APPENDIX Q

Technical Memorandum – Seismic Stability Dynamic Analyses

December 2014
Report No. 12-1125-0045/4500/vol III

TECHNICAL MEMORANDUM

DATE April 7, 2014 **REFERENCE No.** 12-1125-0045

DYNAMIC ANALYSIS – CRRRC SITE, OTTAWA, ONTARIO

This Technical Memorandum presents the results of dynamic analyses carried out for the Capital Region Resource Recovery Centre (CRRRC) landfill that is proposed to be in the eastern part of the City of Ottawa, in the former Township of Cumberland and just southeast of the Highway 417/Boundary Road interchange.

The purpose of the dynamic analysis was to investigate the seismic stability of the conceptual landfill configuration in its closure configuration, at the end of filling to its maximum height, when subjected to strong earthquake shaking. The analyses were carried out for the landfill configuration depicted in the Site Development Plan with approximately 25 m of waste placed to an elevation of about 100 masl and the results from the worst case analysis are presented herein.

This memorandum presents a summary of the methodology used to assess the seismic stability and earthquake-induced deformations in the waste materials and the underlying foundations, and the results of the analyses. Information on the existing subsurface conditions was obtained from the investigations carried out by Golder Associates Ltd. (Golder), as well as from the findings of previous investigations in the area.

1.0 SITE SUBSURFACE CONDITIONS

The overburden soils underlying the CRRRC Site (Site) comprise topsoil overlying surficial sandy soil of about 0.3 to 1.3 m thickness underlain discontinuously by weathered silty clay. The surficial materials and weathered silty clay typically have a combined total thickness of about 1.5 m. Underlying the weathered clay is a deposit of highly plastic silty clay (PI = 30 to 80) that is about 30 m in thickness. The silty clay deposit is, in turn, underlain by approximately 2 to 8 m of basal gravelly glacial till, followed by bedrock. The bedrock consists of inter-bedded shale and limestone of the Carlsbad Formation.

The upper portion of the unweathered silty clay deposit, under current overburden stress conditions, has a soft consistency to a depth of about 9 to 10 m. Below this depth, its shear strength increases with depth gradually and becomes stiff. The silty clay deposit has a high natural water content, typical of the Leda marine clay deposits underlying the Ottawa area.

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2.0 SEISMICITY AND GROUND MOTION PARAMETERS

The seismicity at the site results from upper to mid-crustal earthquakes with predominantly thrust (reverse) faulting mechanisms. The Western Quebec Zone comprising the urban areas of Montreal, Ottawa-Hull and Cornwall was the site of at least three significant earthquakes in the past. An earthquake estimated at magnitude M5.8 shook Montreal in 1732. In 1935, the area of Temiscaming was shaken by an M6.2 earthquake. In 1944, an M5.6 earthquake occurred between Cornwall, Ontario and Massena, New York. Figure 2-1 shows some significant earthquakes that have occurred in eastern Canada. The hypocentres of these earthquakes are reported to be located at depths varying from 10 to 20 km below ground surface.

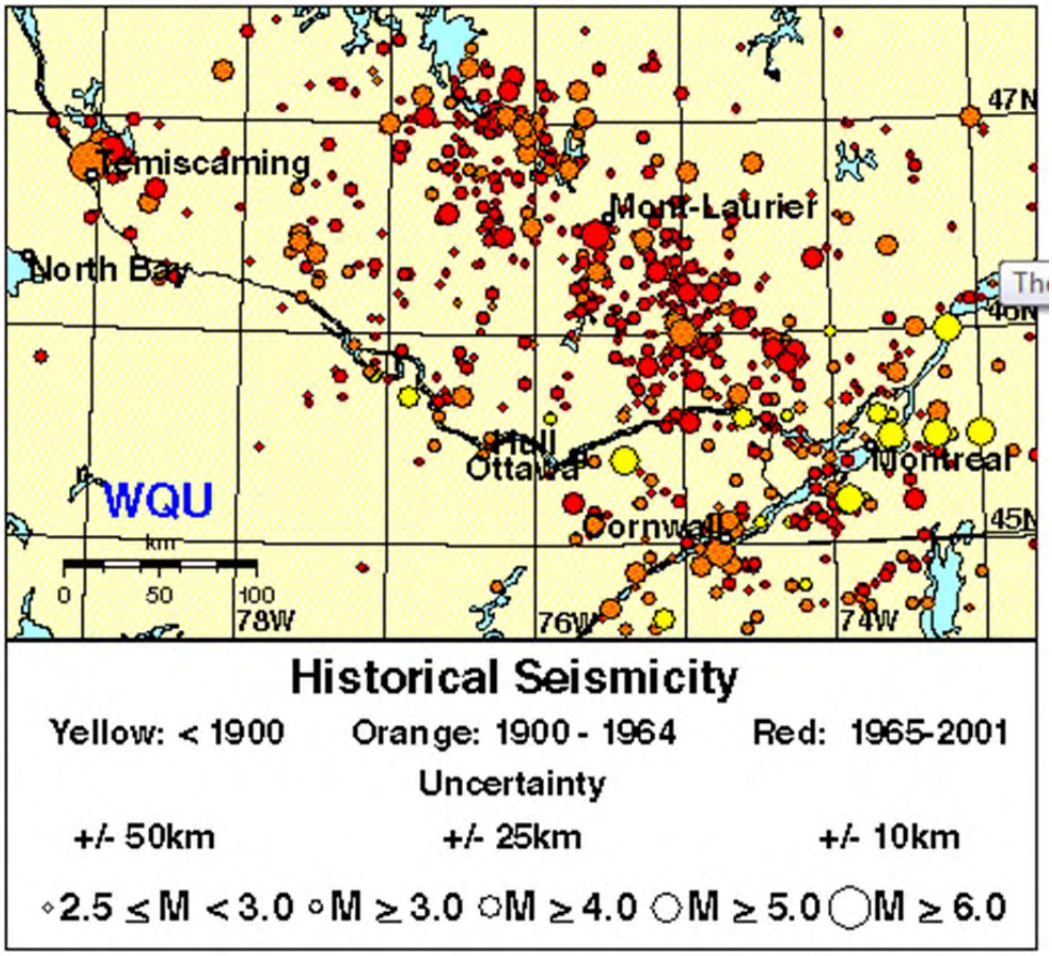

Figure 2-1: Historical Earthquakes in Eastern Canada

2.1 Seismic Design Criteria

Seismic design guidelines established for solid waste landfills in the USA require that such facilities be designed to resist ground motions with a return period of 1:2,475-yrs (ref. RCRA Subtitle D (258) – Seismic Design Guidance for Solid Waste Landfill Facilities published by the Environmental Protection Agency). This level of ground shaking is also consistent with the seismic design provisions in the 2010 NBCC for buildings. Consistent with the guidelines established in practice for similar facilities, earthquake ground motions with a return period of 1:2475-yrs have been considered for the design and analysis of the subject landfill.

2.2 Site Ground Motions

Seismic ground motion parameters for the site have been evaluated using the seismic hazard models and seismogenic zones developed on a regional basis by the Natural Resources Canada for use in the National Building Code of Canada for a return period of 2,475-years (equivalent to having a 2% chance of being exceeded in 50 years). In the analyses completed for this study, the seismic ground motions that correspond to a return period of 2,475-years were used as input and were propagated from bedrock upwards towards the ground surface using ground response analysis models. The models consider the influence of 25 m of waste, about 35 m of silty clay, glacial till, and 10 m of bedrock underlying the site on site response. The input ground motions were applied at bedrock, which has been characterized based on in-situ measurements as having a shear wave velocity in excess of 1,500 m/s (*i.e.*, Site Class A). The applicable input ground motions were developed following a four-step procedure:

Step-1: Obtain the ground motion parameters for the Reference Ground Condition (RGC) at the site from the interactive website maintained by Natural Resources Canada for the site coordinates. These ground motion parameters correspond to Site Class C, which is the RGC where the average shear wave velocity in the top 30 m varies between 360 m/s and 760 m/s.

Step-2: De-aggregate the seismic hazard at periods varying between 0.2s and 2.0s to establish the mean earthquake magnitude and distance that dominates the seismic hazard at the subject site.

Step-3: Develop the response spectrum that corresponds to Site Class A ground motion parameters using the short and long-period amplification factors F_a and F_v as per Tables 4.1.8.4B and 4.1.8.4C of NBCC (2010).

Step-4: Select applicable outcropping acceleration time-histories for Site Class A and for the mean earthquake magnitude that dominates the seismic risk at the subject site following guidelines for ground motion selection by Atkinson (2009).

The Site Class A spectrum derived using the code-based F_a and F_v values following the above-described procedure provides a conservative estimate of spectral accelerations when compared with the spectral accelerations derived from the RGC Factors recommended for hard rock in GSCs Open File No. 4459.

The 5-percent damped target response spectrum established for Site Class C (RGC) is shown in Table 2-1. The response spectrum for Site Class A developed using F_a and F_v factors of 0.75 and 0.5, respectively, is also provided in Table 2-1.

Table 2-1: Ground Motion Parameters

Return Period	PHGA	Sa (0.2s)	Sa (0.5s)	Sa (1.0s)	Sa (2.0s)
2,475-Years [RGC or Site Class C] (2% probability of exceedance in 50 years)	0.32 g	0.64 g	0.31 g	0.14 g	0.05 g
2,475-Years [Site Class A – F_a = 0.75 and F_v = 0.50] (2% probability of exceedance in 50 years)	0.24 g	0.48 g	0.16 g	0.07 g	0.02 g

Note: In Table 2-1, PHGA refers to peak horizontal ground acceleration; Sa refers to the 5-percent damped spectral acceleration for a given period.

The de-aggregated hazard for the subject site has been obtained from Geological Survey of Canada and Figure 2-2 shows the de-aggregation results for the 2,475-year ground motions for spectral accelerations with periods of 0.2 and 1.0 seconds. The de-aggregated hazard indicates that the earthquake characteristics at the subject site correspond to "mean" earthquake magnitudes ranging between M6 and M7 and the associated distances will be between 25 km and 72 km.

(a) Sa(0.2s)

Mean Magnitude = 6.38, Mean Distance = 38 km

(b) Sa(1.0s)

Mean Magnitude = 6.85, Mean Distance = 63 km

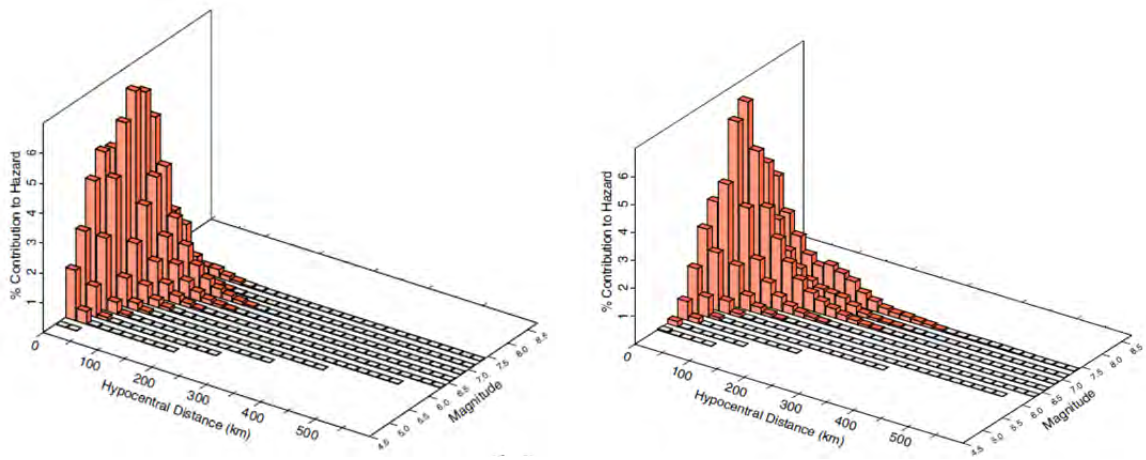

Figure 2-2: Magnitude-Distance Distributions for Spectral Accelerations at Periods of 0.2 and 1 sec.

Figure 2-3 illustrates the response spectra for the site corresponding to Site Class A and Site Class C ground conditions.

