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TITLE OF COMPUTATIONS **LIQUEFACTION POTENTIAL ANALYSES AND THE ADDENDUM**

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LIQUEFACTION POTENTIAL ANALYSES

INTRODUCTION

This calculation package was prepared as part of the In-Lake Waste Deposit (ILWD) geotechnical stability evaluation for the Onondaga Lake Bottom Site. Specifically, the purpose of this package is to present liquefaction potential analyses for the ILWD area. The ILWD area consists of Sediment Management Unit (SMU) 1 and limited portions of SMUs 2, 7, and 8. Liquefaction potential of the Solvay Waste (SOLW) and the underlying soils was evaluated for existing conditions.

The evaluation of the capped condition is not explicitly included herein because the evaluation of the existing SOLW and underlying soils will not be affected by the installation of a cap (anticipated to be approximately 3 to 5.5-ft thick) at slopes that are similar to existing conditions. However, for completeness, the effect of cap weight on the liquefaction potential of the existing SOLW and underlying soils and the liquefaction potential of the cap itself are addressed in an addendum to this calculation package.

The remainder of this calculation package presents: (i) technical framework; (ii) subsurface stratigraphy and material properties; (iii) methodology; (iv) results; and (v) conclusions.

TECHNICAL FRAMEWORK

A technical framework for the proposed liquefaction evaluation is presented in this section. Defining a framework is important because the term "liquefaction" is used to describe a variety of phenomena in the literature. A description of the different liquefaction mechanisms and liquefaction potential evaluation procedures are presented in the following sections.

<u>Liquefaction Mechanisms</u>

Kramer [1996] writes the following about the term "liquefaction":

"*The term liquefaction……has historically been used in conjunction with a variety of phenomena that involve soil deformations caused by monotonic, transient, or repeated* *disturbance of saturated cohesionless soils under undrained loading conditions. The generation of excess pore pressure under undrained loading conditions is a hallmark of all liquefaction phenomena*".

Generally liquefaction phenomena can be divided into two main groups: (i) flow liquefaction (or true liquefaction); and (ii) cyclic mobility (or cyclic liquefaction). These two types of "liquefaction" phenomena are illustrated in Figure 1 and explained in the following paragraphs.

Flow liquefaction can occur when the shear stress required for static equilibrium of a soil mass (static shear stress, τ_d in Figure 1) is greater than the shear strength of the soil in its liquefied state (S_{us} in Figure 1). The shear strength of the soil in its liquefied state is also referred to as the undrained steady state shear strength or residual undrained shear strength in the literature. This shear strength is less than the peak shear strength for strain softening soils and is the same as the peak shear strength for strain hardening soils. Deformations produced by flow liquefaction are driven by static shear stresses and can be very large [Kramer 1996]. Flow liquefaction can be initiated by seismic loading, vibrations such as pile driving, geophysical exploration, blasting, and/or monotonic loading (static liquefaction). Flow liquefaction stress paths due to monotonic loading and cyclic loading are illustrated in Figure 1(a). The above discussion about flow liquefaction is generally applicable to cohesionless soils and soils with low plasticity. The term liquefaction is not generally used for cohesive soils that show "clay-like" behavior. However, undrained shear strength of sensitive clays or cemented soils can reduce from their undisturbed undrained shear strength to remolded undrained shear strength when disturbed and show a "flow liquefaction"-like behavior.

Cyclic mobility can be initiated by cyclic loading (i.e., seismic or periodic wave loading) resulting in the development of incremental deformations during loading [Kramer 1996]. It can occur when the static shear stress is less than the shear strength of the liquefied soil, and it will not result in flow liquefaction, which is discussed in the previous paragraph. However, if the static shear stress is greater than the shear strength of the liquefied soil, cyclic mobility can act as a trigger to push the stress path past the peak shear strength and lead to flow liquefaction. Conversely, if cyclic loading is not strong enough to trigger cyclic mobility, flow liquefaction is not likely to occur under that same loading. Cyclic mobility stress path due to cyclic loading is illustrated in Figure 1(b). Monotonic loading stress paths for the same soil are also provided in this figure to illustrate that the soil deforming due to cyclic mobility still has shear strength to resist shear stresses.

While the term cyclic mobility or cyclic liquefaction is generally used for cohesionless soils and soils with low plasticity, the term cyclic softening is used to describe the behavior of silty and clayey soils during earthquakes [Boulanger and Idriss, 2007].

Liquefaction Potential Evaluation Procedures

The state-of-practice for evaluating the liquefaction potential does not explicitly account for different liquefaction mechanisms. State-of-practice procedures are mainly based on case histories of occurrences and non-occurrences of liquefaction due to past earthquakes. Occurrences (or non-occurrences) of liquefaction are determined by presence (or absence) of surface manifestations of liquefaction such as sand boils, ground cracking, slope movements, and/or flow failures. Surface manifestations are generally present if high pore pressures are generated due to seismic loading and "liquefaction" is triggered. Therefore, if soils at a particular site are deemed to be not susceptible to liquefaction based on methods used in state-of-practice, further analyses such as post-liquefaction slope stability or flow liquefaction are not needed for seismic loading.

An initial step in performing a liquefaction potential evaluation is application of screening criteria based on geotechnical properties to evaluate whether subsurface materials are potentially liquefiable. Seismic loading is not considered in this screening evaluation. In general, soils that show "clay-like" behavior are not susceptible to liquefaction. Boulanger and Idriss [2007] proposed a procedure to evaluate the potential for cyclic softening of silty and clayey soils based on undrained static shear strengths and seismic loading. The screening criteria and the Boulanger and Idriss [2007] procedure to evaluate cyclic softening basically evaluate the potential for significant pore pressure increase due to seismic loading, and therefore cover all forms of "liquefaction" due to seismic loading.

The state-of-practice for liquefaction analysis for cohesionless soils is based on empirical correlations based on insitu soil tests such as Standard Penetration Tests (SPT) or Cone Penetration Tests (CPT). The effect of seismic loading is considered in this approach. This procedure was developed based on field case histories where evidence of liquefaction was or was not observed after earthquakes, and, therefore, covers all forms of "liquefaction" due to seismic loading.

In addition to the state-of-practice based methods, potential for flow liquefaction or sensitivity for loss of shear strength can be directly evaluated for soils based on stress-strain behavior during laboratory tests such as undrained triaxial tests. A pronounced strain softening behavior where the shear strength peaks at relatively low strains and then drops significantly to reach a steady state or residual value may be an indication of the potential to liquefy under certain conditions. On the other hand, a strain hardening behavior where the shear strength keeps increasing as the soil is strained or a limited strain softening behavior where the shear strength peaks and then drops slightly to reach a steady value indicates that flow liquefaction or sensitivity is not an issue. These three types of soil behavior are illustrated in Figure 2. It is noted that liquefaction due to cyclic mobility may still be triggered in a strain hardening soil depending on the acceleration and magnitude of the seismic loading.

SUBSURFACE STRATIGRAPHY AND MATERIAL PROPERTIES

Subsurface soils in the ILWD area consist of primarily seven strata (from top to bottom): (i) SOLW; (ii) marl; (iii) silt and clay; (iv) silt and sand; (v) sand and gravel; (vi) till; and (vii) shale. Standard Penetration Tests (SPT) were conducted in most of the borings to measure the SPT blow count values. Samples of SOLW, marl, and silt and clay were collected during the investigations for laboratory testing of index properties, shear strength, and compressibility. A detailed description of the development of the subsurface model and geotechnical parameters is presented in Appendix A titled "Summary of Subsurface Stratigraphy and Material Properties".

