# 11.0 GEOTECHNICAL CONSIDERATIONS OF SITE DESIGN

This section discusses some of the geotechnical design aspects of CRRRC project, with a focus on the landfill geometry and performance. The geotechnical design aspects of secondary Site components (e.g., pavement designs for roadways, detailed design of building foundations, screening berm construction, etc.) will be addressed subsequently as part of the City site plan and building permit application process.

In general, the subsurface conditions across the Site consist of about 0.05 to 0.3 metres of topsoil/peat underlain by about 0.3 to 2.7 metres of surficial sand and silt, overlying between about 26 to 37 metres of sensitive silty clay. The upper 0.1 to 1.3 metres of the clay deposit at most locations has been weathered to a red brown crust and has a stiff consistency. The underlying silty clay generally has a soft consistency to about 9 to 10 metres depth, followed by a firm consistency to about 15 to 18 metres depth, and is stiff to very stiff below that. The silty clay is underlain by loose to very dense glacial till that ranges from about 2 to 9 metres in thickness. The bedrock surface was encountered beneath the glacial till deposit at depths between about 33 and 41 metres.

The following sections provide a summary of the results of the slope stability and settlement analyses carried out for the Site, along with recommendations for Site design.

## 11.1 Stability Analyses

The presence of the thick deposit of soft silty clay beneath the Site presents a constraint on the landfill geometry.

Various potential waste slope geometries were initially evaluated, in order to optimize the Site design. The currently proposed arrangement (as described in Section 10.0 and below) was ultimately selected as being preferred. Only the proposed Site development landfill arrangement is discussed.

The use of 3.5 metre high perimeter berms, with a crest width of 36 metres, was identified by the analyses as being a key component of the design, from the perspectives of optimising the landfill capacity and achieving the required factor of safety.

Stability analyses have been carried out for the various slope geometries that will exist around the perimeter of the landfill, including the arrangements of the perimeter berms and the adjacent features. The analyses identified that the critical locations/slopes are those on the eastern and northern sides, where shallow excavations will be needed parallel to the slope toe, to accommodate surface water drainage elements. The resulting proposed landfill sideslope geometry along these slopes is described as follows (downward, from peak to toe):

- **The eastern side of the landfill adjacent to the linear storm water management pond:**
  - A maximum peak height of the landfill of about 25 metres above the existing ground surface.
  - A slope down from the peak at 20H:1V (horizontal to vertical inclination) to a height of 13.5 metres above the existing ground surface.
  - A further slope down at 14H:1V to the top of the perimeter berm.
  - A perimeter berm which is 3.5 metres high (relative to the existing/native ground surface) and with a crest width of 36 metres (from the edge of the waste to the crest of the external berm sideslope).
  - An 'outer' berm sideslope inclined at 7H:1V, extending down to the native/existing ground surface.

- About a 23 metre set-back distance from the toe of the perimeter berm to the crest of the linear storm water management pond.
- A 3H:1V and 2-metre high slope down to the floor of the storm water management pond (i.e., reaching to a maximum depth of 2 metres below the existing/native ground surface).

- **The northern end of the landfill adjacent to the Simpson Drain:**
  - A maximum peak height of the landfill of about 25 metres above the existing ground surface.
  - A slope down from the peak at 20H:1V to a height of 13.5 metres above the existing ground surface.
  - A further slope down at 14H:1V to the top of the perimeter containment berm.
  - A perimeter berm which is 3.5 metres high (relative to the existing/native ground surface) and with a crest width of 36 metres (from the edge of the waste to the crest of the external berm side slope).
  - An 'outer' berm side-slope at 7H:1V, extending down to the native/existing ground surface.
  - About a 20 metre set-back distance (minimum) from the toe of the perimeter berm to the crest of the Simpson Drain (which is up to two metres deep relative to the existing/native ground surface).