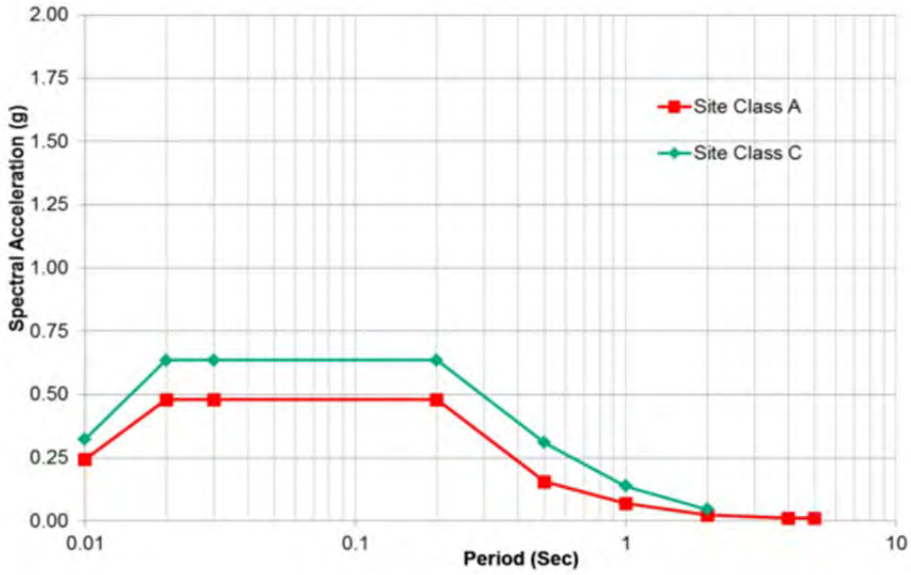

Figure 2-3: Response Spectra for Class A and C Ground Conditions

2.3 Site Classification for Above Ground Facilities

The ground motions described in Section 2.2 above correspond to Site Class A ground condition, or for hard rock, that is present at a depth of about 35 m below the existing ground surface. The near surface ground motions for the design of structures as per the seismic design provisions in the NBCC (2010) are derived based on the following:

- Site Class established based on the time-average shear wave velocity of the overburden soils in the upper 30 m as per Table 4.1.8.4A of the NBCC.
- Shaking level-dependent amplification factors F_a and F_v that correspond to the Sa(0.2s) and Sa(1.0s) established for the RGC and the Site Class as per Tables 41.8.4B and 4.1.8.4C of the NBCC.

Based on site-specific shear wave velocity profiling completed at the subject site, the average shear wave velocity of the upper 30 m of overburden soils has been established as less than 180 m/s (see Figure 4-4 for the profile of shear wave velocity). The overburden soils are, in turn, underlain by bedrock with an average shear wave velocity in excess of 1,500 m/s providing a significant contrast in impedance that can result in amplification of ground motions as they propagate from bedrock towards the ground surface.

The ground surface response spectra for the design of structures supported on shallow foundations may be computed using F_a = 1.2 and F_v = 2.1 as per Tables 4.1.8.4B and 4.1.8.4C and appropriate for Site Class E and Sa(0.2) and Sa(1.0) values summarized in Table 2-1 above for the subject site.

2.4 Bedrock Acceleration Time-Histories

Bedrock acceleration time-histories that correspond to the earthquake magnitudes and distances based upon the determination of the seismic hazard at the subject site were selected for use in the site response analyses. The acceleration time-histories that correspond to a Site Class A conditions were selected from the database maintained by the University of Western Ontario (UWO) (Ref. Engineering Seismology Toolbox at www.seismotoolbox.ca).

The UWO database contains synthetic earthquake records for Eastern Canada for various site classes including Site Class A. The records are available for earthquake magnitudes of M6 and M7. For purposes of this analyses, a total of six M7 earthquake records were selected and they were linearly scaled to match the target uniform hazard response spectrum (UHRS) for the site over the period range of interest of 0.3 to 0.8 sec considering the fundamental period of the soils underlying the site based on the guidelines provided by Atkinson (2009).

The bracketed duration of strong shaking of the selected time-histories varies between 10 and 15 seconds. Details of the selected earthquake records are summarized in Table 2-2.

Table 2-2: Selected Rock Earthquake Records

Ground Motion Identifier	Magnitude	Distance (km)
Earthquake No. 4	7	50
Earthquake No. 5	7	50
Earthquake No.11	7	50
Earthquake No.17	7	63
Earthquake No.37	7	96
Earthquake No.44	7	99

The bedrock acceleration time-histories of the modified records are shown on Figure 2-4; the corresponding response spectra, and the target response spectrum for the site are shown on Figure 2-5.

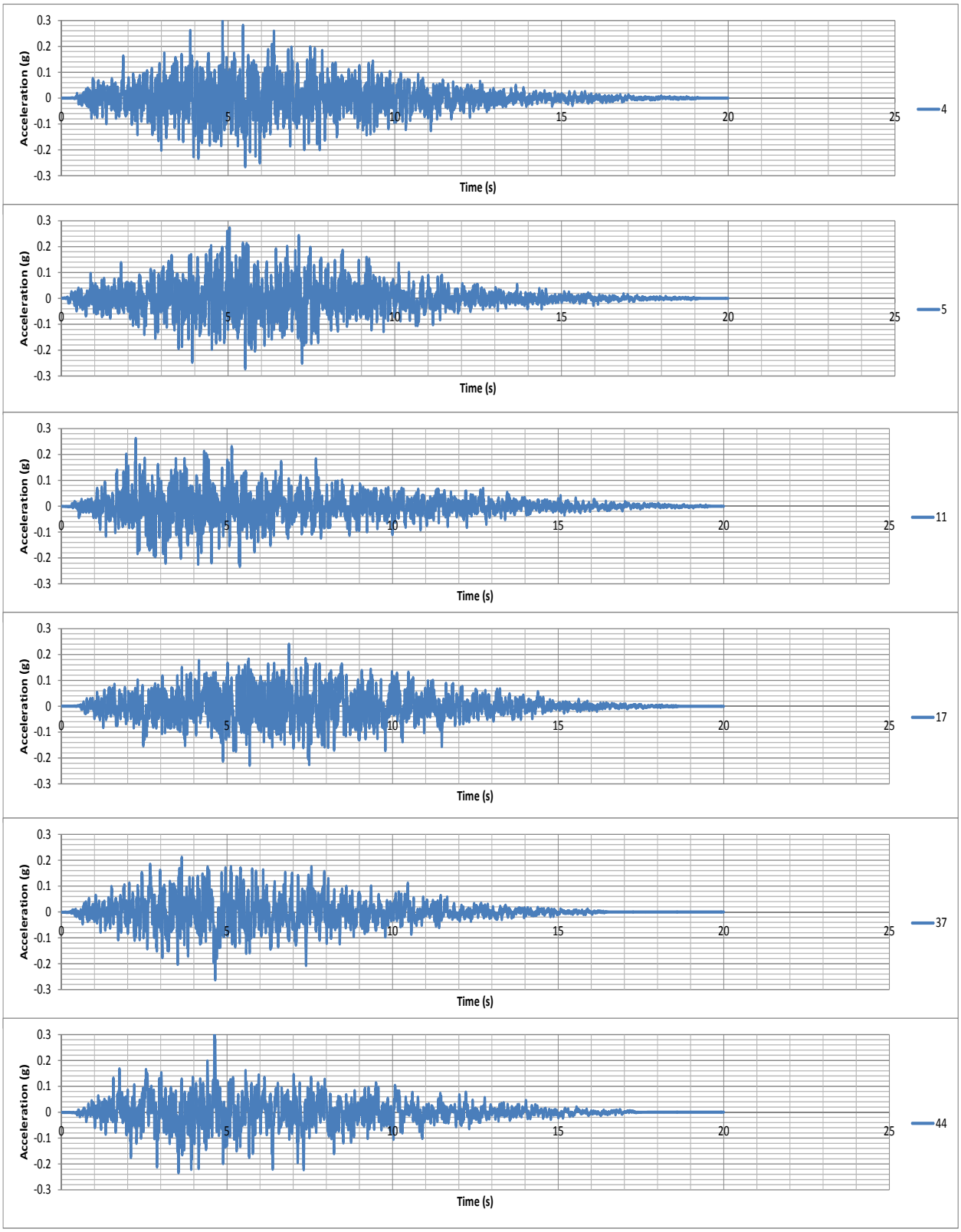

Figure 2-4: Modified Bedrock Acceleration Time-Histories

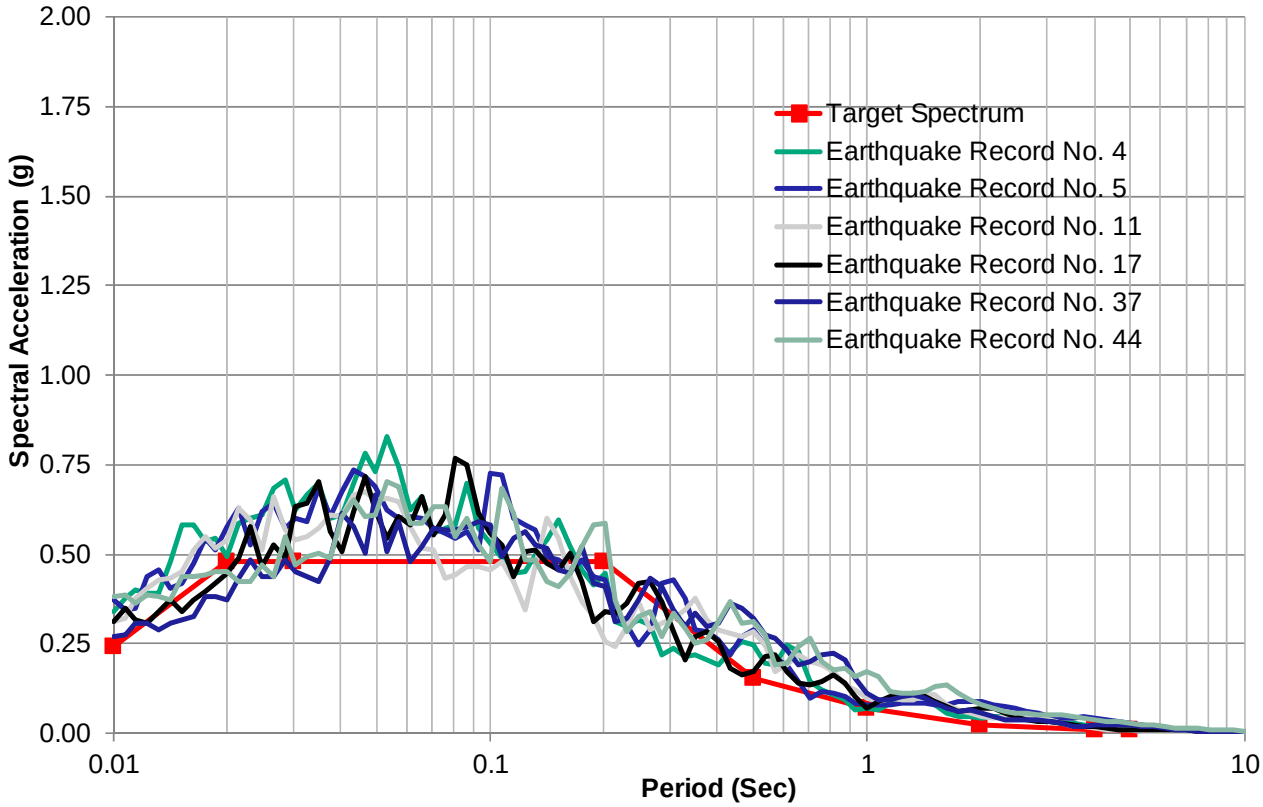

Figure 2-5: Target Response Spectrum and Spectra of the Modified Rock Records

3.0 ANALYSIS METHODOLOGY

Non-linear dynamic time-history analyses were carried out to assess the seismic stability and deformations of the CRRRC landfill at the closure condition when subjected to an excitation level with a 2,475-year return period. Considering the time required to reach the closure condition (planned period of 30 years) in comparison to the return period of the design ground motions (*i.e.,* 2,475-years), analyzing the landfill response for this case is, in our opinion, considered appropriate. The key features of the analyses completed are presented herein.

The dynamic analyses were carried out considering two-dimensional plane strain conditions. In a strict sense, such analyses are representative of stress-deformation conditions along the centerline of the landfill. The model boundaries were extended all the way to the bedrock in the vertical direction and some 160 m to either side of the toe of the landfill or the perimeter berms. The deformations estimated at the landfill corners were established as the resultant of deformations in the two directions. A schematic of the two-dimensional model developed for the analysis of landfill response is shown in Figure 3-1.

