

Appendix E

Earthquake Loading and Analytical Liquefaction Calculations

Part of: The Use of Waste Rock Inclusions to Control the Effects
of Liquefaction in Tailings Impoundments

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1. SEISMIC LOADING PARAMETERS FOR THE REPRESENTATIVE MINE SITE

The section describes the estimation of seismic loading parameters for the representative mine site (Laronde Mine in Pressiac, Quebec). The seismic loading parameters were based on probabilistic seismic hazard calculations by Natural Resources Canada and seismic parameters provided in Adams & Halchuk (2003).

Seismic hazard calculations were obtained from the Natural Resources Canada website (earthquakescanada.ca, 2008) and are presented at the end of this appendix. The calculations provided peak horizontal ground accelerations (PGA) for recurrence intervals of 100, 475, 1,000, and 2,475 years with respect to site conditions characterized as class “C” soil, which is defined by a shear wave velocity between 360 and 760 m/s (Finn & Wightman, 2003; NBCC, 2005).

As recommended by Adams & Halchuk (2003), the PGA values were divided by a reference ground condition factor (r_{gc}) factor of 1.39 to estimate values for “hard” rock as defined by a shear wave velocity greater than 1,500 m/s (Finn & Wightman, 2003; NBCC, 2005). PGA values for recurrence intervals of 5,000 and 10,000 years were interpolated from the data assuming a log-log relationship between the recurrence interval and the PGA (Leahy, 2008). The estimated PGA values are shown on Table 1-1.

Table 1-1 – Estimated PGA values for selected recurrence intervals.

Recurrence Interval (Years)	Annual Probability (%)	PGA on Class “C” Soil (g)	PGA on “Hard Rock” (g)
100	1.00	0.020	0.014
475	0.21	0.047	0.034
1,000	0.10	0.067	0.048
2,475	0.0404	0.101	0.073
5,000	0.02	-	0.105
10,000	0.01	-	0.151

The moment magnitudes associated with the recurrence intervals were estimated using the “R” (robust) model of Adams & Halchuk (2003) and the formula provided in Basham et al. (1985) for eastern North America:

$$N = N_c \frac{(e^{-\beta m} - e^{-\beta m_x})}{(e^{-\beta m_c} - e^{-\beta m_x})} \quad (1-1)$$

where: N is the annual probability of occurrence of an earthquake of Nuttli magnitude (M_{bLg}), m or greater;

m is the Nuttli magnitude of the event of interest;

N_c is a source zone parameter related to the seismicity;

β is a parameter of the source zone that describes the relatively likelihood of large and small earthquakes (Kramer, 1996);

m_x is the Nuttli magnitude of the largest earthquake considered possible in source zone being considered; and

m_c is the lowest Nuttli magnitude of concern (e.g. the lowest magnitude at which liquefaction could occur).

The site lies in the Cochran (COC) source zone and within 50 km of the Iapetan Rift Background (IRB) source zone (Adams & Halchuk, 2003). The IRB source zone can produce earthquakes with moment magnitudes as great as 7 (Adams & Halchuck, 2003) and based on the attenuation relationship of Torro et al. (1994) as presented in Kramer (1996), the PGA at a distance of 50 km would be approximately 0.25 g, which would sufficient to cause liquefaction in loose, saturated, cohesionless soil. Therefore, the potential impact of an event in the IRB source zone at this distance was considered.

The R-model source zone parameters were obtained from Adams & Halchuk (2003) and are presented in Table 1-2.

For each source zone, the earthquake magnitudes associated with the various recurrence intervals were estimated using Equation 1-1 and the source zone parameters in Table 1-2. The estimated

magnitude for each recurrence interval was assumed to be equal to the maximum from the source zones (COC or IRB).

Table 1-2 – R-model source zone parameters (from Adams & Halchuk, 2003).

Parameter	Source Zone	
	COC	IRB
Beta	2.00	2.00
M _C	5.00	5.00
M _X	7.50	7.00
N _C	0.0034	0.0281

The magnitudes were converted from Nuttli to moment scale using the following formulas provided in Adams & Halchuk (2003) and attributed to Atkinson (1993):

For M_{bLg} less than or equal to 5.5:

$$M_w = 0.98 \cdot M_{bLg} - 0.39 \quad (1-2)$$

For M_{bLg} greater than 5.5:

$$M_w = 2.715 - 0.277 \cdot M_{bLg} + 0.127 \cdot M_{bLg}^2 \quad (1-3)$$

The moment magnitudes were rounded to the nearest quarter to reflect the level of uncertainty in their estimation (Kramer, 1996; Adams & Halchuk, 2003). The results are in Table 1-3.

Table 1-3 – Estimated magnitude for selected recurrence intervals.

Recurrence Interval (years)	Annual Probability (%)	Magnitude (M_{blg}) by source zone		Estimated Magnitude	
		COC	IRB	M_{blg}	M_w
100	1.00	4.46	5.50	5.50	5
475	0.21	5.24	6.19	6.19	5.75
1,000	0.10	5.60	6.47	6.47	6.25
2,475	0.0404	6.04	6.71	6.71	6.5
5,000	0.02	6.37	6.84	6.84	6.75
10,000	0.01	6.66	6.91	6.91	6.75

2. EARTHQUAKE GROUND MOTIONS FOR COMPARATIVE EVALUATION

One dimensional earthquake ground motions (horizontal accelerations) were required for comparative dynamic evaluation of the representative tailings impoundment and of the conceptual tailings impoundment, both with and without waste rock inclusions. This section describes the development of those ground motions (evaluation earthquakes).

As noted in Chapter 3, the general parameters for the ground motions were that they should be equivalent to events of moment magnitudes 6.5, 6.75, 7, 7.25, and 7.5 with fault-zone to site distances of 30 km and have characteristics typical of events in eastern North America.

2.1 Parameters of the Evaluation Earthquakes

The parameters of the evaluation earthquakes were estimated using published attenuation relationships.

2.1.1 Peak Ground Acceleration

Using the method of Toro et al. (1994) as given in Kramer (1996) for the PGA on rock in eastern North America:

$$\ln PGA(g) = 2.20 + 0.81(M_w - 6) - 1.27 \ln R_m + 0.11 \max\left(\ln \frac{R_m}{100}, 0\right) - 0.0021 R_m$$

with:

$$R_m = \sqrt{R^2 + 9.3^2} \quad (2-1)$$

where: R is the closest horizontal distance to the fault rupture in km.

2.1.2 Arias Intensity, I_h

As noted in Chapter 2, the Arias Intensity, I_h , is a measure of earthquake shaking intensity and represents the total energy per unit weight absorbed by an idealized set of oscillators during earthquake shaking (Kayen & Mitchell, 1997).

From the method of Wilson (1993) as provided in Kramer (1996):

$$\log I_h = M_w - 2 \log R - kR - 3.990 + 0.365(1 - P) \quad (2-2)$$

where: I_h is the Arias intensity in two orthogonal horizontal directions, $I_h = I_x + I_y$.

$$R = \sqrt{D^2 + h^2} \quad (2-3)$$

D is the closest horizontal distance to the fault rupture in km;

h is a correction factor with a default value of 7.5 km;

k is a coefficient of an elastic absorption with a default value of zero (0 used);
and

P is the exceedance probability (0.5 used)

The Arias intensity is considered to be a more representative means of quantifying earthquake loading than the PGA, which is an isolated peak value and may not be representative of earthquake shaking (Kramer, 1996; Kayen & Mitchell, 1997).

2.1.3 Duration of Significant Shaking

The duration of significant shaking, where significant shaking is defined as acceleration amplitudes exceeding 0.05 g, varies with earthquake magnitude and epicentral distance (Kramer, 1996). Using the relationship provided in Kramer (1996), the duration of significant shaking, t_{ss} , for the evaluation earthquakes varies from 8 seconds for E1 ($M_w=6.5$) to 16 seconds of E5 ($M_w=7.5$).

2.1.4 Summary

The estimated values are presented in Table 2-1.

Table 2-1 – Estimated Evaluation Earthquake Parameters

Evaluation Earthquake	Magnitude, M_w	PGA (g)	I_h (m/s)	t_{ss} (s)
E1	6.5	0.159	0.515	8
E2	6.75	0.195	0.916	10
E3	7	0.238	1.629	11
E4	7.25	0.292	2.897	14
E5	7.5	0.357	5.151	16

2.2 Development of the Evaluation Earthquake Ground Motions

The evaluation earthquake ground motions were developed by modifying an existing ground motion record. The November 25, 1988 Saguenay (Quebec) earthquake ($M_w=5.9$) produced the only strong ground motion records available from eastern North America, thus it was decided to use one of the records from that event.