SOLW, marl, and silt and clay units can be classified as mainly MH, MH and CH, and CL and CH type material based on the Unified Soil Classification System (USCS). SPT values for SOLW, marl, and silt and clay mainly ranged from 0 to 7 (with most of the reported blow counts being 0). Plasticity index values for SOLW mainly ranged from 10 to 80. Most of the SOLW samples had water contents that were higher than their liquid limits. However, under laboratory undrained shearing, 12 out of 17 SOLW samples (two to three specimens were tested for each sample) showed strain hardening ductile behavior. Out of the remaining five samples, three showed limited strain-softening behavior and two showed gradual strain softening behavior. Based on laboratory triaxial test results, an undrained shear strength ratio of 0.35 for SOLW, marl, and silt and clay were selected to model the shear strength under undrained conditions. The undrained shear strength ratio of 0.35 for SOLW was subsequently adjusted to account for overconsolidation, corresponding to an overconsolidation ratio (OCR) of 2. SPT values for the deeper soil layers mainly ranged from 20 to 100. Table 1 summarizes the material properties of each subsurface layer (i.e., SOLW and soils).

METHODOLOGY

The liquefaction potential evaluation methodology used for the ILWD area is presented in this section. Screening criteria, Boulanger and Idriss [2007] evaluation procedure for "cohesive" soils, and Seed and Idriss [1971] evaluation procedure for "cohesionless" soils are applied to evaluate the potential for triggering liquefaction. In addition, the potential for flow liquefaction and/or sensitive behavior is directly evaluated for SOLW using stress paths observed in static triaxial tests. This evaluation is described in the following five steps:

1. A general screening is conducted to assess the liquefaction potential of the ILWD. Several screening criteria are used in state-of-practice for evaluating the liquefaction potential of cohesive soils (generally soils that can be classified as ML, CL, MH, CH or combinations of these). These screening criteria are developed from actual field evidence of both liquefaction and no liquefaction in different soil types and supplemental laboratory studies. These criteria cover both flow and cyclic liquefaction due to seismic loading. The following three criteria were used in the screening evaluation presented in this calculation package:

   a. Chinese criteria [Wang 1979] has been widely used for the past two decades in US engineering practice to screen liquefaction potential of soils. Soil is considered susceptible if all three of the following conditions are met:

      - percent finer than 0.005 mm $\leq$ 15%

      - Liquid Limit $\leq$ 35%

      - Water Content $\geq$ 0.9 x Liquid Limit

      Figure 3 presents these criteria in a chart format.

   b. Andrews and Martin [2000] presented screening criteria to evaluate the liquefaction susceptibility of silty and clayey sands. These criteria are based on clay fraction (minus 0.002 mm) and Liquid Limit of soils. Figure 4 presents these criteria in a tabular form.

c. Bray and Sancio [2006] evaluated several case histories and performed laboratory cyclic triaxial tests to develop screening criteria based on water content, Liquid Limit, and Plasticity Index. These criteria and the data used to develop them are shown in Figure 5.

2. Cyclic stresses caused by seismic loading within soil units need to be evaluated for liquefaction analyses. The cyclic stresses on the soils are calculated as follows to evaluate the liquefaction potential for "cohesive" soils and "cohesionless" soils described below in steps 3 and 4, respectively.

   a. Design bedrock acceleration for a contingency level seismic event (i.e., a seismic event with a 10 percent chance of exceedance in 50 years) was established using United States Geological Survey (USGS) seismic hazard maps [USGS, 2008].

   b. The design earthquake magnitude was established using deaggregated seismic hazard provided by USGS. Deaggregation is done to identify the earthquake that is contributing the most to the total hazard at the site.

   c. Maximum ground surface acceleration for the contingency level seismic event was estimated by considering potential for amplification using the chart proposed by Idriss [1990] for soft soil sites. This chart is presented in Figure 6. Application of this chart in lieu of site response analyses based on time histories is generally considered to be conservative.

   d. Cyclic Stress Ratio (CSR) was evaluated using the simplified procedure proposed by Seed and Idriss [1971]. The steps involved and equations used are described below.

$$CSR_M = 0.65 r_d \frac{a_{\max}}{g} \frac{\sigma_{v0}}{\sigma_{v0}'}$$

   Where:

   CSR_M = Cyclic Stress Ratio due to an earthquake with magnitude M;

r_d = stress reduction factor;

a_{max} = maximum ground surface acceleration;

g = gravitational acceleration;

σ_{v0} = total vertical stress; and

σ_{v0}' = effective vertical stress.

The r_d value was calculated using the following equation presented in NCEER [1997] to approximate the mean values of the possible range of r_d.

$$r_d = \frac{\left(1.000 - 0.4113z^{0.5} + 0.04052z + 0.001753z^{1.5}\right)}{\left(1.000 - 0.4177z^{0.5} + 0.05729z - 0.006205z^{1.5} + 0.001210z^{2}\right)}$$

Where:

z = depth below ground surface in meters.

3. Cyclic softening potential was evaluated for cohesive soils using the procedure proposed by Boulanger and Idriss [2007]. This procedure is similar to the Seed and Idriss [1971] simplified procedure used for liquefaction evaluation of cohesionless soils, with some modifications for application to cohesive soils. Because this procedure evaluates the potential for significant pore pressure increase due to seismic loading, it covers all forms of "liquefaction" due to seismic loading. The steps involved and equations used are described below.

$$CRR_M = 0.8\frac{S_U}{\sigma_{v0}'}K_\alpha MSF$$

Where:

CRR_M = Cyclic Resistance Ratio for an earthquake with magnitude M;

S_U = static undrained shear strength;

k_α = correction factor for driving static shear stresses; and

MSF = Magnitude Scaling Factor.

The k_α is a function of the driving static shear stresses or slope angle. For the mild overall slopes of the ILWD area ranging from three to five degrees, k_α can be assumed to be one.

The MSF for clay type soils can be calculated using the equation proposed by Boulanger and Idriss [2007] as illustrated in Figure 7.

$$MSF = 1.12\exp\left(\frac{-M}{4}\right) + 0.828, \text{ and MSF} \le 1.13 \text{ (for clay)}$$

Factor of safety against liquefaction (FS_{liq}) can be calculated as follows:

$$FS_{liq} = \frac{CRR_M}{CSR_M}$$

4. Liquefaction potential was evaluated for cohesionless soils using the simplified procedure proposed by Seed and Idriss [1971]. Because this procedure evaluates the potential for significant pore pressure increase due to seismic loading, it covers all forms of "liquefaction" due to seismic loading. The steps involved in this SPT based procedure and equations used are described below.

   Figure 8 presents the relationship between SPT blow counts and $CRR_{7.5}$ based on case histories [NCEER, 1997]. The corrected normalized SPT blow count, $(N_1)_{60}$ can be calculated by the following equation presented by NCEER [1997].

$$(N_1)_{60} = N_m C_N C_E C_B C_R C_S$$

   Where:

   N_m = measured SPT blow count;

C_N = correction for overburden pressure;

C_E = correction for energy ratio;

C_B = correction for borehole diameter;

C_R = correction for rod length; and

C_S = correction for sampler.