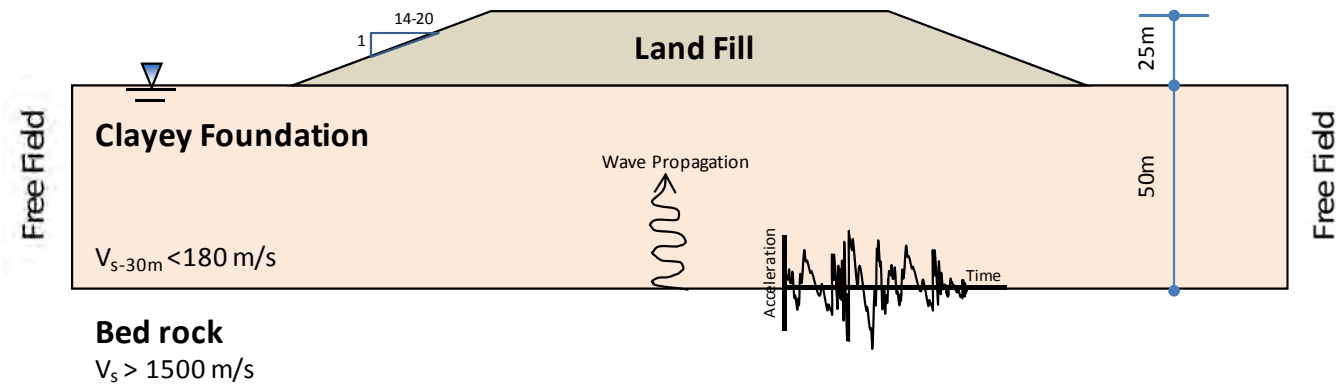

Figure 3-1: A Schematic of the 2D Plane Strain Model Considered for Deformation Analysis

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 coupled stress-flow-deformation response of earth structures that can undergo large deformations under static and dynamic loading conditions. It has several built-in and widely-used constitutive models (*e.g.,* elastic and Mohr-Coulomb) and also has the capability to incorporate user-defined constitutive models. Although the code has the capability to carry out consolidation and seepage analyses, such analyses were not conducted as part of this specific assignment.

FLAC utilizes a time-marching procedure to solve the equations for static and dynamic loading. This procedure first invokes the equations of motion to derive new velocities and displacements from stresses and forces. Then, strain rates are derived from velocities, and new stresses from strain rates. It takes one time-step for every cycle around the loop. The internal dynamic time-step is chosen to be sufficiently small (in the order of 10^{-5} to 10^{-6} seconds) so that no information will physically be transmitted from one element to another over the selected time interval. The dynamic time-step is computed considering the highest stiffness assigned in the model.

It has been considered that the landfill at the Site will be constructed after a shallow excavation is made into the surficial silty sand and weathered clay, with a base level close to the underlying soft clayey soils without the provision of a synthetic liner. The waste materials of the landfill extend from about elevation 76 masl to its maximum elevation at 100 masl over an estimated period of filling of some 30 years. The self-weight loads imposed by the landfill materials will induce consolidation settlements in the underlying clayey soils. The consolidation of those soils will increase the strength and stiffness of the clay foundation soils with time; *i.e.*, larger increases with longer time. The consolidation-induced strength and stiffness gain in the clayey soils were estimated based on SETTLE^{3D} analyses carried out as part of the geotechnical analysis of the landfill to simulate the static loading-induced settlements as a function of time. The estimated degree of consolidation at the end of filling, and beneath the 'youngest' phases of the landfill (*i.e.*, Phases 6, 7, and 8), ranged between 0 and 18% within the upper 7 m of foundation soils and 0% in the soils below 7 m as shown in Figure 3-2. These effective degrees of consolidation were used in the analyses to estimate the undrained shear strength and stiffness at the end of filling.

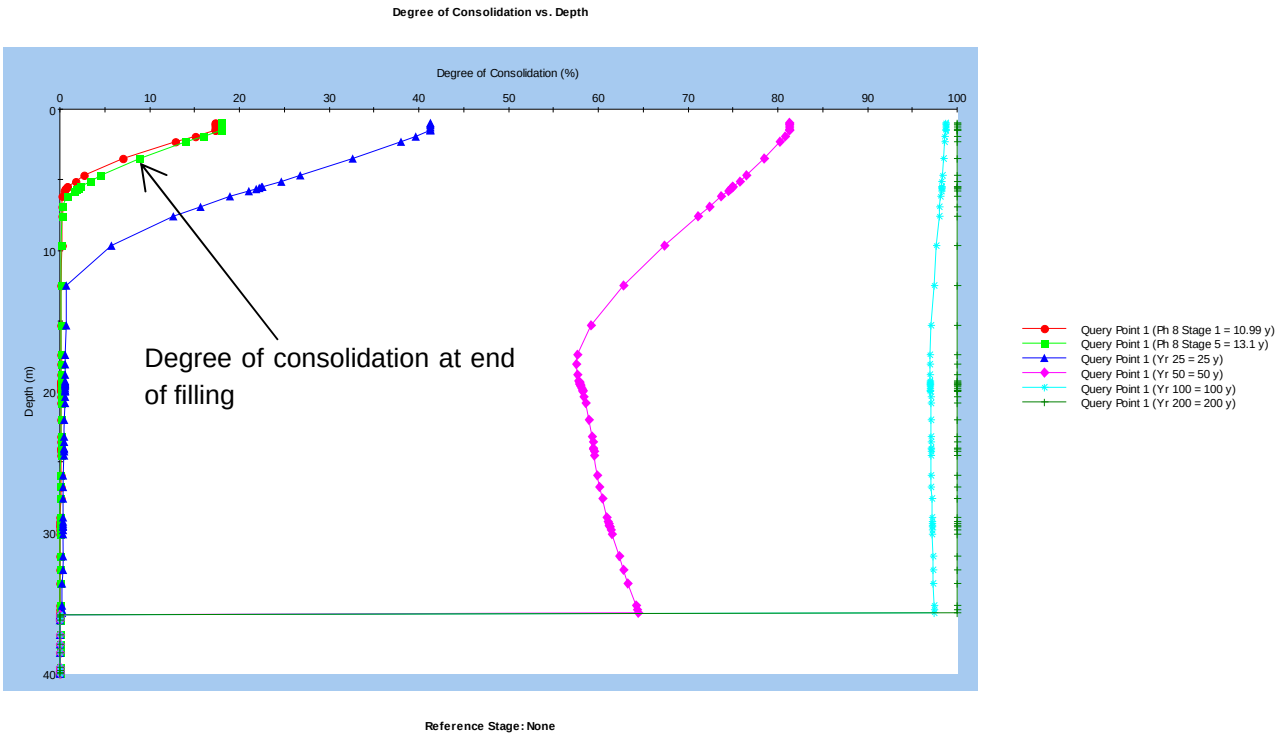

Figure 3-2: Consolidation of Foundations Soils Under Landfill Loading
[Note: Analysis Assumes Landfill Built to Full Height In Stages. The analysis reference time ("time zero") was from the start of filling of Phase 6, with Phase 8 being completed/filled 13.1 years thereafter]

The response of the landfill-foundation system to earthquake shaking was carried out in two stages. In the first stage, the landfill and its foundation was analyzed with the strength and stiffness parameters that reflect the effects of consolidation of clayey soils under gravity loads (static mode) to establish the pre-earthquake stress state. Thereafter, the analysis was switched to the dynamic mode with updated undrained strength and stiffness properties assigned to the foundation soils.

The foundation soils and waste were modeled as Mohr-Coulomb materials under gravity (or static) loading conditions. Under seismic loading conditions, the waste was modeled as a Mohr-Coulomb material with an equivalent linear approach (ELA) to simulate the non-linear cyclic behavior, including shear modulus degradation with increasing shear strain and shear strain-dependent damping, using the three parameter hysteretic model built into FLAC. The foundation soils were modeled as non-linear Mohr-Coulomb materials using the hyperbolic stress-strain model and modified Masing's rule incorporated in the user-defined routine UBCHYST developed at the University of British Columbia, Vancouver, Canada. The bedrock was modeled as a linear-elastic material.

4.0 FLAC^{2D} MODEL AND MATERIAL PARAMETERS

The proposed ~25 m maximum height landfill section with a near-flat crest was discretized into 2,600 smaller zones of 1 m in height, supported on a 50 m thick foundation layer. Considering the soft and compressible silty clay underlying the site, the waste slopes were considerably flatter than typical for landfills constructed in locations with stronger foundation soil conditions, and varied from 14H:1V in the lower portion and becoming flatter to 20H:1V from just above the mid height to the crest (as indicated for the Site Development Plan).

The foundation layer comprised some 7,300 smaller zones with the nominal height of each zone varying from 1 to 5 m. The foundation zone extended laterally about 160 m from the toe of the landfill on either side. The water table was assigned at a depth of 1 m below the existing ground surface.

Figures 4-1 and 4-2 show the FLAC model with the different material zones used in the analyses. As shown in the figures, the landfill model comprises the waste material and two perimeter toe soil berms. The foundation model includes clayey soil layers (*e.g.*, soft upper clay), glacial till and part of the bedrock foundation. It was assumed that the upper surficial sand materials will be excavated and removed from within the footprint of the landfill to create the base of the landfill at shallow depth below original ground surface.

The analyses were conducted using the total-stress approach where undrained shear strength parameters are 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 (Ladd & DeGroot, 2004) and suggestions by Mesri & Huvaj, (2007) using data from the Boundary Road site investigations (*i.e.*, vane tests, CPT tests). Figure 4-3 shows the as-is undrained shear strength (S_u) profile established from the in-situ strength measurements and laboratory consolidation testing of undisturbed samples. The shear strength profile selected for design is also shown on Figure 4-3.

The shear stiffness (*i.e.*, G) of the foundation soils was evaluated based on site-specific in-situ shear wave velocity (V_s) measurements, obtained using the VSP (Vertical Seismic Profiling) method. Figure 4-4 shows the details of the V_s data for the site along with the design shear wave velocity profile used in the analyses. On average, the measured shear wave velocity of site soils is less than 180 m/s.

A time-averaged shear wave velocity close to 140 m/s is representative of the site, which translates to a site fundamental period of about 1 sec. Based on micro-tremor studies completed for the Ottawa Area (ref. Motazedian et al, 2010), the fundamental period of the site is estimated to be between 0.8 and 1.2 seconds. The estimated site period compares well with the micro-tremor measurements.

The silty clay foundation soils underlying the site are of marine origin, with as-is peak undrained shear strengths varying from 10 to 15 kPa in the upper 7 m and varying to in excess of 70 kPa at a depth of 35 m below ground surface. The strength sensitivity (S_t), expressed as the ratio of the peak undrained shear strength to remolded strength, generally varies between 4 and 14 with an average of 9, indicating medium to extra high sensitivity (CFEM, 2006). 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 that correspond to the anticipated intensity of site-specific cyclic loading. At higher intensities of cyclic loading and with the application of a large number of cycles of shear loading (*i.e.*, N = 100 to 500), the foundation soils would be expected to soften considerably, resulting in significant reductions in the shear stiffness and undrained shear strength. However, these stress levels and numbers of cycles are in excess of those anticipated for the seismic loading for the Site.

The strength parameters of the waste materials were estimated based on the data found in the literature (*e.g.*, Kavazanjian et al. 1996, EPA1995, Kavazanjian et al. 2013). Figure 4-5 shows shear wave velocity data from various landfill sites reported by Kavazanjian et al. (1996) along with a recommended design profile.

Table 4-1 lists the material parameters for various soil types and waste used in the analysis.

Table 4-1: Material Properties Used in FLAC Analyses

Material Type	Total Unit Weight (kN/m^3)	Su (As Is) (kPa)	Su (At End-of-Filling)[2] (kPa)	Φ (^{o})
1. Solid Waste Landfill	12	-	-	30
2. Soft Clayey Layer[1]	15	10 to 11	No increase	-
3. Upper Clayey Layer[1]	15	Variable[1]	No increase	-
4. Lower Clayey Layer[1]	16	Variable[1]	No increase	-
5. Glacial Till	21	-		38
6. Bedrock Foundation	22	-	-	-
7. Perimeter Sandy Berm	18	-	-	28

Notes:

1 See Figure 4-3 for S_u profile.

2 The estimated degree of consolidation at the end of filling ranges between 0 and 18% within the upper 7 m (see Figure 3-2). This results in an effective stress lower than pre-consolidation pressure (Pc) of the layer, so no strength gain is predicted by this time.

5.0 RESULTS

The FLAC analysis considered the self-weight loads of the foundation followed by the application of the landfill loading. The resulting landfill-foundation system with waste placed to its full height of some 25 m was brought to equilibrium under self-weight loads. Figure 5-1 shows the computed vertical effective stress contours for the foundation (along with the boundary conditions) for static loading for the as-is case (compression is negative). As shown, the stress pattern uniformly follows the geometry of the foundation model. Figure 5-2 shows the vertical stress contours for the landfill-foundation system.

After the landfill-foundation system was brought to equilibrium under gravity loads, the dynamic mode of the FLAC program was invoked and the model was subjected to horizontal shaking at the model base. A compliant boundary was considered in the analysis. The acceleration time-histories were converted to equivalent shear stress time-histories. The user-defined stress-strain model (UBCHYST) was calibrated to represent the published modulus reduction and damping curves that correspond to clay soils with a PI = 45 (Vucetic & Dobry, 1991) and the curves recommended by Kavazanjian et al (1998) for waste materials. Figure 5-3 shows a comparison of the modulus reduction and damping curves derived from UBCHYST with the base curves obtained from literature for the foundation soils. Figure 5-4 shows the comparison of the modulus reduction and damping curves derived from the FLAC ELA approach with the base curves obtained from literature for the waste materials. While the modulus reduction curves compare well, the FLAC simulations use a conservative estimate of damping for the waste materials.