Two other temporary conditions were also identified as being critical to the design and the following geometries were proposed and analyzed:

- **Internal Perimeter Berm/Excavation Stability:** An *internal* perimeter berm/excavation slope that is inclined at no steeper than 7H:1V from the top of the perimeter berm to the subgrade level of the landfill (based on the 'internal' stability for the creation of that excavation beside the berm).
- **Interim Waste Slope:** A typical interim waste slope geometry between adjacent phases which consists of a 14H:1V inclination from a height of about 13.5 metres above the original ground surface down to the subgrade level of the landfill (based on the stability of the proposed temporary slopes during waste placement, in accordance with the proposed phasing).

The stability analyses were carried out using the SLOPE/W commercial software, which uses Limit Equilibrium methods to calculate a factor of safety against shearing of the soil and resulting instability. The Morgenstern-Price method was used to compute the factor of safety. The factor of safety is defined as the ratio of the magnitude of the forces/moments tending to resist failure to the magnitude of the forces/moments tending to cause failure. Theoretically, a slope with a factor of safety of less than 1.0 will fail and one with a factor of safety of 1.0 or greater will stand. However, because the modelling is not exact and natural variations exist for all of the parameters affecting slope stability, a higher factor of safety is typically required. The following minimum target factors of safety were identified for these analyses:

- Overall Landfill/Waste Slope: 1.4;
- Internal Perimeter Berm/Excavation: 1.3; and,
- Interim Waste Slope: 1.4.

The analyses were carried out for undrained conditions (i.e., short-term conditions, where the full excess porewater pressures are generated in the silty clay due to the applied stress from the full height of waste). This condition is considered to be a conservative assessment, because the actual waste placement will take place over several decades, allowing for some pressure dissipation. Therefore, conditions in the underlying silty clay would actually be intermediate between truly undrained or completely drained (i.e., where the waste is placed sufficiently slowly that excess porewater pressures are not generated in the clay). However, given the uncertainties regarding the actual rate of filling, and to allow flexibility on the rate and location of waste placement, undrained conditions were conservatively selected as the design criteria. This undrained condition would be analogous to the landfill being completely filled and the cover soil placed semi-instantaneously (or over a very short period of time).

The soil parameters used for the analyses were interpreted from the subsurface information collected from the extensive geotechnical investigation carried out for the Site as described in Section 2.0 (methodology) and Section 6.0 (results). Because undrained conditions were analyzed, total stress parameters were used for the silty clay. The selected parameters are summarized in Table 11-1:

**Table 11-1: Summary of Soil Parameters**

| Material | Unit Weight (KN/m$^3$) | Undrained Shear Strength (kPa) | Friction Angle (degrees) |
|---|---|---|---|
| Final Cover | 19 | 0 | 25 |
| Waste | 12 | 0 | 30 |
| Saturated Waste | 16 | 0 | 30 |
| Drainage Layer | 18 | 0 | 30 |
| Perimeter Berm Fill | 18 | 0 | 28 |
| Surficial Soils<br>(0 to 1.5 metres below original ground surface) | 19 | 0 | 28 |
| Upper Clay A<br>(1.5 to 3.5 metres below original ground surface) | 15 | 10 | 0 |
| Upper Clay B1<br>(3.5 to 6.0 metres below original ground surface) | 15 | 12 | 0 |
| Upper Clay B2<br>(6.0 to 7.0 metres below original ground surface) | 15 | 11 | 0 |
| Upper Clay C<br>(7.0 to 15.0 metres below original ground surface) | 15 | Increasing from 11 to 29 | 0 |
| Upper Clay D<br>(15.0 to 20.0 metres below original ground surface) | 15 | Increasing from 29 to 52 | 0 |
| Lower Clay A<br>(20.0 to 25.0 metres below original ground surface) | 16 | 52 | 0 |
| Lower Clay B<br>(25.0 to 35.8 metres below original ground surface) | 16 | Increasing from 58 to 116 | 0 |
| Glacial Till | Impenetrable | | |
| Bedrock | Impenetrable | | |

**Notes:** kN/m$^3$ – kilonewtons per cubic metre; kPa – kilopascals

The values of the 'mobilized' undrained shear strength of the silty clay indicated in the above table were selected based on the *in-situ* vane testing results, the plasticity index values indicated from the laboratory testing program, the assessed preconsolidation pressures, and the CPT results.