C_N can be calculated as follows:

$$C_N = \sqrt{\frac{P_a}{\sigma_{vo}^{'}}}$$

Where:

P_a = atmospheric pressure (2117 psf).

The other corrections will be applied based on NCEER [1997] procedures as needed.

The MSF for cohesionless soils can be calculated using the equation proposed by Idriss [2007], as illustrated in Figure 7.

$$MSF = 6.9\exp\left(\frac{-M}{4}\right) - 0.058 \text{, and MSF} \le 1.8 \text{ (for sand)}$$

CRR_M is calculated by multiplying $CRR_{7.5}$ by the MSF.

FS_{liq} can be calculated as presented in step 3 above.

5. The potential for flow liquefaction or sensitivity for loss of shear strength can be directly evaluated for soils based on stress-strain behavior during laboratory tests such as undrained triaxial tests, as discussed in the technical framework section. Stress-strain plots for SOLW were compared with standard stress-strain plots for strain hardening, limited strain softening, and strain softening soil behavior. These three types of soil behavior were illustrated in Figure 2. It is noted that

liquefaction due to cyclic mobility cannot be evaluated with this procedure; however, it will be evaluated in the steps described previously.

RESULTS

The liquefaction potential evaluation results are presented in this section, and the steps in this section directly correspond to the steps in the methodology section.

1. Application of screening criteria used in state-of-practice for evaluating the liquefaction potential of cohesive soils indicates that SOLW, marl, and silt and clay units in the ILWD area are not susceptible to liquefaction.

   a. Figure 9 presents the application of the Chinese Criteria for SOLW, marl, and silt and clay. Based on these criteria these soils can be considered not susceptible to liquefaction.

   b. Liquid limits for SOLW, marl, and silt and clay are greater than 32. Based on the lab results, the clay content (particle size less than 0.002 mm) typically ranges from 5% to 30% for SOLW, from 20% to 43% for marl, and from 14% to 50% for silt and clay. The average clay content was calculated to be 14%, 30%, and 30% for SOLW, marl, and silt and clay, respectively. Per the screening criteria proposed by Andrews and Martin [2000] if clay content is greater than or equal to 10% and liquid limit greater than or equal to 32, soils can be considered not susceptible to liquefaction. If clay content is less than 10% or liquid limit is less than 32, further studies are required. Therefore, in general SOLW, marl, and silt and clay are not considered susceptible to liquefaction based on these criteria.

   c. Figure 10 presents the application of the criteria proposed by Bray and Sancio [2006] to SOLW, marl, and silt and clay. Values of water content, liquid limit, and plasticity index were used to classify samples as susceptible, moderately susceptible, and not susceptible to liquefaction. Out of a total of 101 SOLW samples, 3, 11, and 87 samples were classified as susceptible, moderately susceptible, and not susceptible to liquefaction, respectively. Out of a total of 35 marl samples, 1, 0, and 34 were classified as susceptible, moderately susceptible, and not susceptible to liquefaction, respectively. Out of a total of 47 silt and clay samples, 3, 9, and 35 were classified as susceptible, moderately susceptible, and not susceptible to liquefaction, respectively. A few samples being classified as susceptible to liquefaction are not likely to cause overall liquefaction of the ILWD. Therefore, based on these criteria, these soils are not considered susceptible to liquefaction.

2. The cyclic stresses on the soils are calculated using the following steps:

   a. Figure 11 presents the peak ground acceleration with a 10% probability of exceedance in 50 years [USGS, 2008]. A latitude of 43° 04' N and a longitude of 76° 11' W were used for the ILWD area to obtain a PGA value of 0.025g (0.02478g) using the interactive maps from the USGS website. Attachment 1 presents the deaggregated seismic hazard for the 10% probability of exceedance in 50 year event. It is noted that the deaggregated hazard was based on the 2002 USGS hazard maps because deaggregated data for 2008 maps are not yet available. Based on the deaggregated hazard, a 5.3 moment magnitude was selected for use in liquefaction analyses as explained in Attachment 1.

   b. Maximum ground surface acceleration of 0.09g was estimated for this seismic event by considering potential for amplification using the recommended mean relation in the chart presented in Figure 6. Application of this chart in lieu of site response analyses based on time histories is generally considered to be conservative.

   c. Table 2 presents the CSR values calculated using the simplified procedure proposed by Seed and Idriss [1971]. The calculated CSR values are plotted with depth in Figure 12. The calculated CSR values generally ranged from 0.10 at 70 feet depth in the silt and clay unit to 0.25 near the top of the SOLW.

3. Table 2 presents the CRR values for cohesive soils calculated using the procedure proposed by Boulanger and Idriss [2007]. A normalized static strength ratio of 0.35 was used for SOLW, marl, and silt and clay as presented in Table 1 and Appendix A titled "Summary of Subsurface Stratigraphy and Material

Properties". The undrained shear strength ratio of 0.35 for SOLW was subsequently adjusted to account for overconsolidation, corresponding to an OCR of 2. For the mild overall slopes of the ILWD area ranging from three to five degrees, k_{α} was assumed to be one. A MSF value of 1.13 was calculated. The calculated CRR values are plotted with depth in Figure 12. The calculated CRR values generally ranged from 0.32 for the marl and the silt and clay units to 0.55 for the SOLW. Calculated factors of safety against liquefaction are plotted in Figure 13. Calculated factors of safety against liquefaction ranged from about 2.2 to 2.3 for SOLW, 1.5 to 1.7 for marl, and 1.9 to 3.1 for silt and clay units. In liquefaction analyses, calculated factors of safety of 1.0 to 1.2 are considered adequate to conclude that adverse effects due to pore pressure buildup are unlikely. Therefore, based on this analysis, SOLW, marl, and silt and clay units in the ILWD area are not considered to be susceptible to liquefaction during the design seismic event established in step 2.

4. Uncorrected SPT blow counts for deeper soil units such as silt and sand, and sand and gravel ranged from 20 to 100 or more. An uncorrected SPT blow count of 20 is very conservatively assumed for demonstration purposes. It is assumed that energy correction is not required because SPT testing was done per standard US practice. After the application of overburden correction, which depends on the effective stress at a particular depth, one can calculate corrected blow count values ($N_{1,60}$) of about 13 to 20 for depths of 70 feet to 120 feet. Based on Figure 8, and assuming a fine content of less than 5%, these correspond to $CRR_{7.5}$ values of about 0.14 to 0.22. An MSF value of 1.7 can be calculated for cohesionless soils based on Figure 7. Therefore, calculated CRR_M values range from about 0.24 to 0.37. These values are much greater than the CSR_M value of 0.10 calculated for 70 feet. CSR_M values below 70 feet will be even smaller. Therefore, based on this simple analysis, the silt and sand, and sand and gravel units are not considered susceptible to liquefaction during the design seismic event established in step 2.