Figure 5-5 shows the acceleration time-history at the landfill crest from earthquake record No. 37. Figure 5-6 shows a typical stress-strain response for a zone in the upper clay layer (see Figure 4-1 for the location of the zone), which indicates no significant strain softening under the design ground motions.

The computed lateral displacement pattern is shown on Figure 5-7 and the results shown correspond to the end of shaking. Due to the symmetry of the landfill configuration analyzed, the results indicate that the landfill experiences similar permanent lateral movements on both sides. The computed seismic loading-induced lateral movements of the section analyzed are less than 400 mm. Figure 5-8 shows the (50 times magnified) distorted mesh compared to the original mesh (undeformed). As shown on the figure, the earthquake-induced deformations of the landfill are the result of deformations occurring in the upper clay layers directly below the landfill; *i.e.*, the waste material rides on top of the soft upper clay layer.

Analyses were carried out for a total of six earthquake records developed following the methods outlined in Section 2.0. All of the analyses showed similar patterns of deformations and the landfill configuration is predicted to be stable under the design ground motions. Table 5-1 lists the lateral and vertical seismic loading-induced displacements computed at the toe of the landfill at the end of shaking from the six earthquake records. At the corners of the landfill, the displacements may be larger by about 40% due to three-dimensional seismic loading effects.

It is noted that the maximum lateral displacements *during* shaking (*i.e.*, under transient loading conditions) were computed to vary from 210 mm to 360 mm, which is up to 20**%** higher than those computed at the end of shaking.

Figure 5-9 shows a typical displacement time-history computed for a location at the toe of the landfill illustrating the variations in the transient displacements computed during shaking and permanent displacements computed after cessation of ground shaking. These results are indicative of a relatively stable landfill under the design seismic loading conditions.

The results of the 2D FLAC analyses were compared with simplified Newmark sliding block analyses using the yield acceleration values established for typical failure surfaces and the peak ground surface acceleration estimated using 1-D wave propagation analyses, and found to be in good agreement.

Table 5-1: Computed Permanent Seismic Displacements[1] at the Toe of the Landfill

Earthquake Record	Vertical Displacement (mm) (End of Shaking)	Lateral Displacement (mm) (End of Shaking)
1. Earthquake No. 4	60	190
2. Earthquake No. 5	80	240
3. Earthquake No. 11	70	250
4. Earthquake No. 17	60	230
5. Earthquake No. 37	40	200
6. Earthquake No. 44	80	340

Note: [1] The seismic displacements summarized here are in addition to the static loading-induced settlements and correspond to conditions at the end of filling.

6.0 SUMMARY

A dynamic analysis of the proposed landfill at the Site has been carried out. The analysis was carried out for the 25 m high maximum landfill configuration with waste slopes varying from 14H:1V to 20H:1V. A geotechnical model was developed that included the waste material, the soft to stiff overburden clayey soils, till, and a portion of the bedrock foundation zone to a total thickness of about 50 m. The primary focus of the analysis was to confirm the landfill seismic stability and to quantify the anticipated deformations for the landfill-foundation system when subjected to ground motions with a return period of 1:2,475-yrs, consistent with the design shaking considered in the National Building Code of Canada. The analyses were carried out using bedrock ground motions developed for "Site Class A" ground conditions applied at a depth of some 50 m below ground surface.

The strength and stiffness properties of the waste materials were obtained from published literature. The strength and stiffness properties of the silty clay deposit comprising the foundation of the landfill were derived from site-specific in-situ measurements (*i.e.*, Nilcon vane shear and downhole shear wave velocity testing) and state-of-the-practice followed in the analysis of these soils in the Ottawa region. The stiffness properties of the till and bedrock were established based on in-situ shear-wave velocity measurements.

The dynamic analyses were carried out using the computer program FLAC (V6, Itasca, 2008) considering a Mohr-Coulomb constitutive model for the waste materials and clayey foundation, and elastic material properties for the bedrock foundation, allowing for stiffness/modulus reduction due to seismic shaking. The seismic response of the foundation soils and waste material was established based on the published data available for similar materials. Two dimensional plane strain conditions were considered.

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 maximum permanent lateral displacements of about 340 mm during shaking that corresponds to the 2,475-year return period ground motions. The resultant permanent ground movements at the corners of the landfill may be larger and by about 40% due to three-dimensional loading effects, reaching values close to 500 mm;

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 m;

4) The permanent lateral displacements are approximately symmetrical, resulting from the near horizontal soil stratigraphy and the landfill configuration;

5) The results of the 2D FLAC analyses predict displacements that are consistent with those established from simplified methods of analyses such as the Newmark sliding block analysis; and,

6) 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.

7.0 CLOSURE

We trust that the contents of this Technical Memorandum meet with the requirements of the study. Please do not hesitate to contact the undersigned, if you have questions or need clarification of contents.

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Attachments:
- Figure 4-1: FLAC Model and Material Types
- Figure 4-2: FLAC Finite Difference Grid and Material Types
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- Figure 4-5: Shear Wave Velocity Data for Solid Waste Landfills
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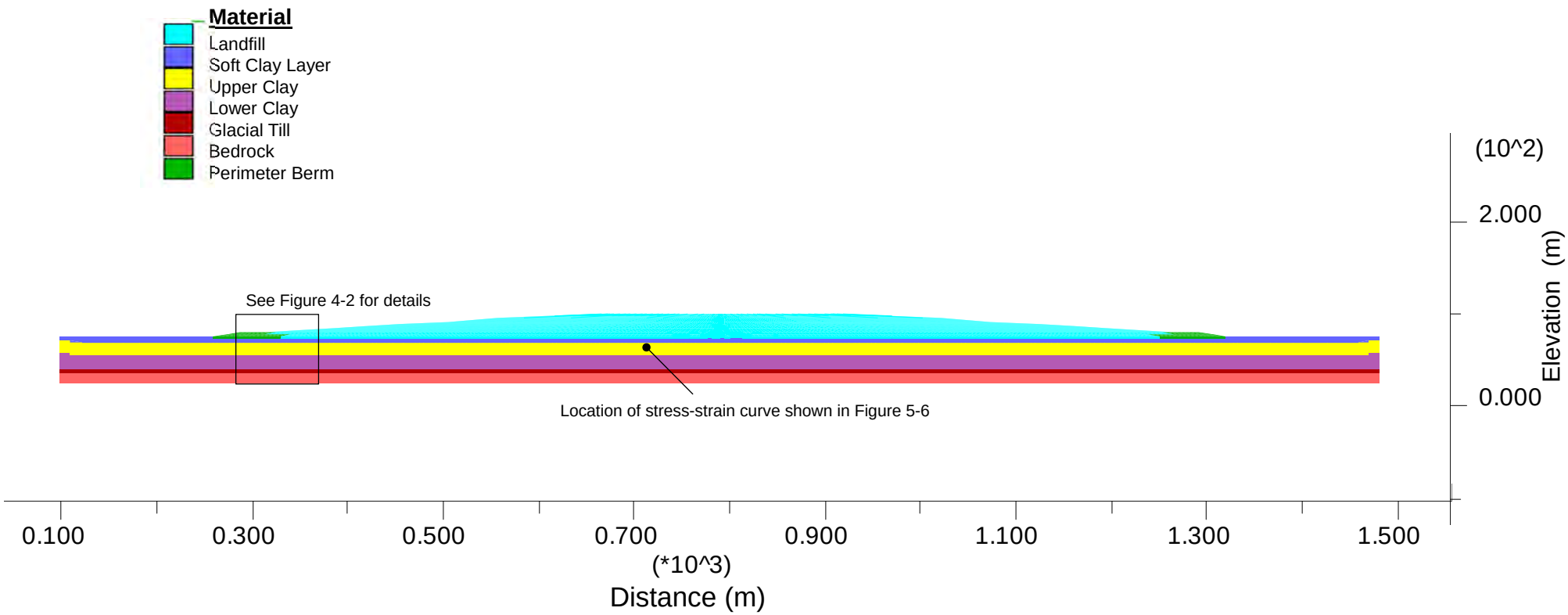
- Figure 5-5: Acceleration Time History at Landfill Crest (Earthquake No. 37)
- Figure 5-6: Typical Stress-Strain Response of Clayey Soils (Earthquake No. 37)
- Figure 5-7: Contours of Lateral Displacements End of Shaking (Earthquake No. 37)
- Figure 5-8: Distorted Mesh Compared with Original Shape (Earthquake No. 37)
- Figure 5-9: Time-History of Lateral Displacement at Landfill Toe (Earthquake No. 37)



PROJECT
CAPITAL REGION RESOURCE RECOVERY CENTRE

TITLE
FLAC MODEL AND MATERIAL TYPES

Golder Associates

PROJECT No.		12-1125-0045	PHASE / TASK No. 4500
DESIGN	MSK	30MAR14	SCALE. REV. -
CADD	----	----	
CHECK	PLE	AUG 14	FIGURE 4-1
REVIEW	PAS	AUG 14	

JOB TITLE : CRRRC MSW Landfill-Foundation system Dynamic Analysis, Ottawa

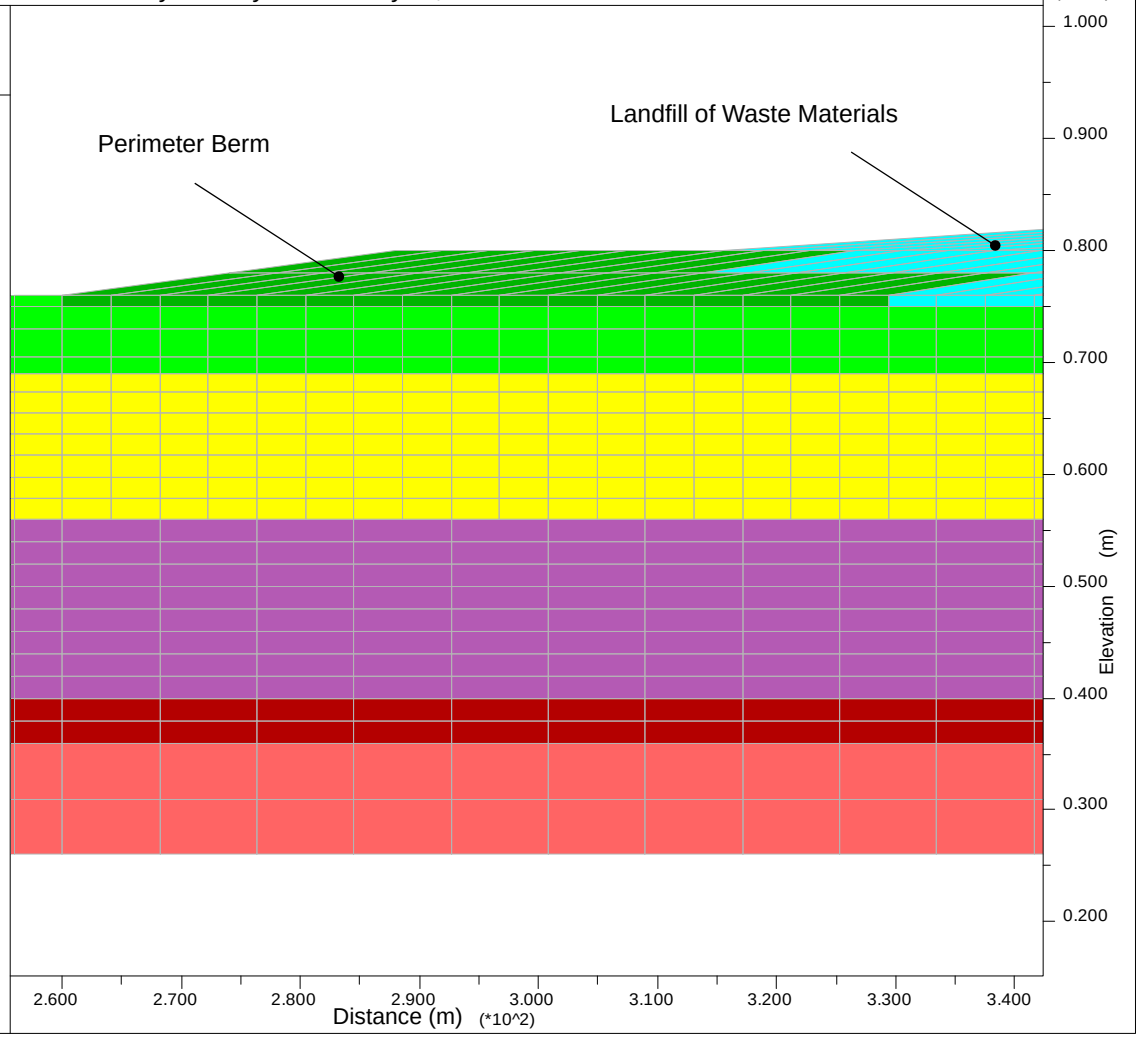
FLAC (Version 6.00)