Ground motion data from the 1988 Saguenay event for 11 sites were obtained from NRC (2003). The data included two orthogonal horizontal components (transverse, T, and radial, R) and one vertical component, Z, for each site. Initial screening to remove incomplete records and select the most suitable horizontal record from each site left the 8 records listed in Table 2-2. Also shown on the table are the Arias intensities, I_x , of the ground motion based on the following formula from Kayen & Mitchell (1997):

$$I_h = I_x + I_y = \frac{\pi}{2g} \int a_x^2(t) dt + \frac{\pi}{2g} \int a_y^2(t) dt \quad (2-4)$$

where: I_h is the Arias intensity in two horizontal directions;

I_x and I_y are the Arias intensity in either horizontal direction;

g is gravity (9.81 m/s^2);

a_x and a_y are the horizontal accelerations in the x and y directions; and

t is time.

The ground motion records were further evaluated based on their PGA values, Arias intensities, and response spectrum diagrams (see Munro & Weichert, 1989) and record 16T was selected to used as a basis for the evaluation ground motions for the following reasons:

It's response spectrum was typical among those evaluated;

- a) Due to its relatively high Arias intensity, it would required less factoring to match the Arias intensities of the evaluation ground motions; and
- b) Its epicentral distance was the closest to the fault-rupture to site distances assumed for the evaluation earthquakes.

- c) The S16T ground motion record is presented on Figure 2-1. This record has a PGA value of 1.287 m/s^2 (0.131 g), a duration of 25 seconds (in 0.005 second intervals), an Arias intensity of 0.1767 m/s, and the duration of significant shaking (0.05 g or greater) is approximately 10 seconds.

Table 2-2 – Selected ground motion records of the 1988 Saguenay earthquake.

Record	Epicentral Distance (km)	PGA (g)	Duration (s)	Nbr of Data Points	I_x (m/s)
1R	113.8	0.121	30.00	6001	0.0940
2T	149.3	0.051	15.00	3001	0.0178
5R	109.2	0.027	20.00	4001	0.0111
8R	93.0	0.124	19.65	3931	0.0735
9T	122.7	0.056	12.50	2501	0.0207
10T	114.4	0.057	12.50	2501	0.0175
16T	43.2	0.131	25.00	5001	0.1767
17R	63.6	0.156	20.00	4001	0.1366

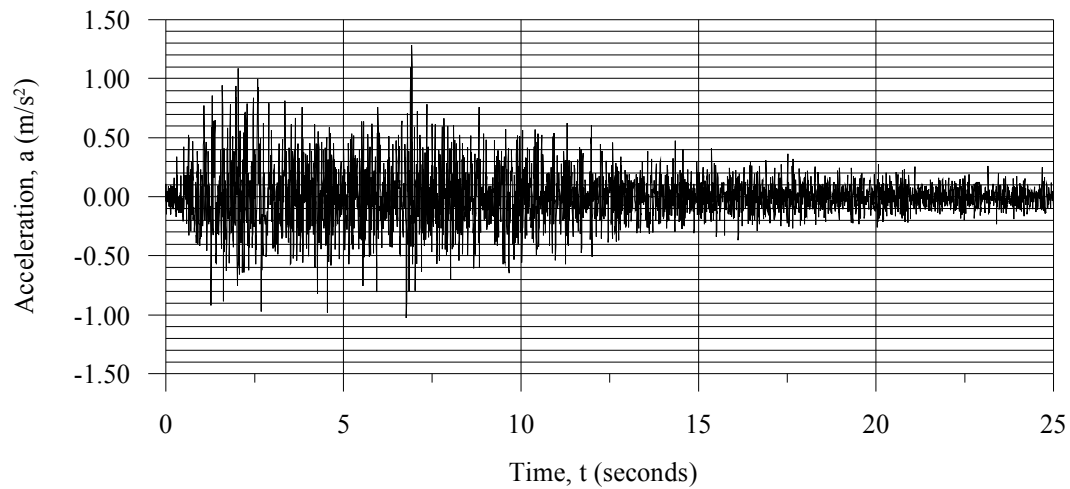


Figure 2-1 – Ground motion record S16T of the 1988 Saguenay earthquake (NRC, 2003).

Referring back to Table 2-1, the estimated parameters for the evaluation earthquakes, the most important parameters are the Arias intensity, I_x , and the duration of significant shaking, t_{ss} . As noted previously, the PGA value may not accurately represent earthquake intensity.

The Arias intensities for the evaluation earthquakes presented in Table 2-1 are for two-dimensional shaking. To estimate equivalent intensities for one-dimensional shaking of a two-dimensional model, the intensities in Table 2-1 were halved and then increased by 20% to account for the second horizontal dimension of shaking in actual earthquakes. The 20% increase was based on experimental findings by Pyke et al. (1974) and Boulanger and Seed (1995) that indicated that the second horizontal dimension in dynamic laboratory testing was equivalent to a 20% increase in the dynamic loading.

To create the evaluation ground motions, the accelerations and time increments of record S16T were factored such that the Arias intensities of the evaluation ground motions were 0.6 (0.5·1.2) times those in Table 2-1 and the duration of significant shaking was similar to that of Table 2-1. The factors used to create the evaluation ground motions and the resulting parameters of the evaluation ground motion are given in Table 2-3.

Table 2-3 – Evaluation earthquake ground motion parameters

Evaluation Earthquake	Magnitude, M_w	Acceleration Factor	Time Factor	I_x (m/s)	PGA (g)	t_{ss} (s)
E1	6.5	1.7493	1.0	0.3091	0.229	10
E2	6.75	3.1109	1.0	0.5497	0.408	10
E3	7	5.0286	1.1	0.9774	0.659	11
E4	7.25	6.5576	1.5	1.7381	0.859	15
E5	7.5	10.9324	1.6	3.0908	1.432	16

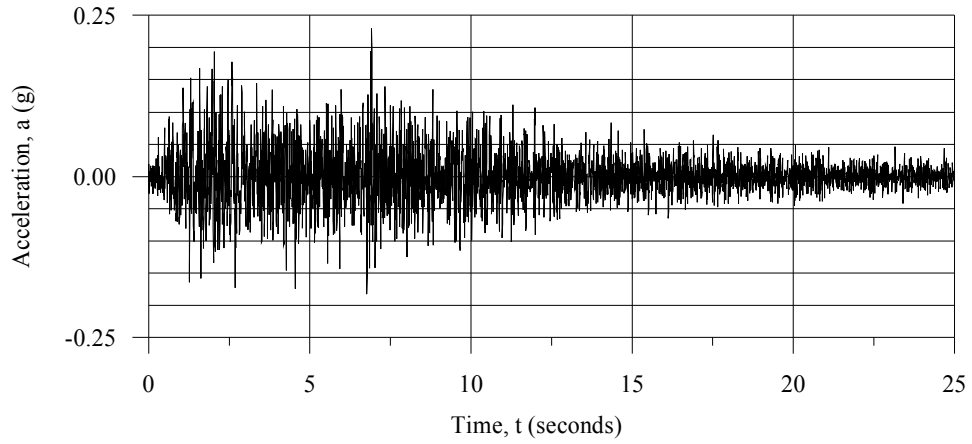
In comparing Tables 2-1 and 2-3, the calculated Arias intensities of the evaluation earthquakes (Table 2-3) are a factor of 0.6 of those predicted (Table 2-1), the durations of significant shaking are similar, and the calculated PGA values of the evaluation earthquakes are significantly higher than those predicted. However, given that the Arias intensity is a more representative measure of earthquake intensity, the PGA values are deemed to be acceptable. The resulting evaluation earthquake ground motions are presented on Figure 2-2 (E1, E2 and E3) and Figure 2-3 (E4 and E5).

The implications of the manipulation of the S16T ground motion were evaluated by comparing the acceleration response spectrums and Fourier amplitudes of the evaluation earthquake ground motions with those of the original ground motion. An acceleration response spectrum represents the maximum response of a single degree of freedom structure to a ground motion as a function of the natural frequency of the structure and damping ratio (Kramer, 1996). A Fourier amplitude diagram shows how the amplitude of the ground motion is distributed with respect to period or frequency (Kramer, 1996).

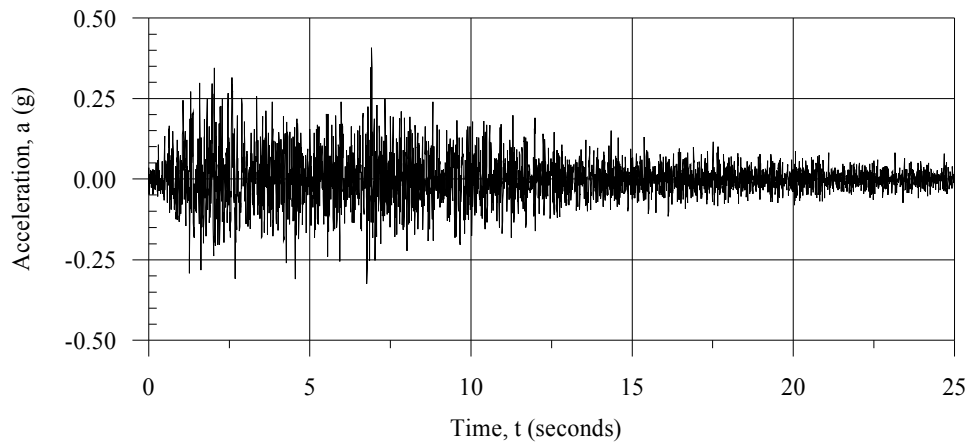
Acceleration response spectrum diagrams for record S16T and Earthquakes E1, E3 and E5 are presented on Figure 2-4. The diagrams were developed using NERA (Bardet & Tobita, 2001) and assuming bedrock with a shear modulus of 20,000 MPa and 5% critical damping. Fourier amplitude diagrams for record S16T and Earthquakes E1, E3 and E5 are presented on Figure 2-5 and were also developed using NERA. The acceleration response spectrum and Fourier amplitude diagrams for Earthquakes E2 and E4 were also developed and are compatible with the results for Earthquakes E1, E3 and E5. However, they have not been included in this appendix.