5. Figures 14 and 15 present the stress-strain and q-p' paths, respectively, for SOLW under laboratory consolidated undrained monotonic triaxial tests. These tests were conducted with applied effective confining stresses that are in the general range of insitu effective vertical stresses. Under laboratory undrained shearing,

12 out of 17 SOLW samples (two to three specimens were tested for each sample) showed strain-hardening ductile behavior. Out of the remaining five samples, three showed limited strain-softening behavior (OL-STA-20052, OL-STA-20038, and OL-SB-10131 in Figure 14) and two showed gradual strain softening behavior (OL-SB-10133 and OL-SB-10135 in Figure 14). This gradual softening appears different from the sudden strength loss that is typical of soils susceptible to flow liquefaction or sensitive behavior. Therefore, based on these tests one can conclude that SOLW in the ILWD area is not likely to be susceptible to flow liquefaction or show sensitive behavior.

CONCLUSIONS

Liquefaction potential of the Solvay Waste (SOLW) and the underlying soils was evaluated for existing conditions. Based on the results summarized herein, the ILWD and underlying soils are not considered to have the potential for liquefaction or cyclic softening during the contingency level seismic event. In addition, the SOLW does not appear to have the potential for sensitive behavior or loss of shear strength.

As indicated previously, the evaluation of the capped condition is not explicitly included herein because the liquefaction potential evaluation of the existing SOLW and underlying soils will not be affected by the installation of a cap (anticipated to be approximately 3 to 5.5-ft thick) at slopes that are similar to existing conditions. However, for completeness, the effect of cap weight on the liquefaction potential of the existing SOLW and underlying soils and the liquefaction potential of the cap itself are addressed in an addendum to this calculation package.

REFERENCES

Andrews, D.C.A. and Martin, G.R. (2000). "Criteria for Liquefaction of Silty Soils", 12th World Conference on Earthquake Engineering, Proceedings, Auckland, New Zealand.

Boulanger, R.W. and Idriss, I.M. (2007), "Evaluation of Cyclic Softening in Silts and Clays", Journal of Geotechnical and Geoenvironmental Engineering, ASCE, Volume 133, No. 6, June 2007, pp. 641 - 652.

Bray, J.D. and Sancio, R.B. (2006). "Assessment of the Liquefaction Susceptibility of Fine-Grained Soils", Journal of Geotechnical and Geoenvironmental Engineering, ASCE, September 2006, Vol. 132, No. 9, pp. 1165-1177.

Castro, G. (1976). "Comments on Seismic Stability Evaluation of Embankment Dams", *Proceedings of the Conference on Evaluation of Dam Safety.*

Finn, W.D.L., Ledbetter, R.H., And Wu, G. (1994). "Liquefaction in Silty Soils: Design and Analysis", Ground Failures under Seismic Conditions, Geotechnical Special Publication 44, ASCE, New York, pp. 51 - 76.

Idriss, I.M. (1990). "Response of Soft Soil Sites during Earthquakes", in J.M. Duncan, ed., Proceedings, H. Bolton Seed Memorial Symposium, BiTech Publishers, Vancouver, British Columbia, Vol. 2, pp. 273 - 289.

Kramer, S.L (1996). "Geotechnical Earthquake Engineering", Prentice Hall, Upper Saddle River, New Jersey.

NCEER (1997). "Summary Paper, Proc. of NCEER Workshop on Evaluation of Liquefaction Resistance of Soils", NCEER-97-0022.

Seed, H.B. and Idriss, I.M. (1971). "Simplified Procedure for Evaluating Soil Liquefaction Potential", Journal of the Soil Mechanics and Foundations Division, ASCE, Vol. 107, No. SM9, pp. 1249 - 1274.

Seed, R. B., Cetin, K. O., Moss, R. E. S., Kammerer, A. M., Wu, J., Pestana, J. M., and Riemer, M. F. (2001) "Recent Advances in Soil Liquefaction Engineering and Seismic Site Response Evaluation", 4th International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, San Diego, March 28-31, 2001.

USGS (2008). "United States Geological Survey (USGS) seismic hazard maps", http://earthquake.usgs.gov/research/hazmaps/

Wang, W. (1979). "Some Findings in Soil Liquefaction", Water Conservancy and Hydroelectric Power Scientific Research Institute, Beijing, China.

Yamamuro, J. and Covert, K. (2001). "Monotonic and Cyclic Liquefaction of Very Loose sands with High Silt Content", Journal of Geotechnical and Geoenvironmental Engineering, ASCE, Volume 127, No. 4, April 2001, pp. 314 - 324.

Tables

Table 1. Summary of material properties

Material	General Classification	Typical Range of Liquid Limit (%)	Typical Range of Plastic Limit (%)	Typical Range of Water Content (%)	Typical Range of Plasticity Index	Typical Range of Liquidity Index	Typical Range of Fines Content	Typical Range of SPT N value	Total Unit Weight (pcf)	Undrained Shear Strength[1]	
										From UU (psf)	From CU (psf)
Silt[2]	NA	NA	NA	NA	NA	NA	NA	NA[1]	98	NA	NA
SOLW	MH	40 – 146	30 – 80	40 – 260	10 – 80	1.0 – 6.4	65 – 100	0 – 7[3]	81	245	$S_u/\sigma'_v = 0.35$[4]
Marl	MH and CH	60 – 80	25 – 45	35 – 85	20 – 50	0.4 – 1.1	96 – 100	0 – 4[3]	98	350	$S_u/\sigma'_v = 0.35$
Silt/Clay	CL and CH	32 – 70	20 – 45	20 – 80	15 – 40	0.4 – 1.0	93 - 100	0[3]	108	350	$S_u/\sigma'_v = 0.35$
Silt/Sand	NA	NA	NA	NA	NA	NA	NA	20 - 80	120	NA	NA
Sand/Gravel	NA	NA	NA	NA	NA	NA	NA	20 to > 100	120	NA	NA
Till	NA	NA	NA		NA	NA	NA	> 100	120	NA	NA
Shale	NA	NA	NA	NA	NA	NA	NA	> 100	120	NA	NA

Notes:

[1] NA – Not Applicable

[2] Properties of marl are also considered applicable for the silt overlying the Solvay waste in certain areas of the ILWD.

[3] SPT N values are zero at most of the depths within the SOLW, marl, and silt/clay layers.

[4] The undrained shear strength ratio of normally consolidated SOLW was estimated to be 0.35 as presented in the Data Package [The value of 240 psf for shear strength of the SOLW reported in Table 5 of the Data Package accounts for the insitu overconsolidation of SOLW. The shear strength ratio of 0.35 used herein conservatively assumes normally consolidated conditions and is used only to simplify the calculations].