LEGEND

27-May-13 20:55
step 55203
2.556E+02 <x< 3.424E+02
1.515E+01 <y< 1.020E+02

Material

- Landfill
- Soft Clay Layer
- Upper Clay
- Lower Clay
- Glacial till
- Bedrock
- Perimeter Berm

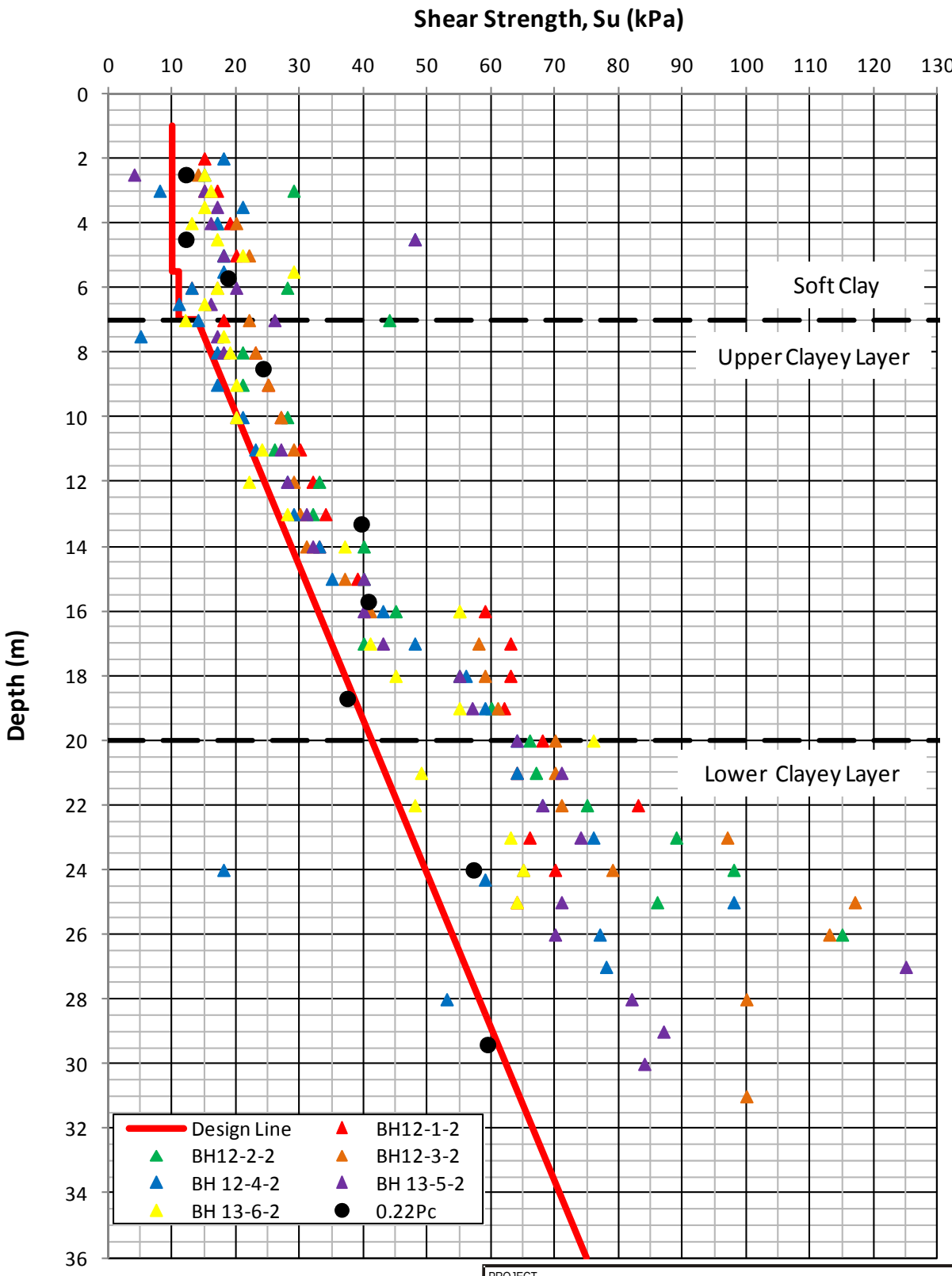


PROJECT
CAPITAL REGION RESOURCE RECOVERY CENTRE

TITLE
FLAC FINITE DIFFERENCE GRID AND MATERIAL TYPES

Golder Associates

PROJECT No.		12-1125-0045	PHASE / TASK No. 4500
DESIGN	MSK	30MAR14	SCALE. REV. -
CADD	----	----	
CHECK	PLE	AUG 14	**FIGURE 4-2**
REVIEW	PAS	AUG 14	



PROJECT

CAPITAL REGION RESOURCE RECOVERY CENTRE

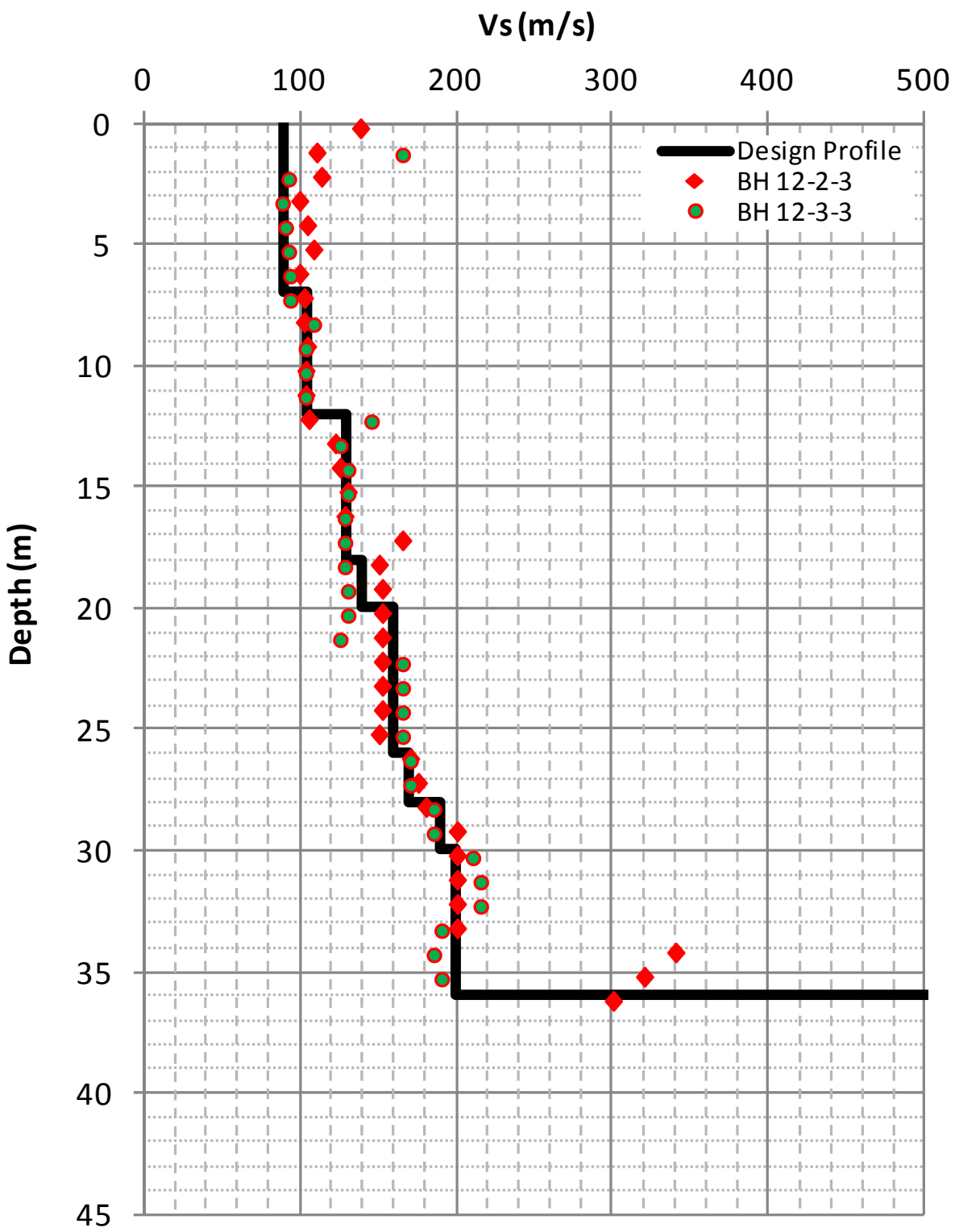
TITLE

STRENGTH PROFILE OF FOUNDATION SOILS (AS-IS)

Golder Associates

PROJECT No.	12-1125-0045		PHASE / TASK No.	4500
DESIGN	M.S.K	31 MAR.14	SCALE NTS	REV.
CADD	--	---		
CHECK	PLE	AUG 2014		
REVIEW	PAS	AUG 2014		

FIGURE 4-3

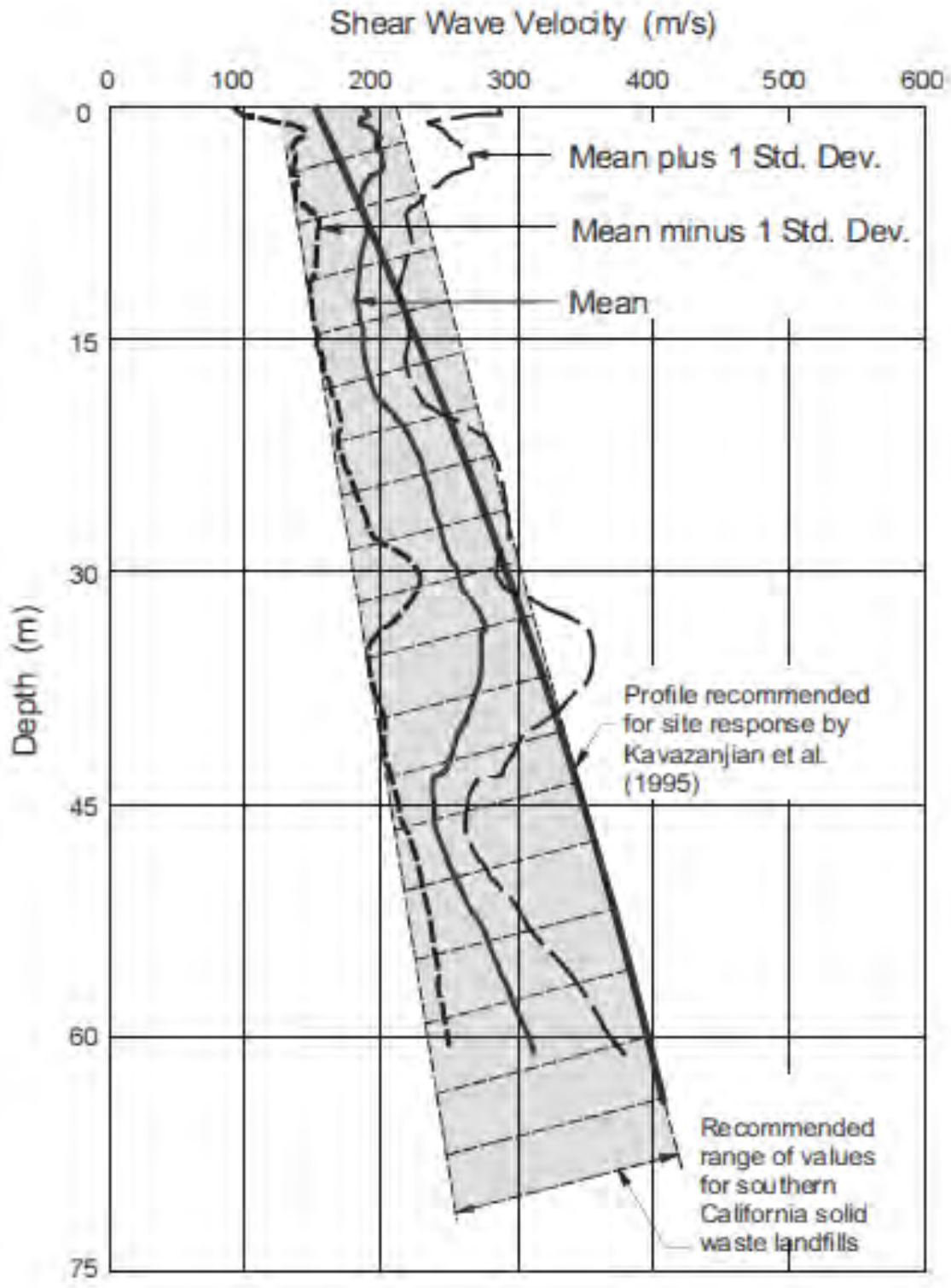


PROJECT
CAPITAL REGION RESOURCE RECOVERY CENTRE

TITLE
SHEAR WAVE VELOCITY PROFILE OF FOUNDATION

Golder Associates

PROJECT No. 12-1125-0045			PHASE / TASK No. 4500	
DESIGN	M.S.K	31 MAR14	SCALE NTS	REV.
CADD	--	--	**FIGURE 4-4**	
CHECK	PLE	AUG 2014		
REVIEW	PAS	AUG 2014		