The acceleration spectrum responses of record S16T and the evaluation earthquakes are very similar in shape (Figure 2-4), except for a slight increase in the period during which the maximum spectral acceleration occurs, from 0.02 s for record S16T to 0.03 s for Earthquake E5, and the magnitude of the spectral acceleration, which increased with increasing PGA value.

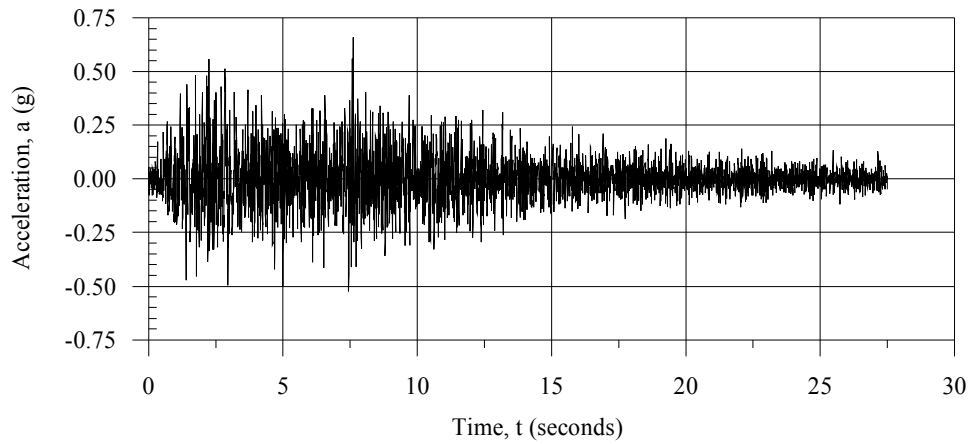
As shown on the Fourier acceleration diagrams (Figure 2-5), aside from the magnitude of the Fourier accelerations, the distribution of the amplitude with respect to period was not significantly affected by the manipulation of the ground motion. The diagrams for Earthquakes E3 and E5 were cut short due to limitations in the NERA program which was developed on the west coast where ground motions tend to have much lower frequency contents.



(a) Earthquake E1, $M_w=6.5$, PGA=0.229 g

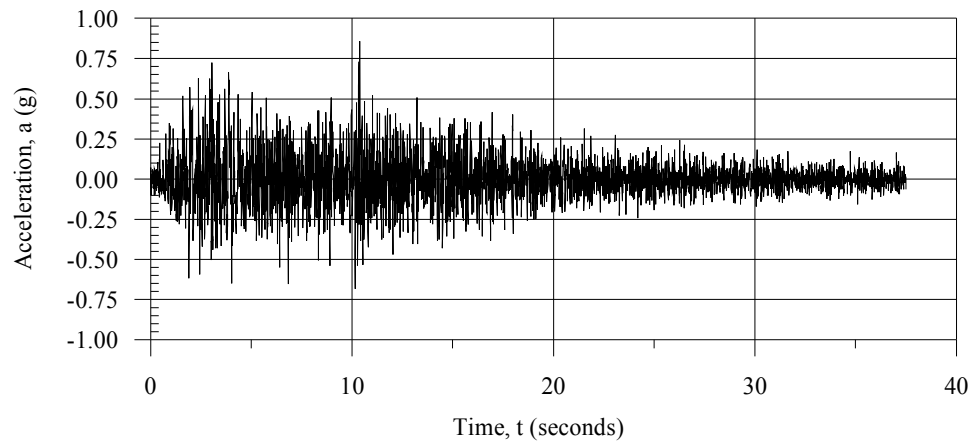


(b) Earthquake E2, $M_w=6.75$, PGA=0.408 g

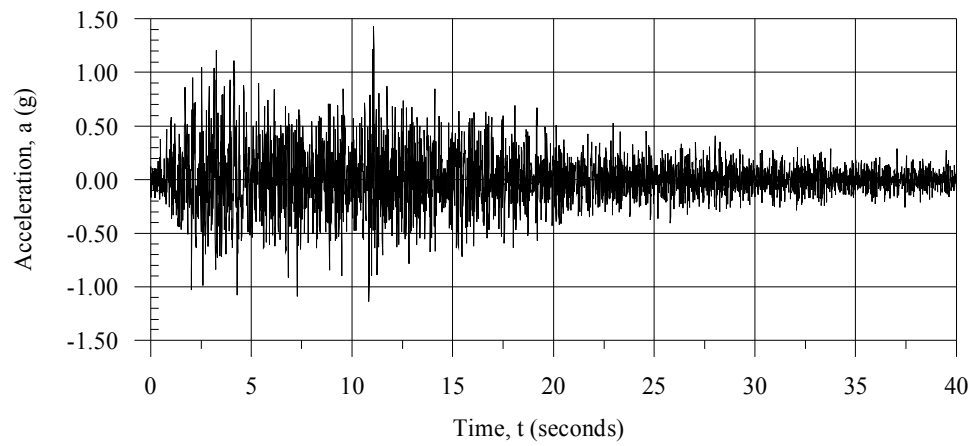


(c) Earthquake E3, $M_w=7$, PGA=0.659g

Figure 2-2 – Evaluation earthquake ground motions (E1, E2 and E3).



(a) Earthquake E4, $M_w=7.25$, $PGA=0.859g$



(b) Earthquake E5, $M_w=7.5$, $PGA=1.432 g$

Figure 2-3 – Evaluation earthquake ground motions (E4 and E5).

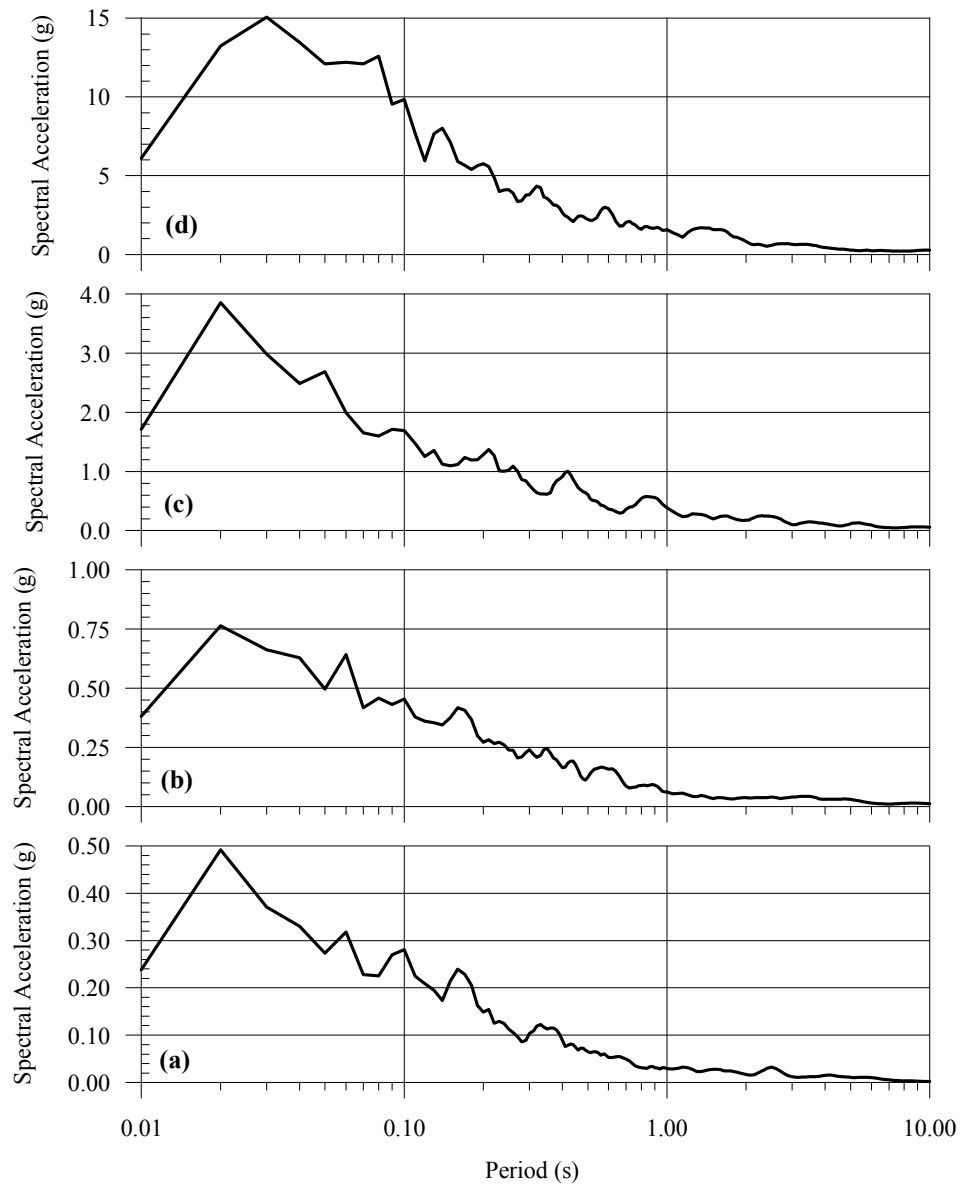


Figure 2-4 – Spectral acceleration diagrams of record S16T (a) and Evaluation earthquakes E1 (b), E3 (c) and E5 (d).