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Client: **Honeywell** Project: **Onondaga Lake ILWD Stability** Project/ Proposal No.: **GD4014** Task No.: **02**

Table 2. Calculation of CSR, CRR, and factor of safety against liquefaction

Unit weight of water =	62.4 pcf
Maximum surface acceleration =	0.09 g
K-alpha =	1
OCR model parameter, m =	0.8
Earthquake magnitude =	5.3
Magnitude scaling factor for cohesive soils =	1.13

Depth (ft)	Depth (m)	Idealized Soil Type	Unit Weight (pcf)	Vertical Effective Stress (psf)	Vertical Total Stress (psf)	Stress Reduction Factor	Equivalent Cyclic Shear Stress (psf)	CSR_M	Cu/Sigv' - NC	OCR	Cu/Sigv' - OC	$CRR_{7.5}$	CRR_M	FSliq
0	0.0	SOLW	81	0	0	1.00	0		0.35	2.0	0.61	0.49	0.55	
5	1.5	SOLW	81	93	405	0.99	23	0.25	0.35	2.0	0.61	0.49	0.55	2.18
10	3.0	SOLW	81	186	810	0.98	46	0.25	0.35	2.0	0.61	0.49	0.55	2.20
15	4.6	SOLW	81	279	1,215	0.97	69	0.25	0.35	2.0	0.61	0.49	0.55	2.22
20	6.1	SOLW	81	372	1,620	0.96	91	0.24	0.35	2.0	0.61	0.49	0.55	2.25
25	7.6	SOLW	81	465	2,025	0.94	112	0.24	0.35	2.0	0.61	0.49	0.55	2.29
30	9.1	SOLW	81	558	2,430	0.92	131	0.23	0.35	2.0	0.61	0.49	0.55	2.34
35	10.7	Marl	98	736	2,920	0.89	152	0.21	0.35	1.0	0.35	0.28	0.32	1.52
40	12.2	Marl	98	914	3,410	0.85	170	0.19	0.35	1.0	0.35	0.28	0.32	1.70
45	13.7	Silt and Clay	108	1,142	3,950	0.80	186	0.16	0.35	1.0	0.35	0.28	0.32	1.94
50	15.2	Silt and Clay	108	1,370	4,490	0.75	198	0.14	0.35	1.0	0.35	0.28	0.32	2.18
55	16.8	Silt and Clay	108	1,598	5,030	0.70	207	0.13	0.35	1.0	0.35	0.28	0.32	2.43
60	18.3	Silt and Clay	108	1,826	5,570	0.66	215	0.12	0.35	1.0	0.35	0.28	0.32	2.68
65	19.8	Silt and Clay	108	2,054	6,110	0.62	222	0.11	0.35	1.0	0.35	0.28	0.32	2.91
70	21.3	Silt and Clay	108	2,282	6,650	0.59	230	0.10	0.35	1.0	0.35	0.28	0.32	3.12

Figures

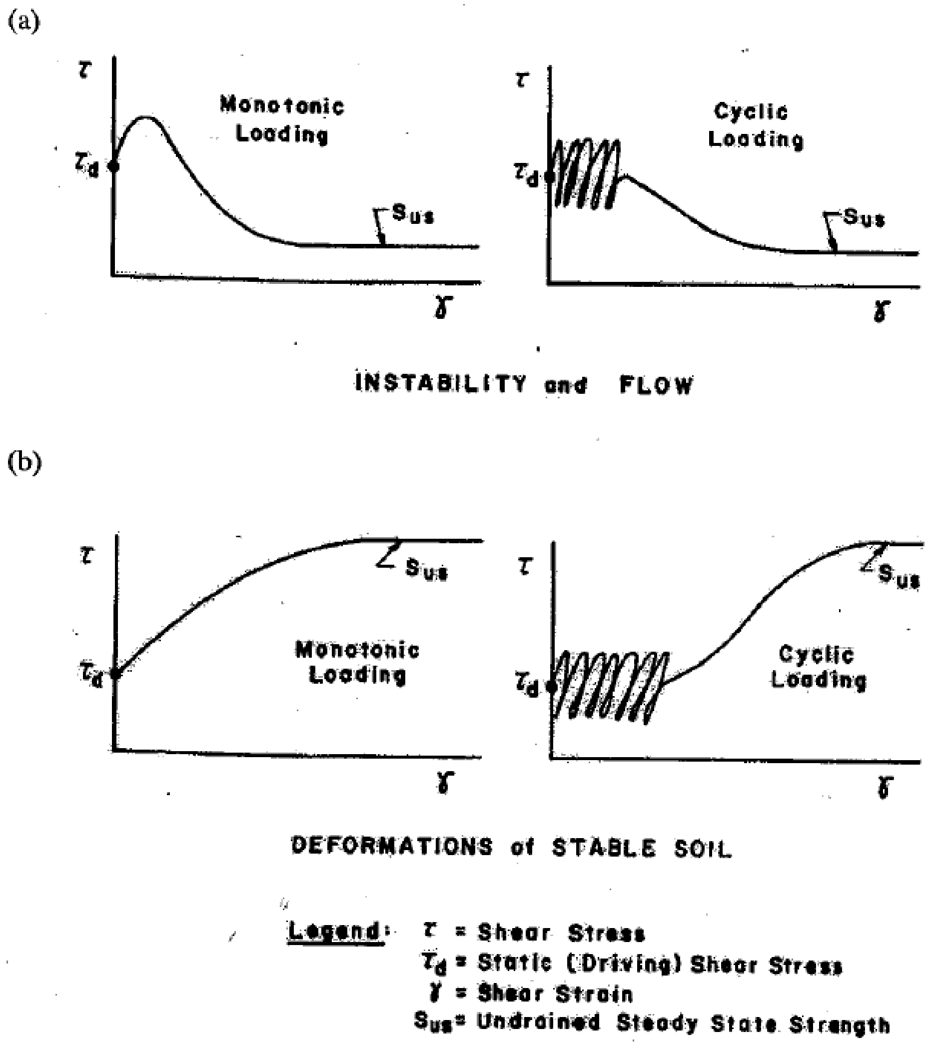
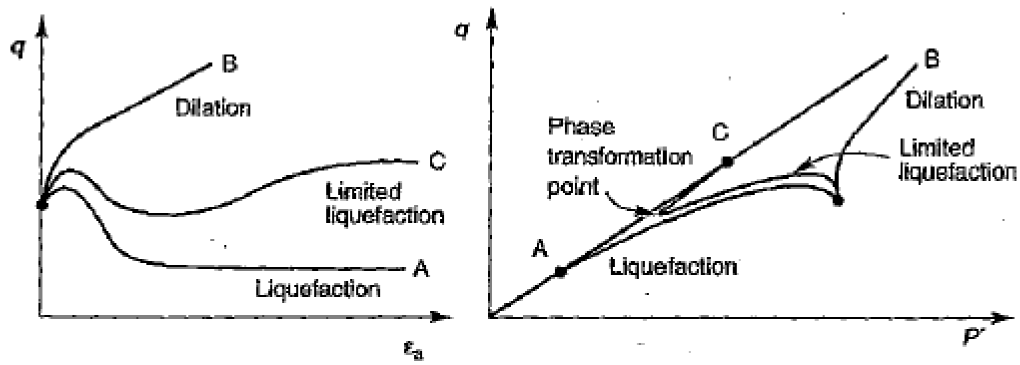


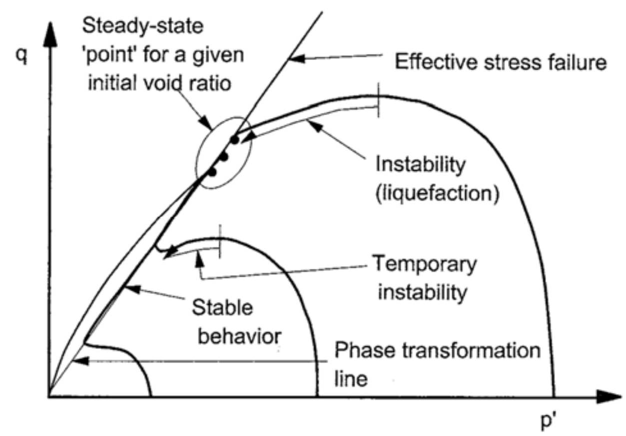
Figure 1. Monotonic and cyclic soil behaviors leading to different liquefaction mechanisms [Castro, 1976].