Several different shearing geometries (i.e., potential failure surfaces) were assessed, including rotational, sliding, and composite failures. The critical failure surface was generally found to consist of a sliding/translational failure through the upper unweathered clay layer between 6.0 and 7.0 metres depth (i.e., just above the zone of significant strength increase with depth).

### 11.1.1 Static Slope Stability Results

A summary of the static slope stability results is presented in Table 11-2:

**Table 11-2: Summary of Static Slope Stability Results**

| Critical Slope Cross Section | Calculated Static Factor of Safety |
|---|---|
| The eastern side of the landfill adjacent to the storm water management pond | 1.4 |
| The northern end of the landfill adjacent to the Simpson Drain | 1.4 |
| Interim waste slope | 1.5 |
| Internal Perimeter Berm/Excavation | 1.3 |

The analysis results for the first 3 cases in the above table are shown graphically on Figures 11-1 to 11-3, respectively. Each pair of figures presents the information using a normal scale (Figure A) to show the landfill in context and with the vertical scale exaggerated three times (Figure B) to allow the descriptions for the layers to be legible.

Based on the above results, it is considered that the proposed waste slope geometries and berm/excavation geometries have an acceptable static factor of safety against slope instability (i.e., the proposed design meets the design criteria).

It should be noted that the landfill geometry used in the analyses, and described above, is the theoretical geometry without accounting for subgrade settlements. As discussed subsequently in Section 11.3 of this report, the subgrade settlements due to consolidation of the underlying silty clay will be time-dependant (taking many years/decades to occur). It is expected that the subgrade surface will be settling while waste is placed. Therefore, it would not likely be technically feasible to actually fill to the theoretical slope/cover elevations considered in these analyses. It will therefore be necessary to monitor the subgrade settlements (see Section 11.5 for the proposed geotechnical monitoring program).

The stability analyses are also dependent on the unit weight of the waste and, in view of the low shear strength of the underlying clay, it will be important to also carry out monitoring to evaluate the unit weight of the as-placed waste to assess the overall waste *weight* (i.e., stress imposed on the subgrade) compared to the weight considered in the stability analyses.

It may also be feasible to re-evaluate on an ongoing basis the actual permissible finished slope/cover geometry (not to exceed the final design elevation contours) based on the strength gain that will occur as the underlying clay consolidates and compresses. To do so, it will be necessary to monitor the landfill subgrade settlements (as a measure of the degree of consolidation), the rate of excess porewater pressure dissipation in the silty clay deposit, and the rate and magnitude of the lateral deformation of the silty clay beneath the perimeter berms.

To evaluate landfill capacity at this stage of the project, the geometry/volume defined by the stability analysis was used (i.e., the theoretical volume corresponding to the final waste elevations described above, in the absence of subgrade settlements).

The construction of the perimeter berms will require control on the material type used for the berm fill (specifically its unit weight) and on the level of compaction achieved, because the berm improves the stability of the landfill slope due to its overall *weight*. The stability analyses were based on a unit weight for the berm fill of 18 $kN/m^3$. A lower in-place unit weight for the fill would reduce the factor of safety against instability of the overall waste slope. Conversely, a significantly higher unit weight could reduce the factor of safety against localized instability of the berm itself, in particular along the east and north sides of the landfill where adjacent shallow excavations will be required for a storm water management pond and the Simpson Drain. As a preliminary guideline, the berm fill should be restricted to an in-place unit weight between about 17.5 and 18.5 $kN/m^3$.