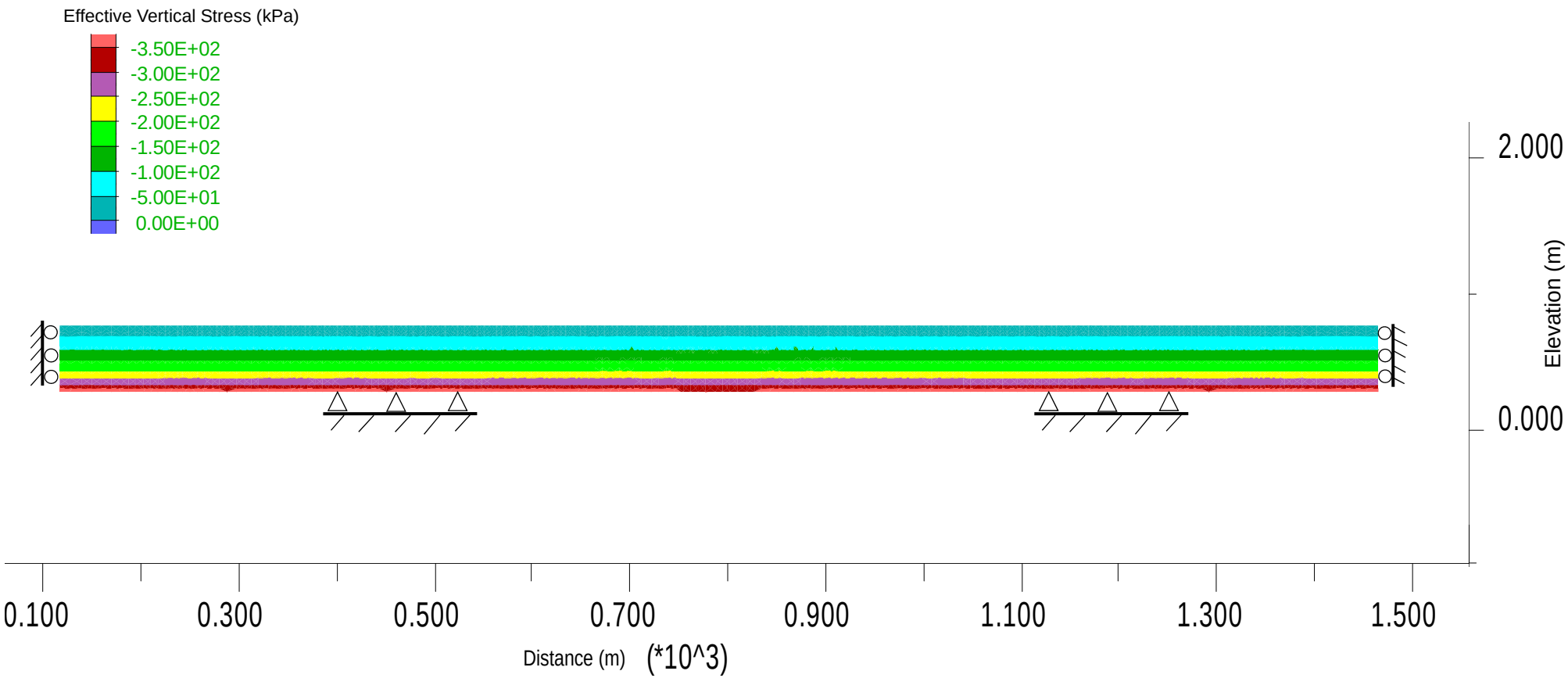


PROJECT
CAPITAL REGION RESOURCE RECOVERY CENTRE

TITLE
SHEAR WAVE VELOCITY DATA FOR SOLID WASTE LANDFILLS

Golder Associates

PROJECT No. 12-1125-0045			PHASE / TASK No. 4500	
DESIGN	M.S.K	31 MAR14	SCALE NTS	REV.
CADD	--	--	**FIGURE 4-5**	
CHECK	PLE	AUG 2014		
REVIEW	PAS	AUG 2014		



Note: Negative is compression stress

PROJECT

CAPITAL REGION RESOURCE RECOVERY CENTRE

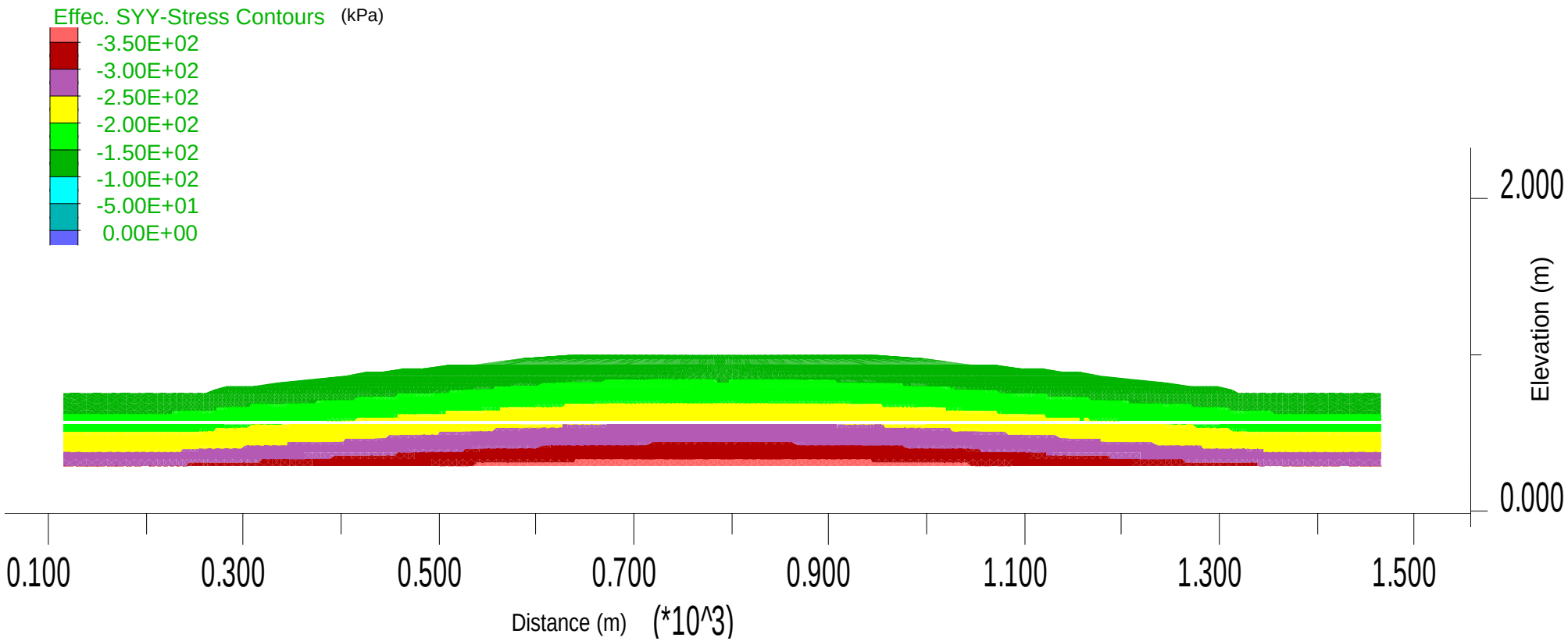
TITLE

CONTOURS OF EFFECTIVE VERTICAL STRESS FOR FOUNDATION, AS-IS

Golder Associates

PROJECT No.		12-1125-0045	PHASE / TASK No.	4500
DESIGN	MSK	31MAR14	SCALE.	REV. -
CADD	----	----		
CHECK	PLE	AUG 2014		
REVIEW	PAS	AUG 2014		

FIGURE 5-1



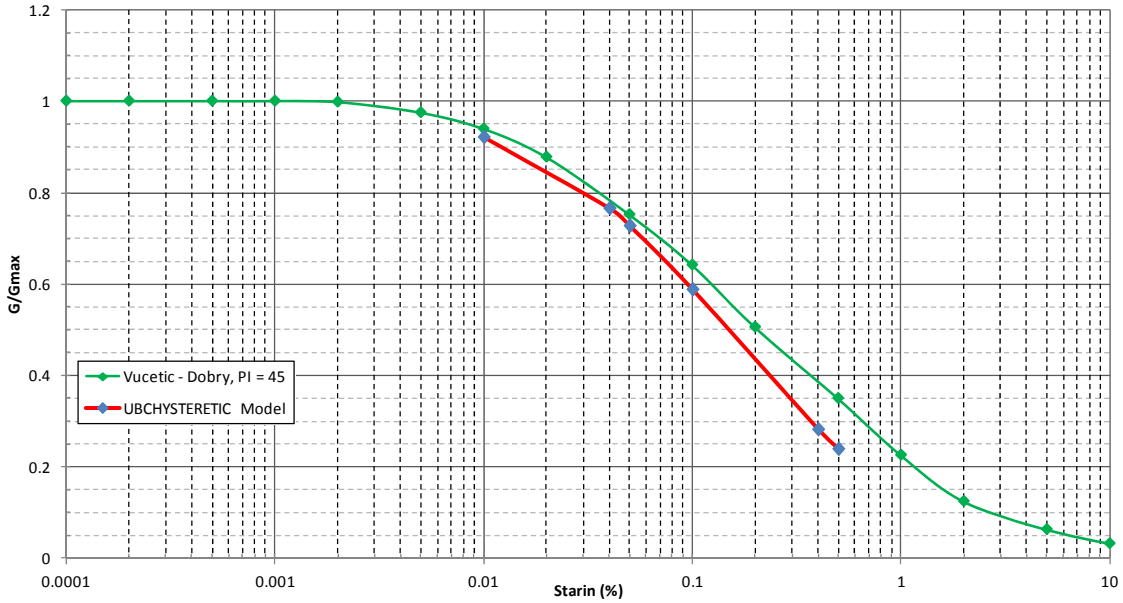
Note: Negative is compression stress

PROJECT
CAPITAL REGION RESOURCE RECOVERY CENTRE

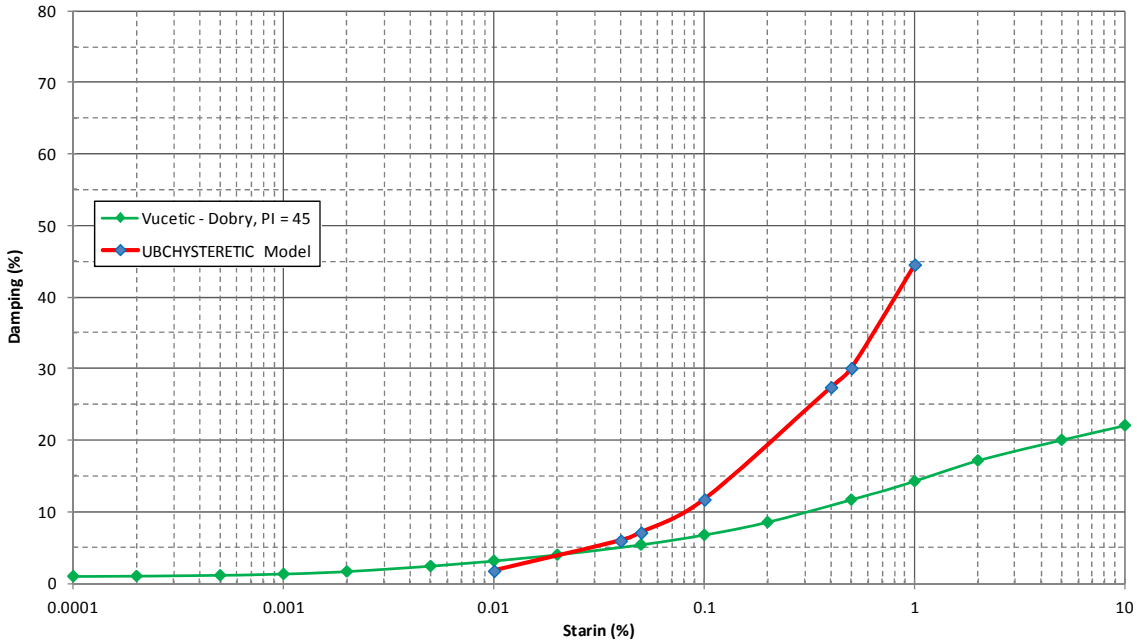
TITLE
CONTOURS OF TOTAL VERTICAL STRESS FOUNDATION-LANDFILL SYSTEM

Golder Associates

PROJECT No.		12-1125-0045	PHASE / TASK No. 4500
DESIGN	MSK	31MAR14	SCALE. REV. -
CADD	----	----	
CHECK	PLE	AUG 2014	**FIGURE 5-2**
REVIEW	PAS	AUG 2014	