Notes: the following is noted in regards to the terminology used in this figure and the text:

1. The terms static loading and monotonic loading are used to describe similar loading.
2. Cyclic loading may include seismic (earthquake) or periodic wave loading.
3. “Instability and flow” corresponds to flow liquefaction behavior.
4. “Deformations of stable soil” corresponds to cyclic mobility (or cyclic softening).



(a)



(b)

Figure 2. Undrained stress-strain behavior of soils. (Figures from: (a) Kramer [1996]; and (b) Yamamuro & Covert [2001]).

Notes: the following is noted in regards to the terminology used in this figure and the text:

1. The term dilation in the figure corresponds to the strain hardening behavior described in the text.
2. The term limited liquefaction in the figure corresponds to the limited strain softening behavior described in the text.
3. The term liquefaction in the figure corresponds to the strain softening behavior described in the text.

Written by:	**R. Kulasingam**	Date:	**8/20/2008**	Reviewed by:	**Ming Zhu/Jay Beech**	Date:	**8/20/2008**
Client:	**Honeywell**	Project:	**Onondaga Lake ILWD Stability**	Project/ Proposal No.:	**GD4014**	Task No.:	**02**

1. Percent Finer than 0.005 mm ≤ 15%
2. Liquid Limit (LL) ≤ 35%
3. Water Content ≥ 0.9 x LL

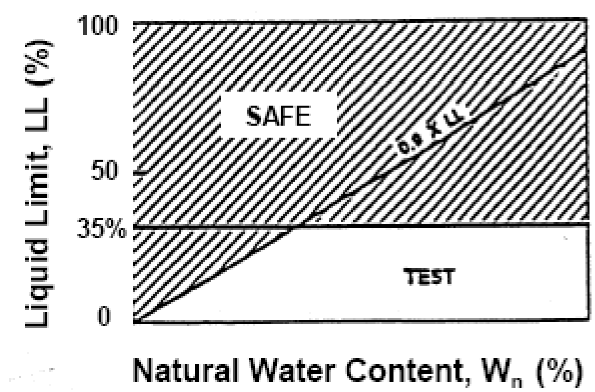


Figure 3. Modified Chinese criteria for screening liquefaction potential [Finn et al., 1994] (figure taken from Seed et al., 2001).

	Liquid Limit[1] < 32	Liquid Limit ≥ 32
Clay Content[2] < 10%	Susceptible	Further Studies Required *(Considering plastic non-clay sized grains – such as Mica)*
Clay Content[2] ≥ 10%	Further Studies Required *(Considering non-plastic clay sized grains – such as mine and quarry tailings)*	Not Susceptible

Notes:

1. Liquid limit determined by Casagrande-type percussion apparatus.
2. Clay defined as grains finer than 0.002 mm.

Figure 4. Liquefaction susceptibility of silty and clayey sands [Andrews and Martin, 2000] (figure taken from Seed et al., 2001).

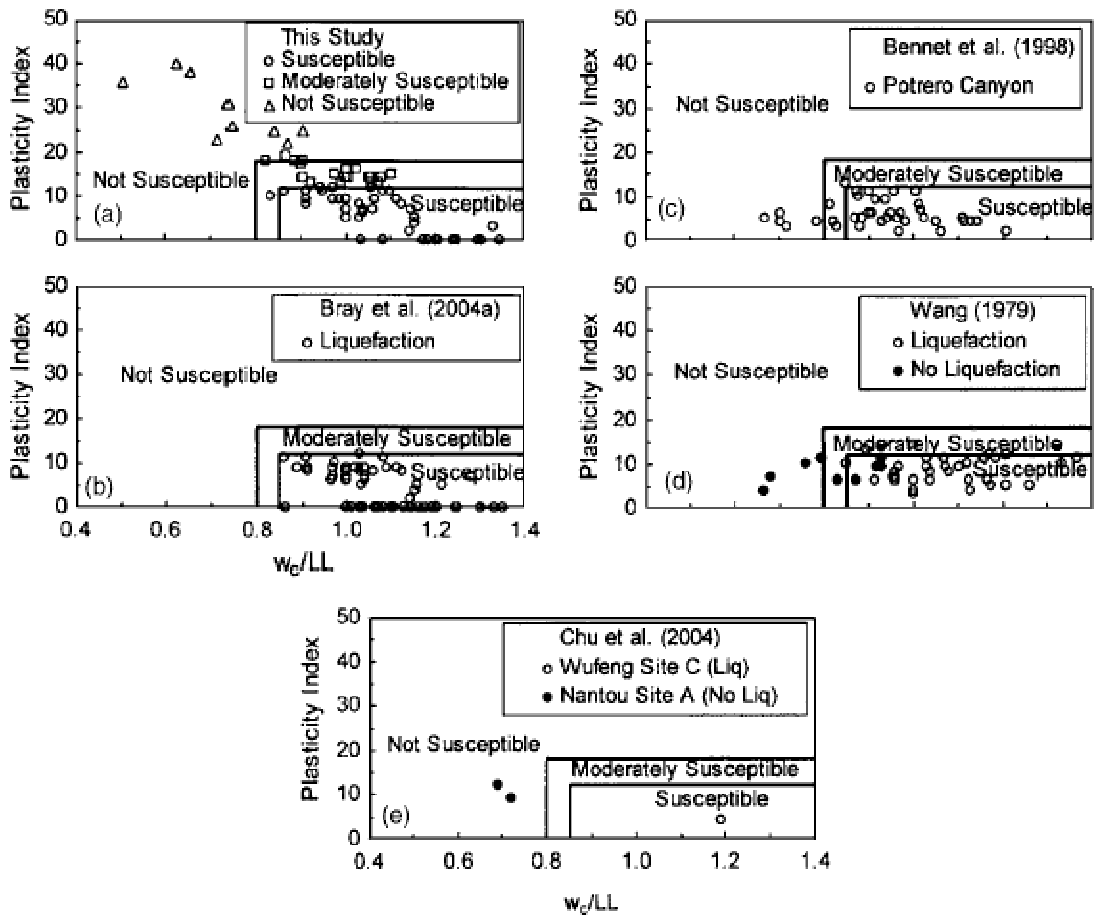


Figure 5. Liquefaction susceptibility criteria with data plotted from: (a) laboratory cyclic triaxial testing; (b) field data from Turkey (Kocaeli) earthquake; (c) field data from Northridge earthquake; (d) field data used for developing Chinese criteria; and (e) field data from Taiwan (Chi-Chi) earthquake [Bray and Sancio, 2006].