### 11.1.2 Static Stability Guidelines for Related Site Features

In addition to the analysis results described above in relation to the landfill, static slope stability analyses were also carried out for various other features on the Site, such as fire, leachate, and storm water management ponds, and the Primary Reactor cells to be used in the organic processing compost facility. These features are considered to have adequate factors of safety provided the following guidelines are adhered to:

- Side slopes for fire, leachate, and those storm water management ponds not adjacent to the landfill should be sloped at 4H:1V. However, this guideline assumes that any grade raise fill or berm fill placed adjacent to the ponds will not be initially constructed within 15 metres of the crest of the slopes (i.e., a delay of approximately six months will be required between the pond excavation being made and the fill being placed).
- Any ponds placed adjacent to (i.e., north of) the Simpson Drain should be offset at least 10 metres from the crest of the exterior slope of the Drain (crest-to-crest distance).
- The external side slopes of the Primary Reactor cells should be sloped at 5.25H:1V for a maximum compost thickness of 6.5 - 7.0 metres and width of 70 metres.
- Any ponds placed adjacent to the Primary Reactor cells should be offset by at least 20 metres from the toe of the Reactor.

## 11.2 Seismic Assessment

Dynamic analyses were also carried out to investigate the seismic stability of the proposed landfill configuration when subjected to strong earthquake shaking. A summary of the analyses and results is provided in this section of the report. A memorandum with further details on the methodology used to assess the seismic stability and earthquake-induced deformations of the waste materials and the underlying foundations, and the results of the analyses, is provided in Appendix Q.

Seismic design guidelines established for solid waste landfills in the USA require that such facilities be designed to resist ground motions with a 2,475-year return period, which has been considered for the analysis of this landfill.

The corresponding seismic ground motion parameters for the Site have been evaluated using the seismic hazard models and seismogenic zones developed on a regional basis by Natural Resources Canada for use in the National Building Code of Canada.

The de-aggregated hazard for the Site indicates that the earthquake characteristics correspond to “mean” earthquake magnitudes ranging between M6 and M7 with associated distances between 25 kilometres and 72 kilometres.

Bedrock acceleration time-histories that correspond to those earthquake magnitudes were then selected from available synthetic earthquake records for Eastern Canada.

A total of six M7 earthquake records were selected and they were linearly scaled to match the response spectrum for the Site over the period range corresponding to the expected fundamental period of the soils underlying the Site. The duration of strong shaking of the selected time-histories varies between 10 and 15 seconds.

Non-linear dynamic time-history analyses were then carried out to assess the seismic stability and deformations of the CRRRC landfill at the closure condition. The seismic ground motions were propagated from the bedrock upwards towards the ground surface using ground response analysis models.

The analyses considered conditions at the end of filling. Over time, the self-weight loads imposed by the landfill materials will induce consolidation settlements in the underlying clayey soils, which will increase the strength and stiffness of the clay foundation soils. However, at the end of filling, the analyses indicate that, beneath the ‘youngest’ portions of the landfill (i.e., Phases 6, 7, and 8) there will only have been fairly limited consolidation and therefore no significant strength gain. The ‘end of filling’ time is therefore considered to be a conservative condition for which to check the seismic stability.

The analyses were carried out using the computer code FLAC$^{2D}$ *V6* (*Itasca,* 2008), which is a commercially available finite difference code with the capability to analyse the coupled stress-flow-deformation response of earth structures that can undergo large deformations under static and dynamic loading conditions.

The dynamic analyses were carried out considering two-dimensional plane strain conditions.

The analyses were conducted using the total-stress approach, with undrained shear strength parameters assigned to the clayey foundation soils. The shear strength profile for the clayey soils comprising the foundation under as-is conditions was established based on the SHANSEP concept. Laboratory cyclic simple shear tests carried out on undisturbed soil samples obtained from similar deposits in the Ottawa region indicate only nominal strain softening as a result of the application of up to 10 uniform cycles of shear loading (consistent with the anticipated shaking duration) that correspond to the projected intensity of Site-specific cyclic loading.