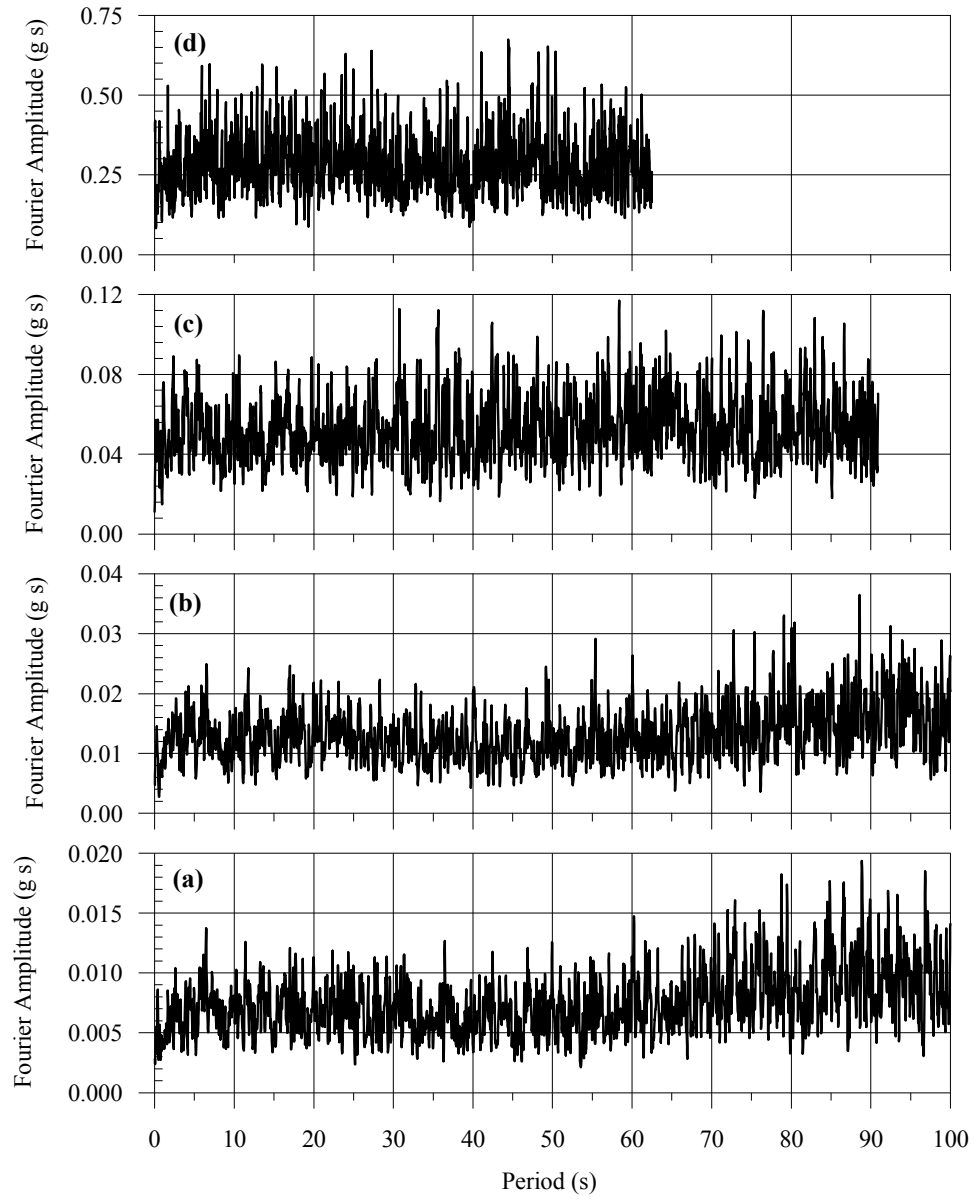


Figure 2-5 – Fourier amplitude diagrams of record S16T (a) and Evaluation earthquakes E1 (b), E3 (c) and E5 (d).

3. DETAILS OF THE LIQUEFACTION ANALYSIS USING CONE PENETRATION TEST RESULTS

Section 3.9.1 of the thesis presents liquefaction analyses of the tailings based on the results of cone penetration testing and the assumption of “level ground” conditions. This section of the appendix presents details of the analyses, specifically the site response analysis and the calculation of the cyclic stress ratios and cyclic resistance ratios.

3.1 Site Response Analyses

The response of the impoundment was modeled using the NERA (Nonlinear Earthquake Response Analyses) computer program by Bardet & Tobita (2001). NERA uses the finite difference method and a nonlinear, hysteretic material model to simulate the two-dimensional response of a level-ground site to earthquake ground motion. All of the input and output of the program is done using spreadsheets. NERA was validated by Bardet & Tobita (2001) through comparative analysis with EERA (Bardet et al., 2000) and SHAKE (Schnabel et al., 1972), which are equivalent linear site response analysis software programs.

The input required for operation of the program includes:

- An earthquake data file and a factor for scaling the accelerations to fit the peak horizontal ground acceleration desired;
- A description of the soil profile in tabular format, including soil type, layer thickness, total unit weight, and shear modulus (or shear wave velocity) of each soil layer, and the location of the groundwater table, if present;
- Shear modulus reduction (damping) curves for each soil type; and
- Program control flags to control the frequency cut-off, set the type of ground motion (outcrop or internal), output parameters, etc.

The program produces the following output in tabular and graphic format for any or all of the soil layers:

- Acceleration, velocity, displacement, stress-strain and dissipated energy time-histories;

- Fourier spectra; and
- Acceleration, velocity and displacement spectra for given critical damping ratios.

NERA requires the subsurface to be divided into horizontal layers, including bedrock. The tailings impoundment was assumed to consist of fifteen 1-m-thick layers of tailings, a 2-m-thick layer of dense glacial till and then bedrock. The groundwater level was assumed to be at the surface.

The properties used for the tailings were derived from cone penetration testing and supplemented with the results of laboratory testing. The cone tip resistance of CPT 02-02 is typical of the CPT results from the impoundment and was used to characterize the tailings for the site response analysis.

The unit weight of each layer of tailings was estimated using Equation 3-1 which relates the void ratio to the effective vertical consolidation stress of the tailings and the measured average specific gravity of 3.88. Given that significant zones within the tailings are under-consolidated with respect to existing loads, the resulting estimated unit weight of the layers are probably slightly greater than the actual values.

The shear modulus of each layer of tailings was estimated from the profile of the tip resistance of CPT 02-02 using Equation 3-11. Figure 3-1 is a profile of the estimated shear modulus of the tailings at the location of CPT 02-02.

The shear modulus reduction curve of Seed et al. (1984) at an effective mean stress of 100 kPa was used for the tailings. This curve is shown on **Error! Reference source not found.**

The properties of the glacial till were estimated from typical values provided in Milligan (1976) and Bowles (1996). The shear modulus reduction curve of Seed et al. (1994) for an effective mean stress of 150 kPa was used for the glacial till. This curve is shown on **Error! Reference source not found.**

Goodman (1989) was used to estimate the properties of the bedrock based on the properties of andesite or similar rock.

The material properties used in the site response analyses are shown on Table 3-1.