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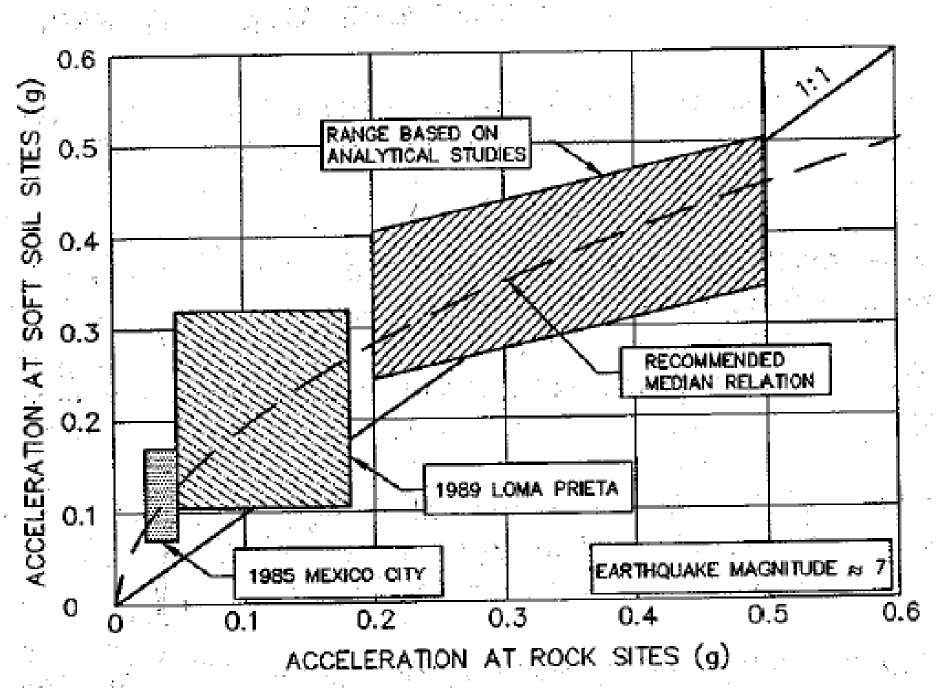

Figure 6. Relationship between peak acceleration on rock and soft soil sites [Idriss, 1990].

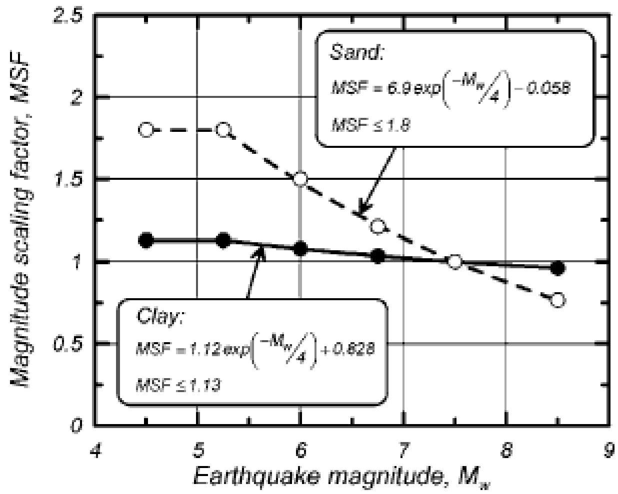

Figure 7. MSF correlations proposed by Boulanger and Idriss for clayey and sandy soils [2007].

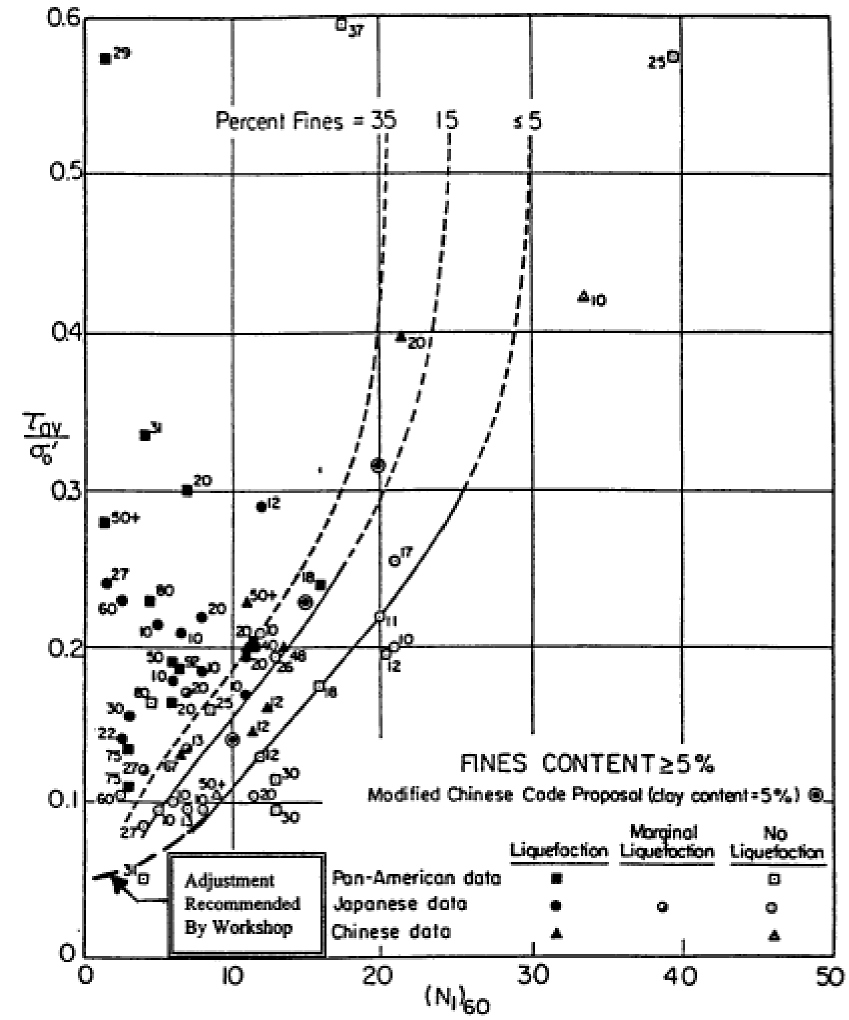

Figure 8. Relationship between SPT blow counts and CRR based on case histories [NCEER, 1997].