The computed seismic loading-induced lateral movements of the landfill for all six of the analyzed time histories are less than 340 millimetres. The calculated earthquake-induced deformations of the landfill are the result of deformations occurring in the upper clay layers directly below the landfill.

These results are indicative of a stable landfill under the design seismic loading conditions.

Further details on the analyses and results are provided in Appendix Q. In summary, the results indicate the following:

1) The landfill configuration is stable under the design seismic loading conditions;

2) The zones closest to the landfill toe undergo permanent lateral displacements of less than 340 millimetres during shaking (for 2,475-year return period ground motions). The resultant permanent ground movements at the corners of the landfill may be larger by about 40% due to three-dimensional loading effects, reaching values close to 500 millimetres;

3) The landfill lateral displacements are mainly controlled by the response of the soft clayey foundation soils directly below the waste materials and in the upper 20 metres; and,

4) Because the ongoing consolidation of the clay deposit beneath the waste will result in increased shear strength and corresponding increased resistance to the effects of earthquake shaking, the stability of the landfill will improve and the potential displacements will decrease with time after filling is complete.

## 11.3 Settlement Analyses

The development of the landfill (i.e., the placement of up to 25 metres of waste) will induce time-dependant consolidation of the underlying clay soil deposit. Due to the low hydraulic conductivity of the silty clay, the settlements will be time-dependant in nature and will occur over many years/decades.

The settlement estimates discussed in this section of the report represent the settlement of the landfill subgrade (i.e., at the base elevations of the waste), due to consolidation of the underlying silty clay deposit. There would be additional settlements of the landfill surface/cover, due to compression of the waste itself.

In order to estimate the magnitude of settlement of the silty clay underlying the landfill, analyses were carried out using the commercially-available ‘Settle-3D’ software.

The calculated ultimate effective stress levels in the silty clay will exceed the deposit’s preconsolidation pressure. The consolidation settlements will therefore occur in the ‘virgin’ compression range and will be significant in magnitude.

Porewater will need to be expelled for these settlements to occur. Therefore, due to the low hydraulic conductivity of the silty clay, the settlements will be time-dependant in nature and will occur over many years/decades.

Two key parameters in the evaluation of the magnitude and rate of consolidation settlement are:

- The preconsolidation pressure ($\sigma'_p$) of the silty clay, which is effectively its 'yield strength' and varies with depth (increasing in approximate correlation with the undrained shear strength); and,
- The coefficient of consolidation ($c_v$), which is related to the soil's hydraulic conductivity (and the ability to expel porewater), and which decreases as the clay consolidates.

The vertical profile of the preconsolidation pressure, through the soil deposit, has been selected based on the results of the laboratory oedometer consolidation testing. However, because the undrained shear strength is generally expected to correlate with the preconsolidation pressure (as evidenced by the data on Figure H1 in Appendix H), and there is significantly more data available on the undrained shear strength than there is for the preconsolidation pressure, consideration has also been given to the undrained shear strength profile in making the selection of the preconsolidation pressure profile with depth used in the settlement analyses.

The $c_v$ has been interpreted from the results of the laboratory oedometer consolidation testing (with emphasis on the $c_v$ data for those tests carried out with greater load increment ratios), as well as from the results of the porewater pressure dissipation tests carried out as part of the CPT program (see Appendix G). In addition, because there is considerable published evidence that the coefficient of consolidation as measured by these methods is often not consistent with actual/measured settlement performance, consideration has been given to published values of the coefficient of consolidation for Champlain Sea clay, as determined from the results of monitoring of the settlements of other embankments in eastern Ontario and western Quebec.

With the above approach, there is considerable variation in the values that could be selected for both parameters. A range of values/profiles for both parameters was therefore considered, and several combinations of the two used in the analyses. This methodology results in a range of the calculated possible settlements over time.