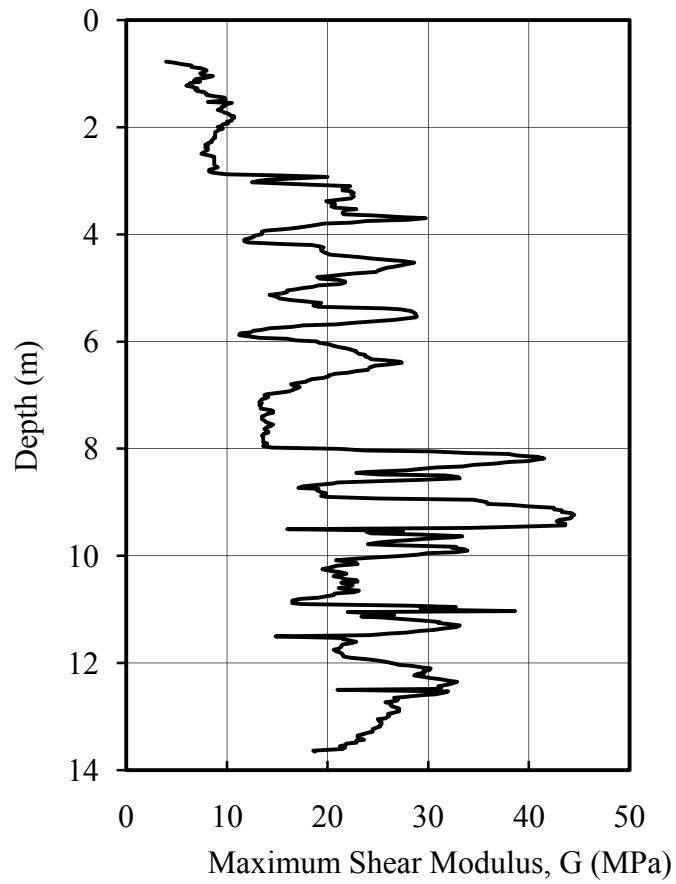


Figure 3-1 – Profile of estimated maximum shear modulus based on the results of CPT No. 02-02.

Table 3-1 - Material properties used in the site response analyses based on CPT data.

Layer No.	Material Type	Depth to Center	Dry Unit Weight	Shear Modulus
		(m)	(kN/m ³)	(MPa)
1	Tailings	0.5	22.0	6.06
2		1.5	22.6	8.69
3		2.5	22.8	9.06
4		3.5	23.0	20.38
5		4.5	23.1	19.27
6		5.5	23.2	21.19
7		6.5	23.3	13.85
8		7.5	23.4	27.25
9		8.5	23.4	35.43
10		9.5	23.5	21.73
11		10.5	23.5	25.43
12		11.5	23.6	28.72
13		12.5	23.6	24.34
14		13.5	23.7	24.34
15		14.5	23.7	24.34
16	Glacial till	16.0	19.60	360
17	Bedrock	-	21.60	7,700

Site response analyses were completed using NERA (Bardet and Tobita, 2001), the values in Table 3-1 and earthquakes E1, E3 and E5. The results of the site response analyses are summarized on Table 3-2. The amplification factors, F_a were calculated using the following formula:

$$F_a = \frac{PGA_x}{PGA_{BR}} \quad (3-1)$$

where: PGA_x is the peak (horizontal) ground acceleration at some reference point (e.g. the ground surface); and

PGA_{BR} is the peak (horizontal) ground acceleration on the top of bedrock.

The amplification factors vary from 0.109 and 0.424 and decrease with the intensity of the applied earthquake load. Application of the ground motions (horizontal accelerations) resulted in severe de-amplification (damping) of the accelerations as the ground motion traveled from the bedrock to the surface. The reason for the severe damping of the accelerations was the relatively low shear modulus of the tailings and the drastic reduction of the shear modulus with shear strain (as discussed below).

Table 3-2 – Summary of site response analyses based on CPT 02-02 data with respect to Earthquakes E1, E3 and E5.

Earthquake	Moment Magnitude, M_w	PGA on Rock (g)	PGA on Surface of Tailings (g)	Amplification Factor, F_a
E1	6.5	0.229	0.097	0.424
E3	7	0.659	0.127	0.193
E5	7.5	1.432	0.156	0.109

Figure 3-2 contains profiles of the maximum acceleration, a_{max} , and maximum shear strain, γ_{max} , calculated in the site response analysis of the CPT 02-02 data for earthquake E1. As shown on the figure, there was moderate amplification of the maximum acceleration within the glacial till (depth, z of 15 and 17 m) and high damping of the maximum acceleration within the tailings ($z = 0$ to 15 m). Within the tailings, the average maximum acceleration was 0.11 g, less than half that on the top of bedrock (0.229 g). The maximum shear strains within the tailings varied from 0.03 to 0.19%. The shear modulus reduction curve applied to the tailings, shown on **Error! Reference source not found.**, indicates that the shear modulus of the tailings would have been reduced by 55 to 84% due to that level of shear strain.

Profiles for a_{max} and γ_{max} for the CPT 02-02 site response for earthquake E3 ($M_w=7$) are presented on Figure 3-3. The value of a_{max} increased from 0.659 g at the top of bedrock to 0.84 g within the glacial till. However, within the lower 2 m of the tailings the value of a_{max} was significantly reduced ($a_{max}=0.24$ g at $z = 13$ m). As shown on Figure 3-3, the ground motion was highly dampened within the tailings. The maximum shear strains within the tailings varied

between 0.05 and 0.55%. The shear modulus of the tailing would have been reduced by 60 to 90% as a result of this level of shear strain.

For the site response analysis of CPT 02-02 using Earthquake E5 ($M_w=7.5$), the maximum acceleration and maximum shear strain profiles are shown on Figure 3-4. In this case, the value of a_{max} was 1.432 g on top of bedrock and 1.9 g within the glacial till, indicating moderate amplification. The maximum acceleration was reduced significantly within the lower 2 m of the tailings ($a_{max}=0.65$ g at $z = 14$ m) and continued to decrease as the ground motion rose through the soil profile (refer to Figure 3-4). The maximum shear strains within the tailings reflected the influence of the strong ground motion as they varied from 0.25 to 1.8%. The maximum shear strains within the tailings would have resulted in a 90 to 95% reduction in the shear modulus.

The shear modulus relates shear stress to shear strain and also indicates the ability of a soil layer to transmit shear stresses to overlying layers. A soil layer with a low shear modulus, or a reduced shear modulus due to shear strain, cannot transmit significant shear stresses to overlying layers. As shear stresses are the means by which earthquake ground motions are transmitted through soil, a soil layer of low shear modulus will undergo significant shear strain leading to a reduction in the shear modulus and increased damping.

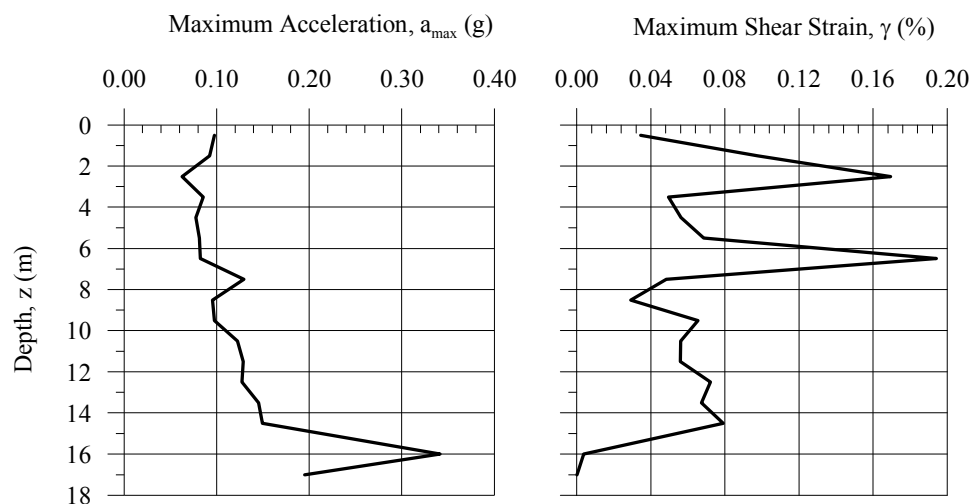


Figure 3-2 – Profiles of the maximum acceleration and maximum shear strain from site response analyses of CPT 02-02 data using earthquake E1 ($M_w=6.5$).

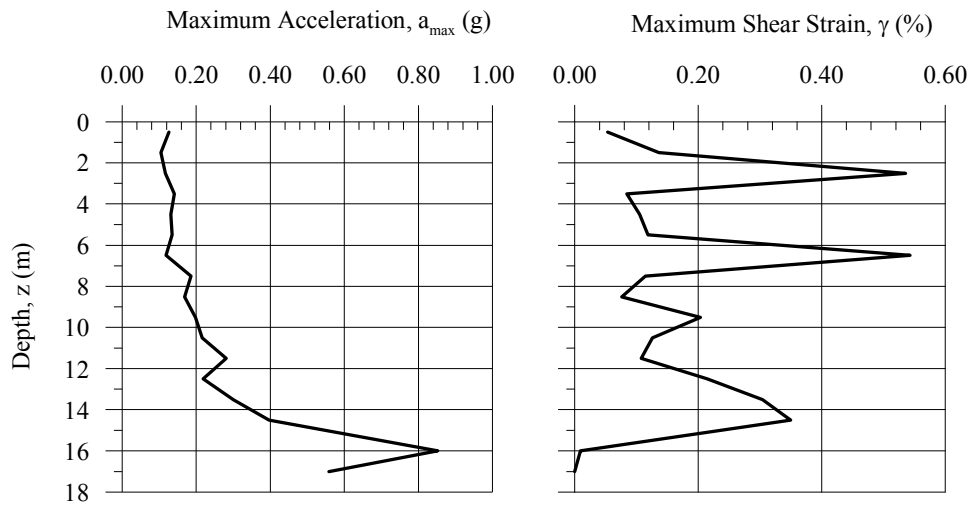


Figure 3-3 – Profiles of the maximum acceleration and maximum shear strain from site response analyses of CPT 02-02 data using earthquake E3 ($M_w=7$).

