R.M., Fernandez, F., and Ohikere C., 1989. Clay leachate compatibility study hydraulic conductivity of Ottawa/Carleton 'Leda' clay barrier soils permeated with domestic waste leachate. Report prepared for Environment Ontario. 130p.

Torrance, J.K., 2014. Chemistry, sensitivity and quick-clay landslide amelioration. *In*: J.S. L'Heureux et al. (eds), Landslides in Sensitive Clays: from Geosciences to Risk Management. Advances in Natural and Technological Hazards Research 36, pp.: 15-24.