Modulus Reduction Curves



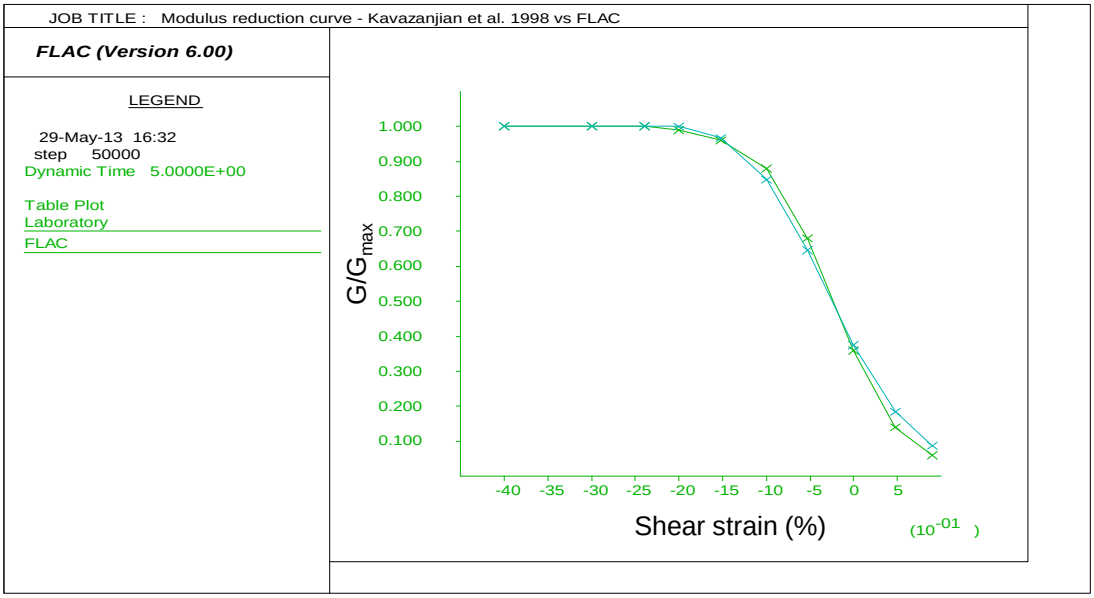
Damping Curves

PROJECT
CAPITAL REGION RESOURCE RECOVERY CENTRE

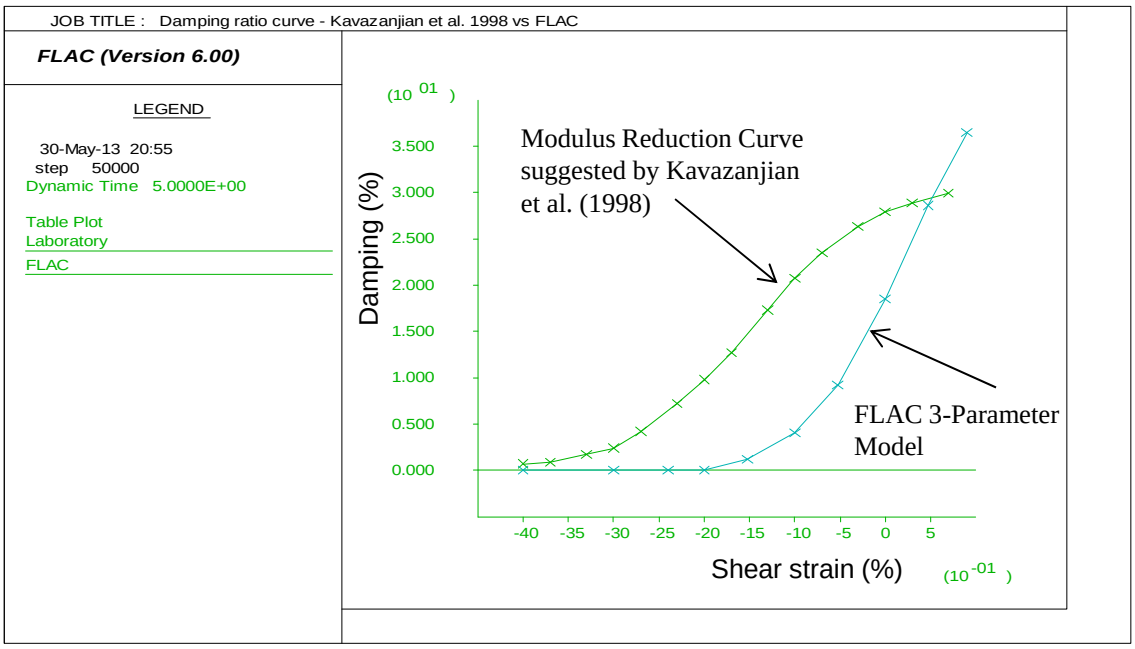
TITLE
MODULUS REDUCTION AND DAMPING CURVES FOR FOUNDATION SOILS FLAC SIMULATION VS. PUBLISHED DATA

Golder Associates

PROJECT No.	12-1125-0045		PHASE / TASK No.	4500
DESIGN	M.S.K	31 MAR.14	SCALE NTS	REV.
CADD	--	--	FIGURE 5-3	
CHECK	PLE	AUG 2014		
REVIEW	PAS	AUG 2014		



Modulus Reduction Curves



Damping Curves

PROJECT
CAPITAL REGION RESOURCE RECOVERY CENTRE

TITLE
MODULUS REDUCTION AND DAMPING CURVES FOR SOLID WASTE MATERIALS FLAC SIMULATION VS. PUBLISHED DATA

Golder Associates

PROJECT No.	12-1125-0045		PHASE / TASK No. 4500
DESIGN	M.S.K	31 MAR.14	SCALE NTS REV.
CADD	--	--	FIGURE 5-4
CHECK	PLE	AUG 2014	
REVIEW	PAS	AUG 2014	

REVISION DATE: BY: FILE:

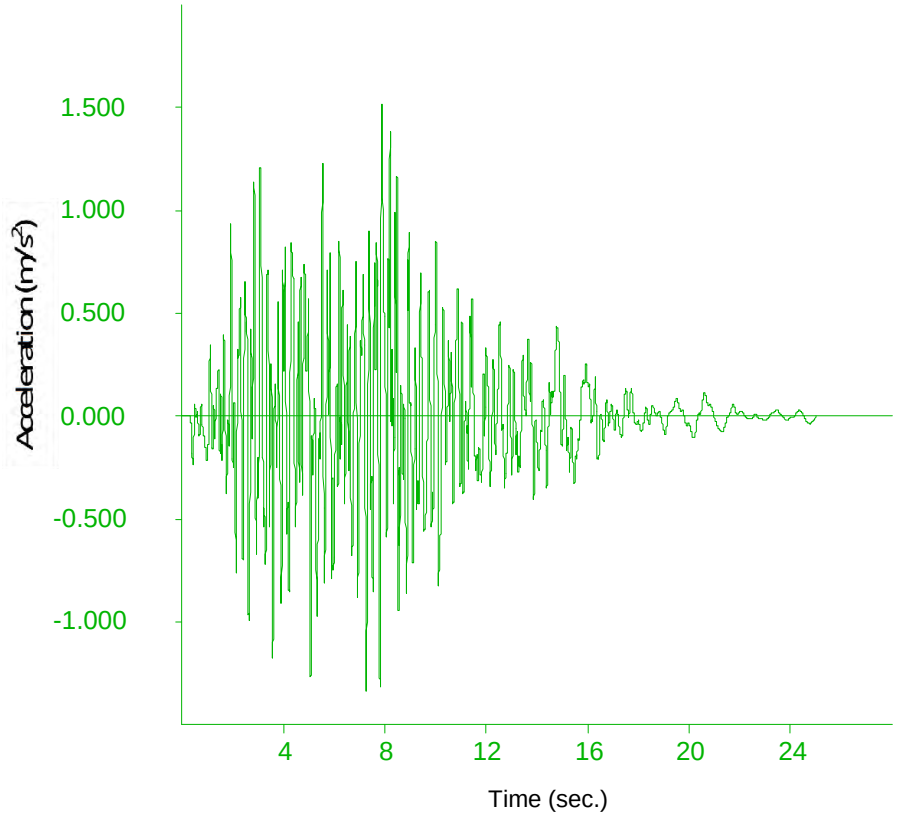
JOB TITLE : CRRRC MSW Landfill-Foundation system Dynamic Analysis, Ottawa

FLAC (Version 6.00)

LEGEND

31-Mar-14 11:28
step 2545773
Dynamic Time 2.5000E+01

HISTORY PLOT
Y-axis :
30 X acceleration(152, 35)
X-axis :
2 Dynamic time



PROJECT
CAPITAL REGION RESOURCE RECOVERY CENTRE

TITLE
ACCELERATION TIME HISTORY AT LANDFILL CREST (EARTHQUAKE NO. 37)

Golder Associates

PROJECT No. 12-1125-0045			PHASE / TASK No. 4500	
DESIGN	M.S.K	31 MAR.14	SCALE NTS	REV.
CADD	--	--	**FIGURE 5-5**	
CHECK	PLE.	AUG 2014		
REVIEW	PAS.	AUG 2014		

REVISION DATE: BY: FILE:

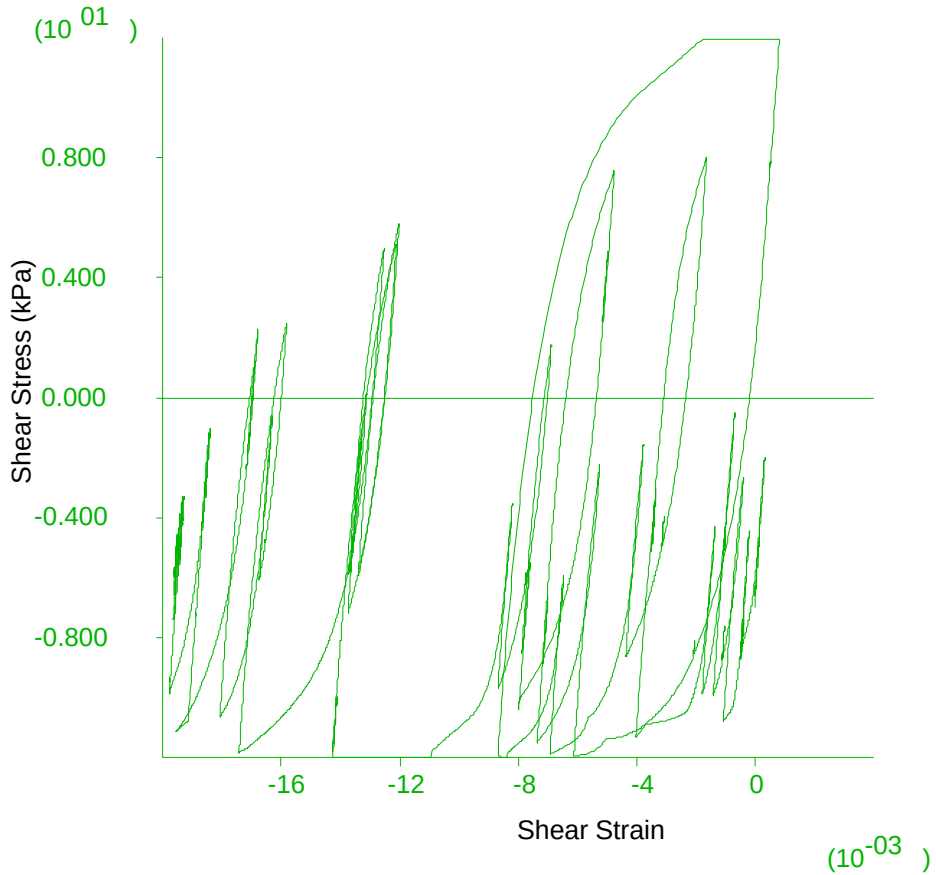
JOB TITLE : CRRRC MSW Landfill-Foundation system Dynamic Analysis, Ottawa

FLAC (Version 6.00)

LEGEND

31-Mar-14 11:12
step 2545773
Dynamic Time 2.5000E+01

HISTORY PLOT
Y-axis :
127 Ave. SXY (100, 22)
X-axis :
128 EX_13 (100, 22)