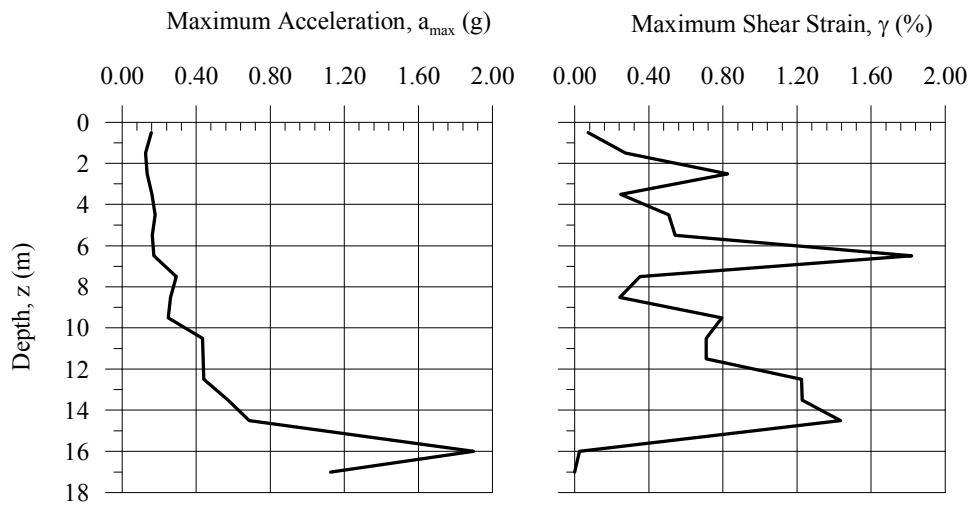


Figure 3-4 – Profiles of the maximum acceleration and maximum shear strain from site response analyses of CPT 02-02 data using earthquake E5 ($M_w=7.5$).

3.2 Calculation of the Cyclic Stress and Cyclic Resistance Ratios

For the three earthquake loads considered in the liquefaction evaluation (E1, E3 and E5), the cyclic stress ratio, CSR, induced in the tailings was calculated using Equation 2-5 with stress reduction coefficient, r_d , estimated from the site response analyses as described below.

For each earthquake, the stress reduction coefficient was estimated for each soil layer by dividing the maximum acceleration of each layer by the maximum acceleration of the input ground motion ($r_d = a_{\max}/PGA$). These values are plotted on Figures 3-34, 3-35 and 3-36 for Earthquakes E1, E3 and E5, respectively. Equations describing the variation of r_d with depth, z , are shown on the figures and given below in the same sequence.

$$\text{For Earthquake E1:} \quad r_d = \frac{z+6.8796}{31.373} \quad (3-2)$$

$$\text{For Earthquake E3:} \quad r_d = e^{\left(\frac{z-21.346}{10.455}\right)} \quad (3-3)$$

$$\text{For Earthquake E5:} \quad r_d = e^{\left(\frac{z-20.62}{7.627}\right)} \quad (3-4)$$

where z is the depth below the surface in meters.

This method of estimating the stress reduction coefficient is based on that used by Seed & Idriss (1982). The stress reduction coefficient recommended by Seed & Idriss (1982) is a generic parameter based on evaluation of many soil profiles and assumes that the soil properties due not change appreciably until liquefaction is imminent. Site-specific response analyses are preferred over use of the generic stress reduction coefficient (Seed & Idriss, 1982; Kramer, 1996).

The stress reduction coefficient recommended by Seed and Idriss (1982) was based on analyses using the equivalent linear method of damping. In that method an average shear strain is estimated for each soil layer and the shear modulus is reduced to a level associated with the average shear strain for the duration of shaking. This method generally produces amplification of the ground motion, while non-linear methods, which are considered to be more accurate, produced amplification or damping, depending on the soil conditions and shape of the earthquake ground motion (Itasca, 2005).

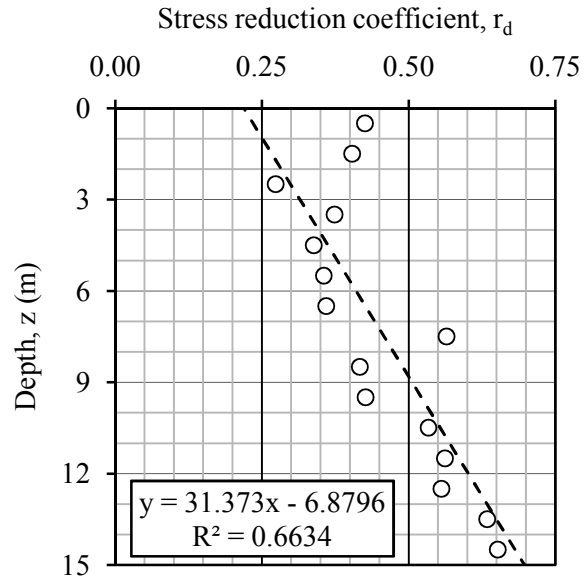


Figure 3-5 – Estimation of the stress reduction coefficient, r_d , based on site response analysis of CPT data for Earthquake E1 ($M_w=6.5$).

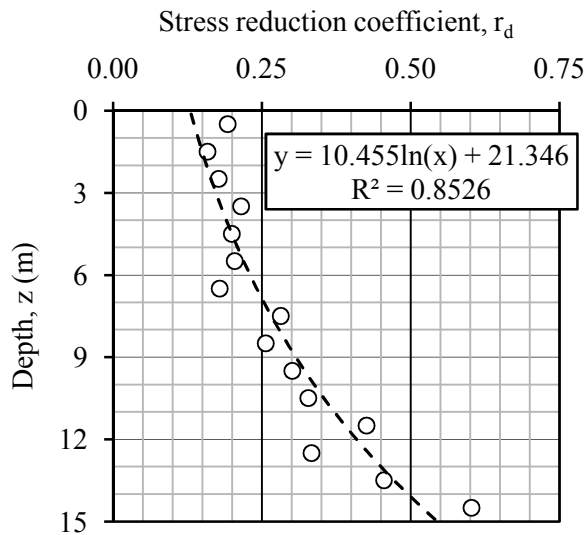


Figure 3-6 – Estimation of the stress reduction coefficient, r_d , based on site response analysis of CPT data for Earthquake E3 ($M_w=7$).

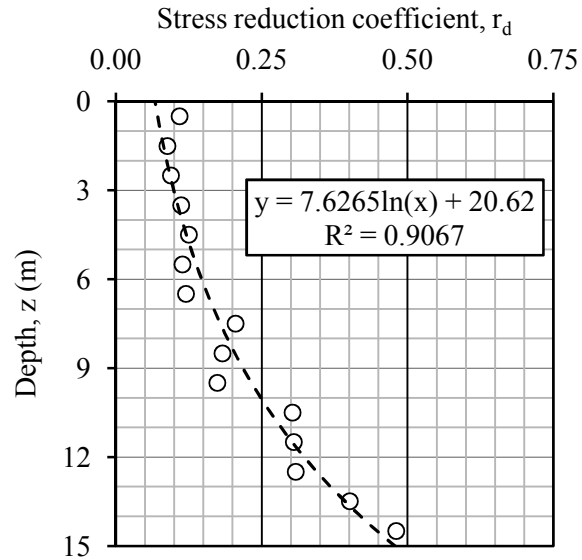


Figure 3-7 – Estimation of the stress reduction coefficient, r_d , based on site response analysis of CPT data for Earthquake E5 ($M_w=7.5$).

The cyclic resistance ratio, CRR, of the tailings was estimated using Figure 2-21 and the results of cone penetration testing normalized as per Youd et al. (2001). The results were not corrected for silt content (soil type) as recommended by Youd et al. (2001) due to the fact that the tip resistances of apparently consolidated zones within the tailings correlate well with that expected for sands as opposed to silts (refer to Section 3.7.3.5). Also the finding of Ulrich and Hughes (1994) tends to indicate the tip resistances of tailings does not require correction for the fines content for conversion to standard penetration test values. The CRR of the tailings was corrected for the earthquake magnitude using the values given on **Error! Reference source not found.** that were based on the results of CDSS testing.

The factors of safety with respect to liquefaction were calculated using Equation 2-12. However, since the consolidation stress had no effect on the liquefaction resistance in the CDSS testing and typically has no effect for soils of high silt content (refer to Section 2.2.4), the overburden stress correction factor was not applied. The static shear stress factor was taken as 1.0 due to the absence of static shear stresses on horizontal planes (level ground conditions).