It was also considered in the analyses that the upper portion of the clay deposit, to a depth of about 20 metres, appears to have a higher compression index, slightly lower unit weight, and higher void ratio than the deeper clay. Different properties were therefore assigned in the model to the upper 20 metres of silty clay versus the deeper portion of the deposit.

It is noted that the properties of the silty clay deposit that affect its compressibility appear to be relatively uniform across this large Site (i.e., the silty clay properties are fairly homogenous in terms of horizontal variation). Therefore, only a single soil 'model' was developed to represent the conditions at this Site.

The initial effective stress profile used in the model, with depth, also considered that there appears to be a slightly downward hydraulic gradient through the silty clay deposit.

A one-way 'drainage' condition (upward) was selected for the analyses as being most representative of the anticipated behaviour during consolidation, for the groundwater flow associated with dissipation of the excess porewater pressures. This selection was based on the significant thickness of the deposit and considering that most of the settlements are calculated to occur within the upper portion. It should also be noted that the analysis software can only consider one-dimensional flow (i.e., up, or up and down). However, considering the significant horizontal dimension of the landfill, this drainage condition is considered to be a reasonable approximation of the real conditions, at least for the areas not directly along the perimeter of the footprint (where horizontal groundwater consolidation flow could occur), and is therefore reasonable for the most heavily loaded areas.

A waste unit weight of 12 $kN/m^3$ was used in the analyses, based on the type of waste to be placed in the landfill (including daily cover soil) and using published unit weight values. The lower one metre of material (waste and drainage materials) above the subgrade was considered to be saturated (and therefore heavier), based on a conservative assessment of the potential leachate level.

Based on the above, the 'net' applied stress on the subgrade under the highest portion of the landfill, based on the excavated soil to reach subgrade level, the waste height to be placed, and the cover material and drainage layers, is estimated at about 300 kPa.

The results of the analyses indicate that, under the highest portions of the landfill, the settlements resulting from *primary consolidation* of the deposit are expected to be in the order of 6 to 8 metres, by a time of about 100 years from the start of consolidation.

In the longer term, the settlements would increase beyond this estimate due to secondary compression of the deposit. The secondary compression index used to calculate these potential additional settlements was conservatively selected based on generally-accepted published correlations with the compression index. The results of the two long-term/sustained consolidation tests indicated secondary compression index values that were much less than would typically be expected, given the other properties of the silty clay deposit, and therefore the higher values based on published correlations were used. The resulting analysis results could therefore potentially over-estimate the secondary compression component of the overall settlements.

Based on the above methodology, the calculated range of settlements over time, based on the combination of primary consolidation and secondary compression, are shown on Figure 11-4.

The landfill subgrade settlements will also vary across the footprint, due to the variation in the landfill waste thickness. For example, the calculated range of settlements under a 13.5 metre waste height (i.e., beneath the transition level between the 14H:1V and 20H:1V side-slopes), over a 100 year time frame, are shown on Figure 11-5. These settlements are expected to range from about 3.5 to 5 metres in magnitude (combined primary consolidation and secondary compression).

The Settle-3D model was therefore developed to approximately correspond to the semi-rectangular landfill footprint and varying waste height. The resulting analyses indicate that the vertical stress increases generated in the underlying silty clay very closely correspond to the imposed stress directly above each location. There does not appear to be significant vertical dissipation of stress, or 3-dimensional effects, to the state of vertical stress. This result is considered to be attributed to the fact that the horizontal dimensions of the landfill are much larger than the thickness of the clay layer. As such, an essentially 1-dimensional assessment of the incremental vertical stresses beneath the landfill footprint is feasible for this project. Based on this assessment, the calculated range of settlements under waste heights varying up to the maximum proposed waste height, at a time of 100 years following that start of consolidation, are shown on Figure 11-6. These results can be used to evaluate the potential differential settlements of the subgrade (and drainage system) beneath different points in the landfill footprint.