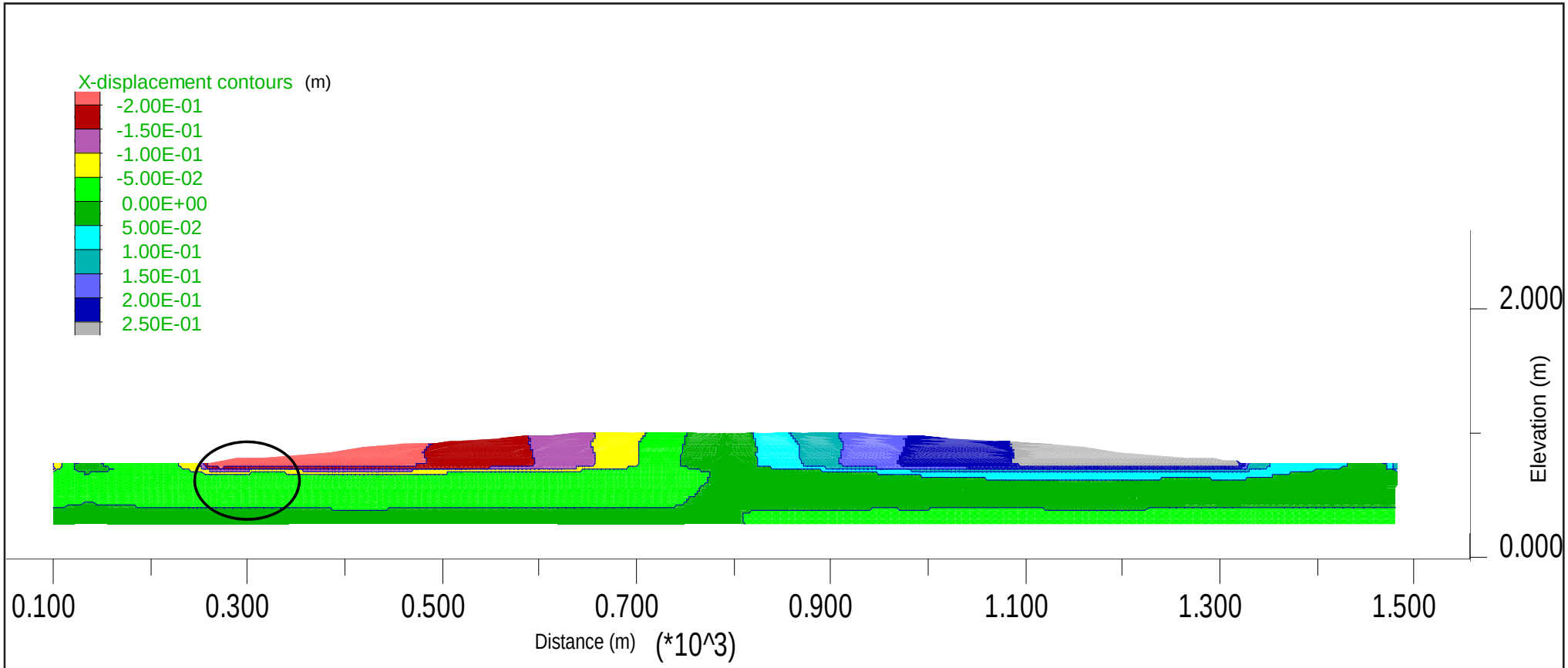


PROJECT
CAPITAL REGION RESOURCE RECOVERY CENTRE

TITLE
TYPICAL STRESS-STRAIN RESPONSE OF CLAYEY SOILS (EARTHQUAKE NO. 37)

Golder Associates

PROJECT No. 12-1125-0045			PHASE / TASK No. 4500	
DESIGN	M.S.K	31 MAR.14	SCALE NTS	REV.
CADD	--	--	**FIGURE 5-6**	
CHECK	PLE.	AUG 2014		
REVIEW	PAS.	AUG 2014		



Note: Displacement contours are in meters.

PROJECT

CAPITAL REGION RESOURCE RECOVERY CENTRE

TITLE

CONTOURS OF LATERAL DISPLACEMENTS END OF SHAKING (EARTHQUAKE NO. 37)

Golder Associates

PROJECT No.		12-1125-0045	PHASE / TASK No.	4500
DESIGN	MSK	31MAR14	SCALE.	REV. -
CADD	----	----		
CHECK	PLE	AUG 2014	**FIGURE 5-7**	
REVIEW	PAS	AUG 2014		

REVISION DATE: BY: FILE:

JOB TITLE : CRRRC MSW Landfill-Foundation system Dynamic Analysis, Ottawa

FLAC (Version 6.00)

LEGEND

31-Mar-14 12:27
step 2545773
Dynamic Time 2.5000E+01
3.047E+02 <x< 3.856E+02
1.549E+01 <y< 9.643E+01

Material

- Landfill
- Soft Clay Layer
- Upper Clay
- Lower Clay
- Glacial Till
- Bedrock
- Perimeter Berm

Exaggerated Grid Distortion

Magnification = 5.000E+01
Max Disp = 2.174E-01

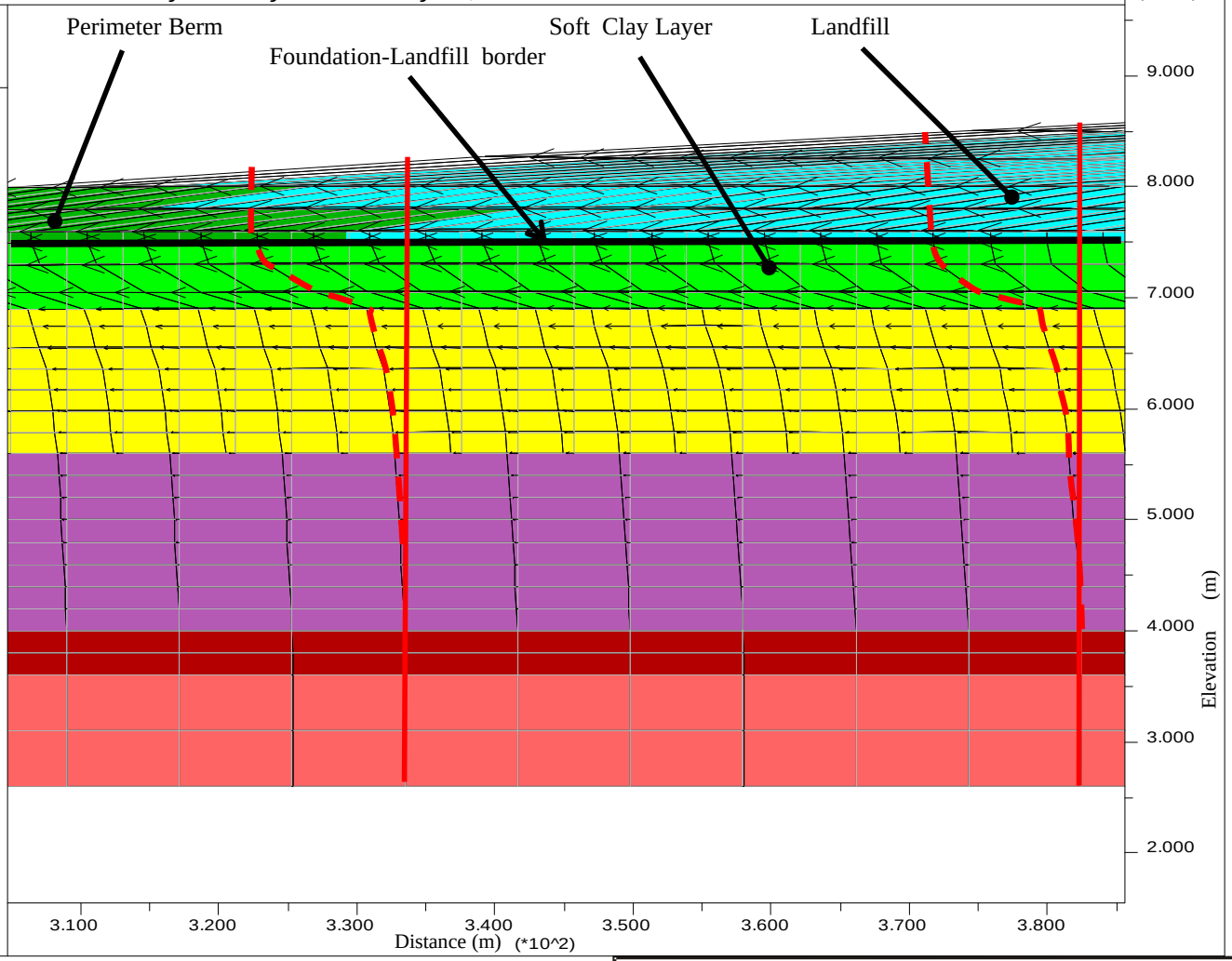
Exaggerated Grid Distortion

Magnification = 0.000E+00
Max Disp = 2.174E-01

Displacement vectors

max vector = 2.174E-01

0 5E -1



PROJECT

CAPITAL REGION RESOURCE RECOVERY CENTRE

TITLE

DISTORTED MESH COMPARED WITH ORIGINAL SHAPE (EARTHQUAKE No. 37)

Golder Associates

PROJECT No. 12-1125-0045			PHASE / TASK No. 4500	
DESIGN	M.S.K	31 MAR.14	SCALE NTS	REV.
CADD	--	--		
CHECK	PLE	AUG 2014	**FIGURE 5-8**	
REVIEW	PAS	AUG 2014		

REVISION DATE: BY: FILE:

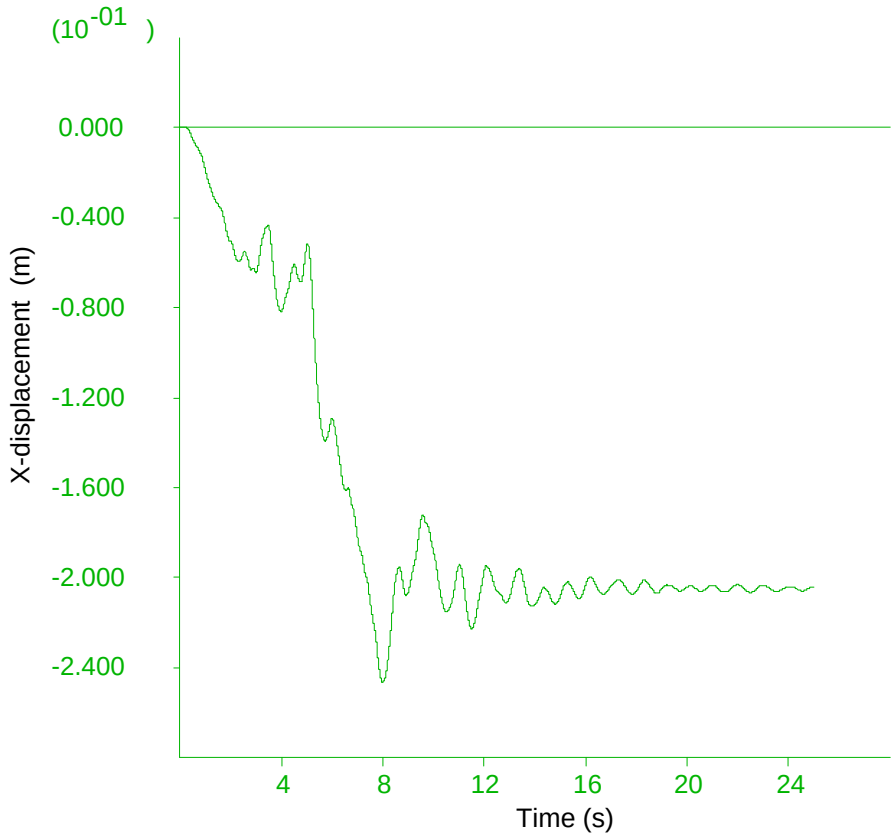
JOB TITLE : CRRRC MSW Landfill-Foundation system Dynamic Analysis, Ottawa

FLAC (Version 6.00)

LEGEND

31-Mar-14 11:32
step 2545773
Dynamic Time 2.5000E+01

HISTORY PLOT
Y-axis :
53 X displacement(23, 25)
X-axis :
2 Dynamic time



PROJECT

CAPITAL REGION RESOURCE RECOVERY CENTRE

TITLE

TIME HISTORY OF LATERAL DISPLACEMENT AT LANDFILL TOE (EARTHQUAKE NO. 37)

Golder Associates

PROJECT No. 12-1125-0045			PHASE / TASK No. 4500	
DESIGN	M.S.K	31 MAR.14	SCALE NTS	REV.
CADD	--	--	**FIGURE 5-9**	
CHECK	PLE	AUG 2014		
REVIEW	PAS	AUG 2014		