4. DETAILS OF THE LIQUEFACTION EVALUATION USING CDSS TEST RESULTS

Section 3.9.2 of the thesis presents liquefaction analyses of the tailings based on the results of cyclic direct simple shear testing and the assumption of “level ground” conditions. This section of the appendix presents details of the analyses, specifically the site response analysis and the calculation of the cyclic stress ratios, and cyclic resistance ratios.

4.1 Site Response Analyses

Dynamic responses of the consolidated tailings deposit to the earthquake loadings were analyzed using the NERA computer program (Bardet & Tobita, 2001). The same subsurface profile, material properties and shear modulus reduction curves used in the analyses of the CPT data were used in the analyses of the CDSS testing data, except that the shear modulus of the tailings were based on the CDSS testing (refer to Section 3.7.2.2). The shear modulus of the tailings was estimated to average 50 MPa at all levels of effective consolidation stress anticipated in the impoundment. However, to account for the variability typical of even normally consolidated tailings deposits, the shear modulus of each layer was randomly varied by plus or minus 10% based on the variability observed in the CDSS test results (see **Error! Reference source not found.**). The material properties used in the analyses are presented on Table 4-1.

The site response analyses produced the amplification factors, F_a , of 0.694, 0.420 and 0.233 for Earthquakes E1, E3 and E5, respectively (refer to Table 4-2). All of the ground motions were damped as they transited the tailings with the degree of damping increasing with the intensity of the loading. The amplification factors produced in the site response analyses using CDSS test results were roughly double those produced in the analysis using CPT results (see Table 3-2), indicating significantly less damping in the normally consolidated tailings compared to the under-consolidated tailings.

Table 4-1 – Material properties used in the site response analyses
based on CDSS test results.

Layer No.	Material Type	Depth to Center	Dry Unit Weight	Shear Modulus
		(m)	(kN/m ³)	(MPa)
1	Tailings	0.5	22.0	51.39
2		1.5	22.6	47.49
3		2.5	22.8	49.77
4		3.5	23.0	48.46
5		4.5	23.1	45.56
6		5.5	23.2	51.46
7		6.5	23.3	52.89
8		7.5	23.4	46.26
9		8.5	23.4	50.00
10		9.5	23.5	47.40
11		10.5	23.5	45.59
12		11.5	23.6	52.04
13		12.5	23.6	48.59
14		13.5	23.7	51.73
15		14.5	23.7	46.63
16	Glacial till	15.5	19.60	360
17	Bedrock	-	21.60	7,700

Table 4-2 – Results of site response analyses based on CDSS test results.

Earthquake	Moment Magnitude, M_w	PGA on Rock (g)	PGA on Surface of Tailings (g)	Amplification Factor, F_a
E1	6.5	0.229	0.159	0.694
E3	7	0.659	0.277	0.420
E5	7.5	1.432	0.334	0.233

The maximum (horizontal) accelerations, a_{max} , and maximum shear strains, γ_{max} , estimated by the site response analyses varied significantly with the earthquake loading. Figure 4-1 presents

profiles of the maximum (horizontal) acceleration, $a_{\max}$, and maximum shear strain, $\gamma_{\max}$, from the site response analysis based on the CDSS test results and Earthquake E1. The profile of $a_{\max}$ indicates slight amplification of the ground motion within the glacial till ($z = 15$ to 17 m), followed by moderately low damping of the ground motion within the tailings ($z = 0$ to 15 m). The value of $a_{\max}$ varies roughly from about 0.25 g at $z = 16.5$ m to about 0.15 g near the ground surface ($z = 0.5$ m). The corresponding maximum shear strains, $\gamma_{\max}$, in the tailings vary from about 0.11 to 0.08% within the lower 9 m of the tailings and decrease from about 0.08% at a depth, z , of 6 m to near zero at the surface ($z = 0$). Referring to **Error! Reference source not found.**, this level of shear strain resulted in a reduction in the shear modulus of the tailings by as much as 50 to 80% .

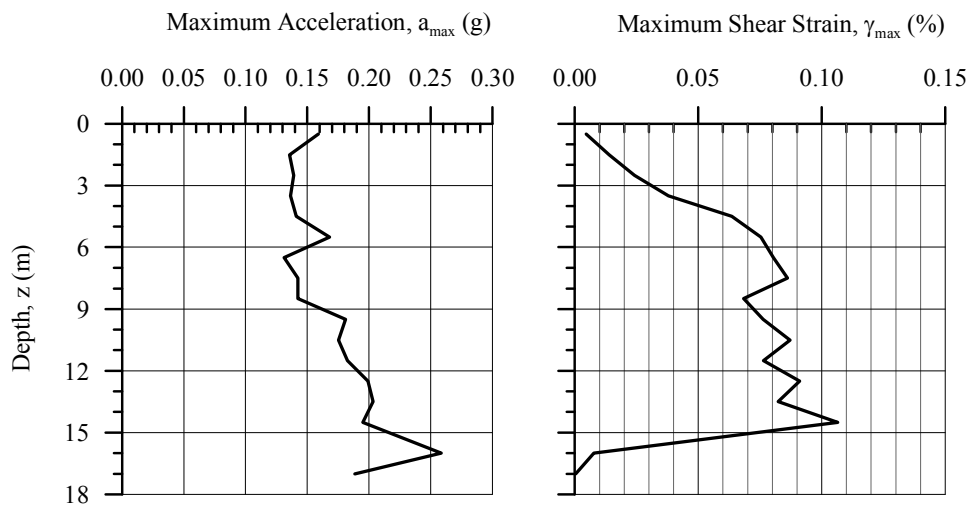


Figure 4-1 – Profiles of the maximum acceleration and maximum shear strain from the site response analysis using the CDSS testing data for Earthquake E1 ($M_w=6.5$).

Figure 4-2 shows the maximum acceleration, $a_{\max}$, and maximum shear strain, $\gamma_{\max}$, profiles from site response analysis of the CDSS data for Earthquake E3. The value of $a_{\max}$ decreases relatively linearly from about 0.55 g at the bottom of the tailings ($z = 15$ m) to about 0.30 g at the ground surface ($z = 0$), indicating moderate damping throughout the depth of the tailings. The maximum shear strains, $\gamma_{\max}$, varied irregularly from 0.3% at a depth, z , of 15 m to about 0 at the ground

surface, with a peak value of about 0.38 occurring at a depth of about 13 m. These levels of shear strain are associated with reductions in the shear modulus as great as 90% of the initial value (see **Error! Reference source not found.**).

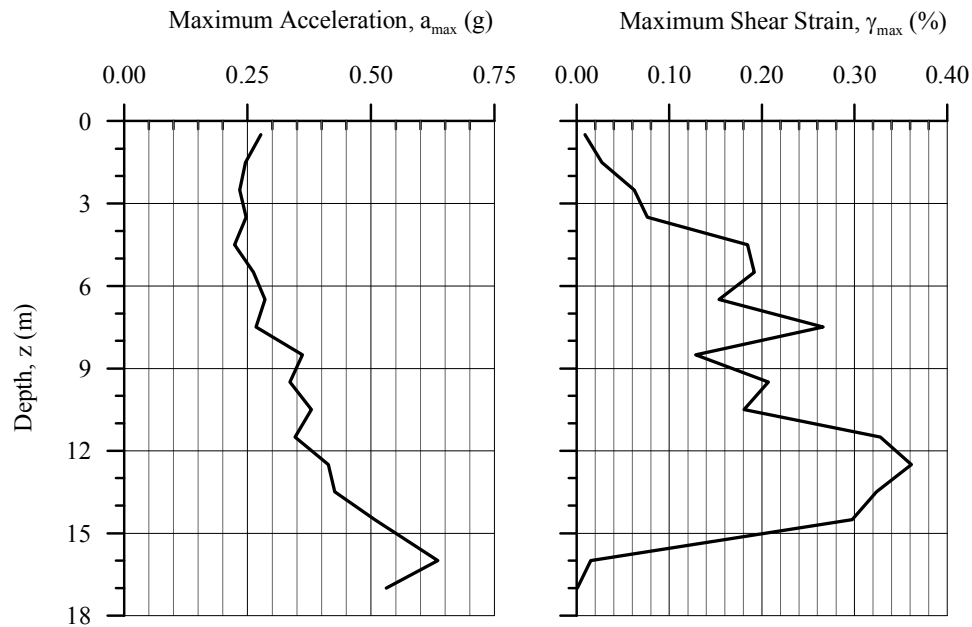


Figure 4-2 – Profiles of the maximum acceleration and maximum shear strain from the site response analysis using the CDSS testing data for Earthquake E3 ($M_w=7$).