In regards to these results, the following should be noted:

- The settlement calculations shown on Figure 11-6 may be of reduced accuracy in the area directly along the toe/perimeter of the landfill, where the one-dimensional assessment may be less representative; the settlements in those areas would potentially be slightly less than those indicated on Figure 11-6.
- The analyses are based on the simplification of the landfill being constructed essentially instantaneously and the settlements occurring thereafter. In actuality, the waste placement will occur over many years and therefore some of the settlements will occur during waste placement. The reference time for the settlement results provided on Figures 11-4 to 11-6 is therefore actually an intermediate time between the start and end of filling. Once the rate of filling has been defined (over time and by area of the landfill), these analyses could be refined.

As discussed in Section 11.1.1, the completed landfill geometry (i.e., the elevation of the 'finished' landfill surface and sideslopes) will need to account for subgrade settlements. Because the subgrade surface will be settling while waste is placed, it will not, therefore, likely be technically feasible to actually fill to the theoretical slope/cover geometry. Based on monitoring and the associated gain in strength of the clay as it consolidates, the appropriate final waste thickness (not to exceed the final elevation contours assumed for purposes of this study) will be determined in consultation with the MOECC prior to placement of the waste in the uppermost phases of the landfill. Subgrade settlements will be monitored (see Section 11.5).

## 11.4 Potential Geological and Geotechnical Related Effects on Landfill Design and Performance

The evaluation of potential geological impacts is provided in Section 9.0, while the geotechnical considerations are described in Sections 11.1, 11.2 and 11.3. The geological assessment concluded, based on available information, that there is no evidence of surface fault ruptures from historical earthquakes at the proposed CRRRC Site or its immediate vicinity. The assessment further concluded that there is negligible hazard at the CRRRC Site of future fault movement resulting in large scale differential displacements at the surface or shallow subsurface and that there is also little potential for differential settlement associated with strong earthquake shaking (liquefaction) at the CRRRC Site.

In any event, in terms of the engineering significance or potential effects of surface or subsurface displacements from potential future fault movement on the design and performance of the proposed CRRRC landfill, both the landfill mass itself and the proposed leachate containment and collection system (and its components), are very capable of withstanding significant differential displacements. There is no constructed or manufactured liner system at the base of the landfill as designed; rather, the containment of landfill leachate relies on the natural containment properties of the 30 metres of low permeability silty clay underlying the Site. The proposed leachate containment and collection system has been designed to withstand relatively large differential movements and continue to perform its intended function. For example, this containment and collection system has been designed to function when experiencing the predicted movements associated with long term consolidation of the clay deposit beneath the landfill, i.e., total settlements of 6 to 8 metres under the central portion of the landfill. The containment and collection system has also been designed to accommodate lateral displacements of up to 350 mm under seismic loading conditions. The effects of small-scale surface or subsurface displacements from fault displacement are, therefore, inconsequential for the engineering design and performance of the landfill component of the CRRRC.

## 11.5 Geotechnical Monitoring Program

It is recommended that a geotechnical monitoring program be implemented for the purposes of:

- Confirming that the performance/behaviour of the underlying foundation soils is consistent with those expected based on the geotechnical investigation program and analyses, to thereby confirm the applicability of the design recommendations provided; and,
- Providing information to optimize the design and/or operation of the landfill, as construction and filling progress.

The following monitoring measures are therefore recommended:

- The subgrade settlements should be monitored by means of surveying of the elevations of the leachate collection system manholes. If better definition of the settlement pattern is determined to be helpful (i.e., at a better horizontal resolution than can be achieved using only surveying of the manholes), then the feasibility of also monitoring the settlements by means of instrumentation placed on the landfill subgrade (such as with a grid of vibrating wire settlement monitors) could also be considered.
- The unit weight of the as-placed waste should be evaluated on a periodic basis (e.g., semi-annually) by means of weigh-scale records and air-space utilization surveys, and also using the subgrade settlement surveys.
- The lateral displacements of the silty clay beneath the perimeter berm of the landfill should be monitoring by means of the following:
  - Inclinometers should be used to measure the horizontal deformation profile in the silty clay with depth, using casings installed from the surface of the perimeter berm and anchored into the bedrock. Based on the anticipated performance, at least one inclinometer casing should be installed per side/face of the landfill. Note: Specialized telescoping casings will need to be used to avoid having the casing deformed by downdrag forces resulting from consolidation and settlement of the silty clay beneath the perimeter berms. The casing grout will also need to be designed to be compatible in physical behaviour with the surrounding soft soil.
  - Surface survey point/monuments should be installed along the surface of the perimeter berm and at the toe of the perimeter berm, which can be used to monitor the surface deformations (both horizontal and vertical). The monitoring of these can be carried out using conventional survey equipment/methods. Monitors should be installed every 200 metres along the perimeter of the landfill.

It is also recommended that the rate of porewater pressure dissipation in the underlying clay be monitored by means of vibrating wire piezometers installed at the time of landfill cell construction at various depths in the upper portion of the silty clay deposit. As discussed in Section 11.1.1, this data, in conjunction with the monitoring of the lateral deformations of the silty clay beneath the perimeter berms and monitoring of the landfill subgrade settlements should permit ongoing evaluation of the actual permissible finished slope/cover geometry, based on the strength gain that will occur as the underlying clay consolidates and compresses. Additional laboratory triaxial testing would be needed to provide the necessary soil parameters for these analyses. For the installation of these piezometers, it would not be planned to fully penetrate the silty clay layer (i.e., they would only be installed in the upper portion of the deposit) and the boreholes would be fully grouted; a path for preferential leachate migration to the underlying more permeable strata would not be created.

## 11.6 Buildings and Site Grading

As discussed previously, the focus of this overall section of the report has been the geotechnical design aspects of the landfill geometry. The following preliminary/general comments are provided for other Site components:

- Given the limited capacity of the underlying soils to support additional load/stress without experiencing significant compression, the overall grade raise on the Site (as required for Site drainage purposes) would ideally be restricted to a low value. A grade raise of no more than about 0.6 metres would likely be required if the general ground settlements are to be limited to very low values. However, it is understood that this level of grade raise is unlikely to be feasible. It is expected that, for grade raises of up to about one metre in magnitude, the settlements would be limited to values which could feasibly be accommodated by on-going Site maintenance. Grade raises of more than about one metre may require mitigating measures and/or perpetual and costly maintenance. A particular issue would be the differential settlements around pile-supported buildings (see next item); the settlements at the entrance thresholds could impede equipment movements. One option that could be considered would be to preload portions of the Site and to thereby have some of the settlement occur prior to the Site being developed. Consideration could also be given to a test filling program, to monitor actual settlements, and thereby refine the theoretical predictions that have been made using the *in-situ* and laboratory testing data.
- The overall Site development will include the construction of several buildings. Given the limited capacity of the silty clay deposit to support foundation loads, it is expected that the buildings will need to be supported on deep foundations, such as driven steel piles which derive their support from end-bearing on the bedrock. It is also expected that, given the anticipated grade raises (which are likely to exceed 0.6 metres), and the potentially significant floor loading, it will probably be necessary to provide the buildings (or at least some buildings) with structural floor slabs, which are supported on deep foundations.
- Shorter/lighter buildings could potentially be supported on helical pier foundations, which are supported below the softest portions of the clay deposit.
- Based on Site-specific shear wave velocity profiling completed at the this Site, the average shear wave velocity of the upper 30 metres of overburden soils has been established as less than 180 m/sec. A Site Class E would likely apply for the seismic design of buildings at this Site.

The feasibility of a larger-scale ground improvement program could also be evaluated for this Site. The use of light weight fill materials, such as expanded polystyrene Geofoam blocks, could also be considered in some applications/locations on this Site, to lessen the applied load on the clay and reduce the expected settlements.