The profiles of $a_{\max}$ and $\gamma_{\max}$ for Earthquake E5 site response analysis of the CDSS test results are presented on Figure 4-3. The damping within the tailings was high as shown by the decrease in $a_{\max}$ from about 1.7 g within the glacial till ($z = 15$ to 17 m) to about 0.3 g close to the ground surface. In the lower 7 m of the tailings ($z = 8$ to 15 m), the value of $\gamma_{\max}$ varies from about 0.9 to 1.4%. From a depth of 8 m to the surface, $\gamma_{\max}$ decreases to near zero. The high shear strains developed in the lower part of the tailings would have resulted in a 90 to 95% reduction in the shear modulus.

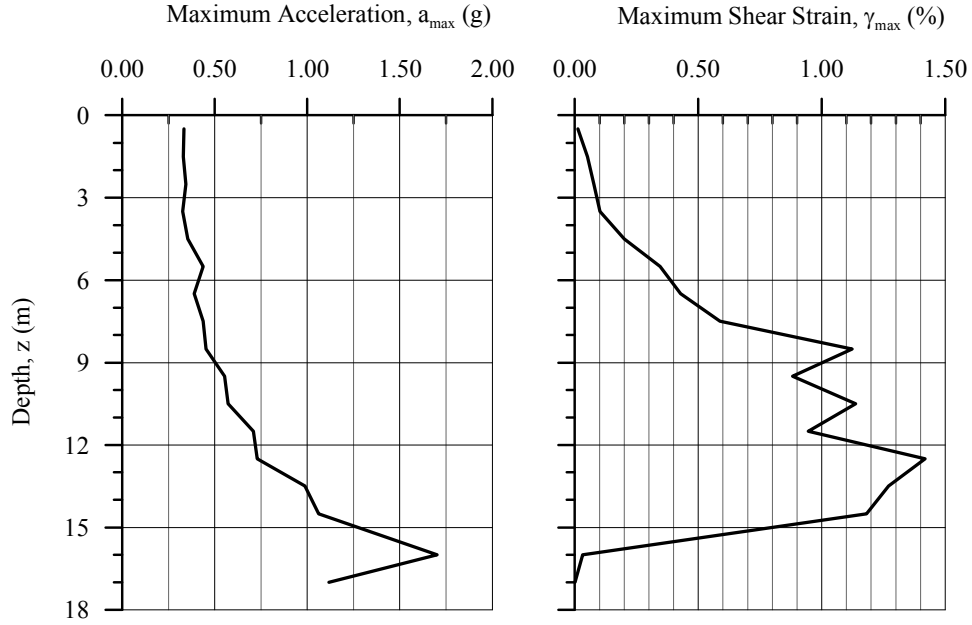


Figure 4-3 – Profiles of the maximum acceleration and maximum shear strain from the site response analysis using the CDSS testing data for Earthquake E5 ($M_w=7.5$).

As noted previously, the (initial or maximum) shear modulus of the tailings as estimated from CDSS testing is relatively low which indicates that the tailings have a moderately low capacity to transmit shear waves (and thus earthquake ground motions) and will developed significant shear strains in response to earthquake loading. Also, the degree of damping and amount of shear strain were expected to increase significantly with increasing earthquake load. The results of the site response analyses agree with the expected behavior.

4.2 Calculation of the Cyclic Stress and Cyclic Resistance Ratios

Cyclic stress ratio, CSR, profiles within the tailings due to Earthquakes E1, E3 and E5 were estimated using Equation 2-5 with the stress reduction coefficient, r_d , developed from the site response analyses.

The maximum acceleration values, a_{max} , estimated at the center of each layer in the tailings during the site response analyses of Earthquakes E1, E3 and E5 were divided by the PGA of the input ground motion and assumed to be equivalent to the stress reduction coefficients at the corresponding levels. Since the site response of the three earthquake loads varied significantly,

the results of each load were considered separately to produce r_d values that were site and load-specific. The estimated r_d ($a_{\max}/\text{PGA}$) values for Earthquakes E1, E3 and E5 are plotted on Figures 3-43, 3-44 and 3-45, respectively. The figures also contain a trend line and an equation for the trend line. The following equations were developed from Figures 3-43, 3-44 and 3-45 to estimate the stress reduction coefficients.

For Earthquake E1:
$$r_d = e^{\left(\frac{z-15.536}{22.509}\right)} \quad (4-1)$$

For Earthquake E3:
$$r_d = e^{\left(\frac{z-19.603}{16.414}\right)} \quad (4-2)$$

For Earthquake E5:
$$r_d = e^{\left(\frac{z-18.896}{10.714}\right)} \quad (4-3)$$

where z is the depth below the surface in meters.

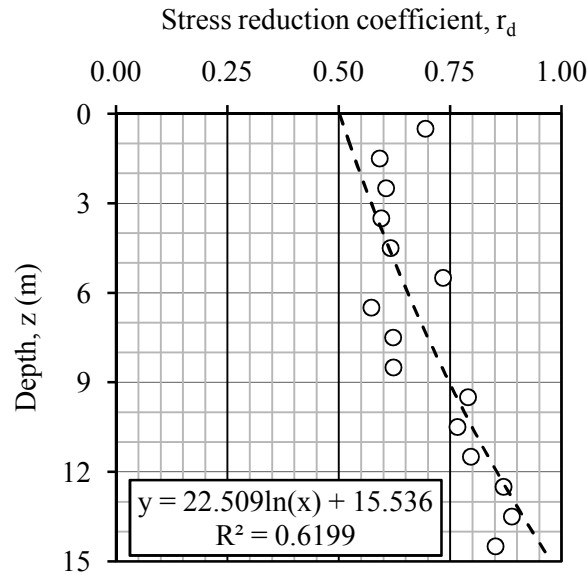


Figure 4-4 – Estimation of the stress reduction coefficient, r_d , based on site response analysis of CDSS data for Earthquake E1 ($M_w=6.5$).

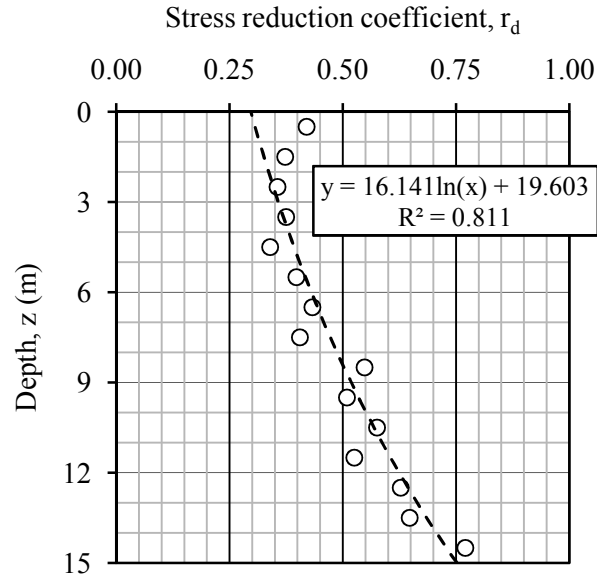


Figure 4-5 – Estimation of the stress reduction coefficient, r_d , based on site response analysis of CDSS data for Earthquake E3 ($M_w=7$).

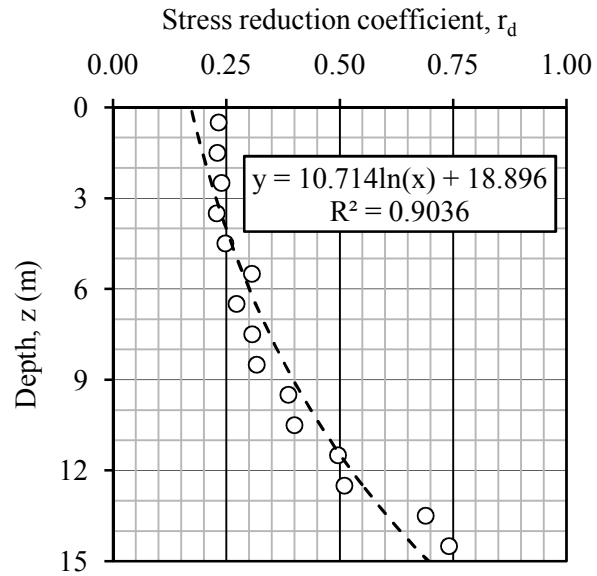


Figure 4-6 – Estimation of the stress reduction coefficient, r_d , based on site response analysis of CDSS data for Earthquake E5 ($M_w=7.5$).

The cyclic resistance ratio, CRR, of the tailings was calculated using Equation 3-5 with the value N , number of significant cycles, equal 15 for Earthquake E5 ($M_w=7.5$). Then N value is based on the findings of Arango (1994), refer to Section 2.3.1.3, specifically Table 2-5. The cyclic resistance ratios for the magnitude 6 and 7 events (E1 and E3, respectively) were calculated by factoring the CRR of the magnitude 7.5 event by the appropriate magnitude scaling factor, MSF, estimated from the CDSS testing and given in **Error! Reference source not found.** The cyclic resistance ratio profiles of the tailings are constant with depth; this is a reflection of the lack of influence of the consolidation stress on the liquefaction resistance as observed in the CDSS testing.