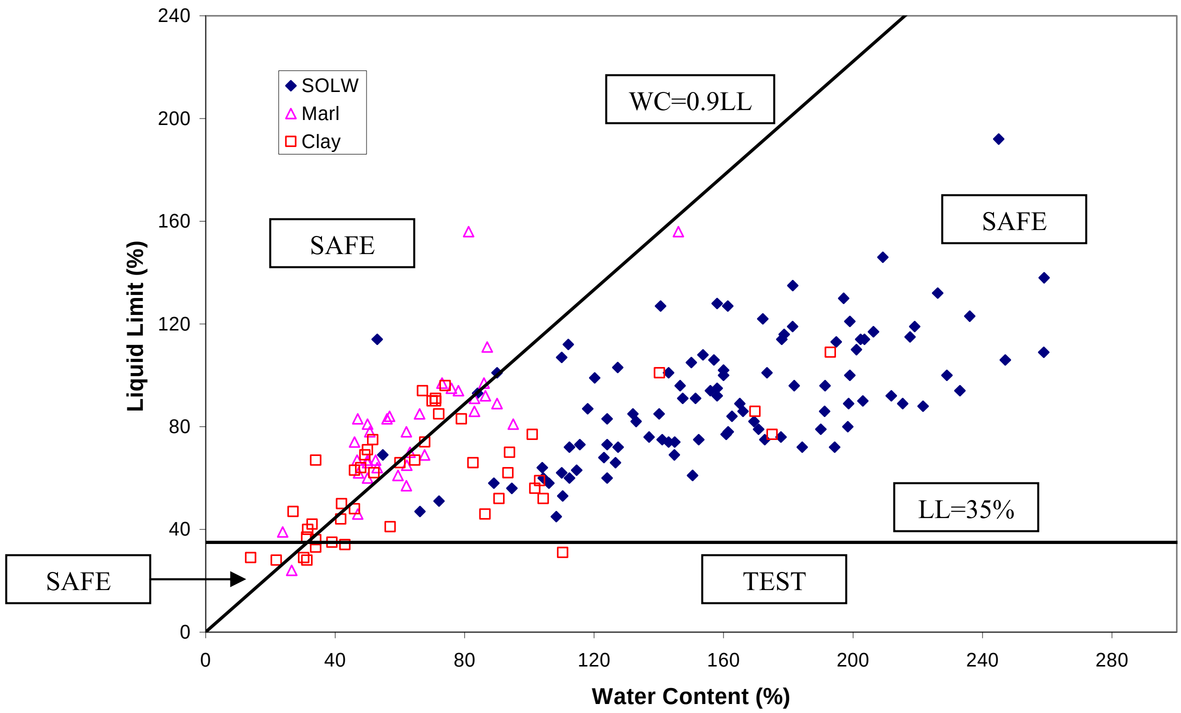

Figure 9. Application of Chinese criteria for SOLW, marl, and silt and clay.

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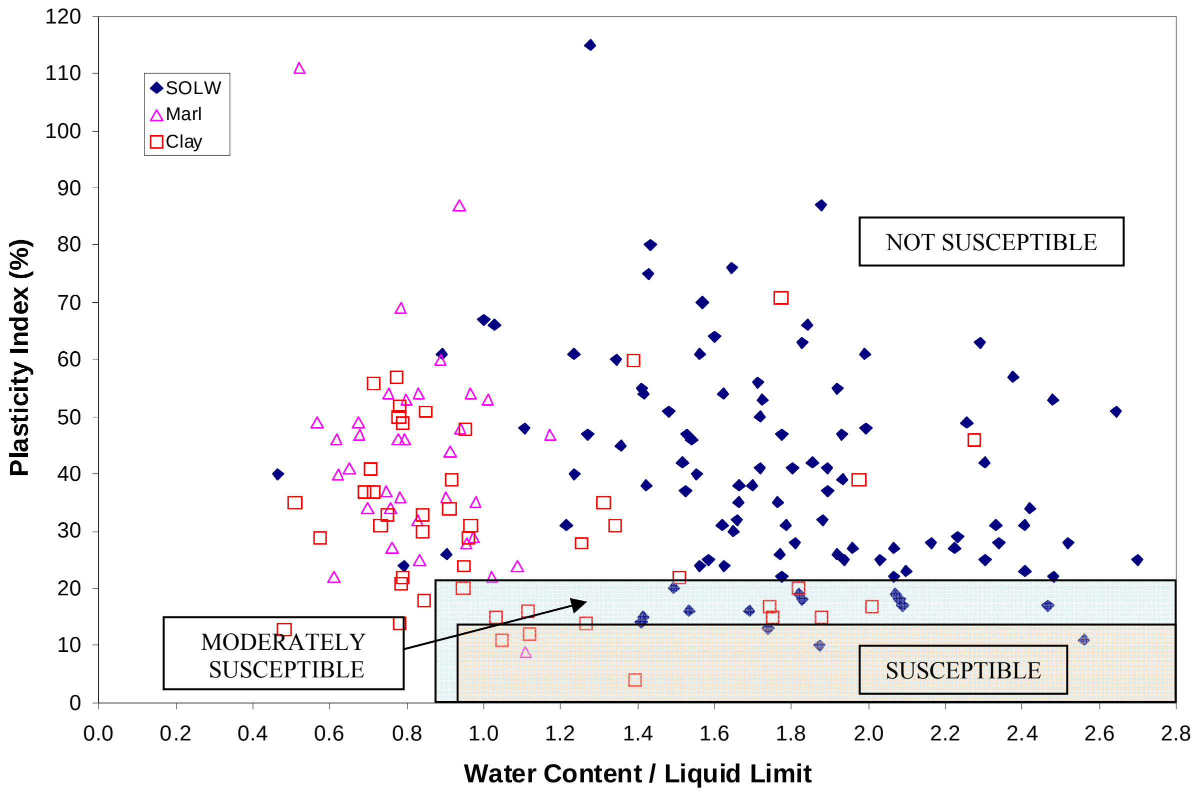

Figure 10. Application of the criteria proposed by Bray and Sancio [2006] for SOLW, marl, and silt and clay.

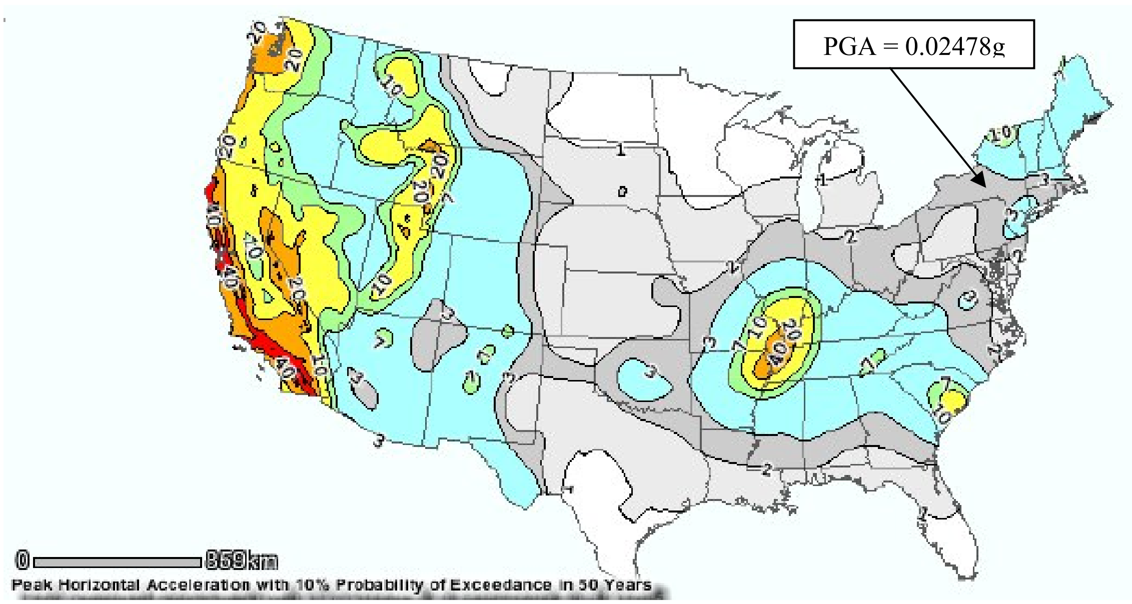

Figure 11. Peak ground acceleration with a 10% probability of exceedance in 50 years [USGS, 2008]. A latitude of 43° 04' N and a longitude of 76° 11' W were used to obtain the PGA value using the interactive maps from the USGS website.

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Client:	Honeywell	Project:	Onondaga Lake ILWD Stability	Project/ Proposal No.:	GD4014	Task No.:	02

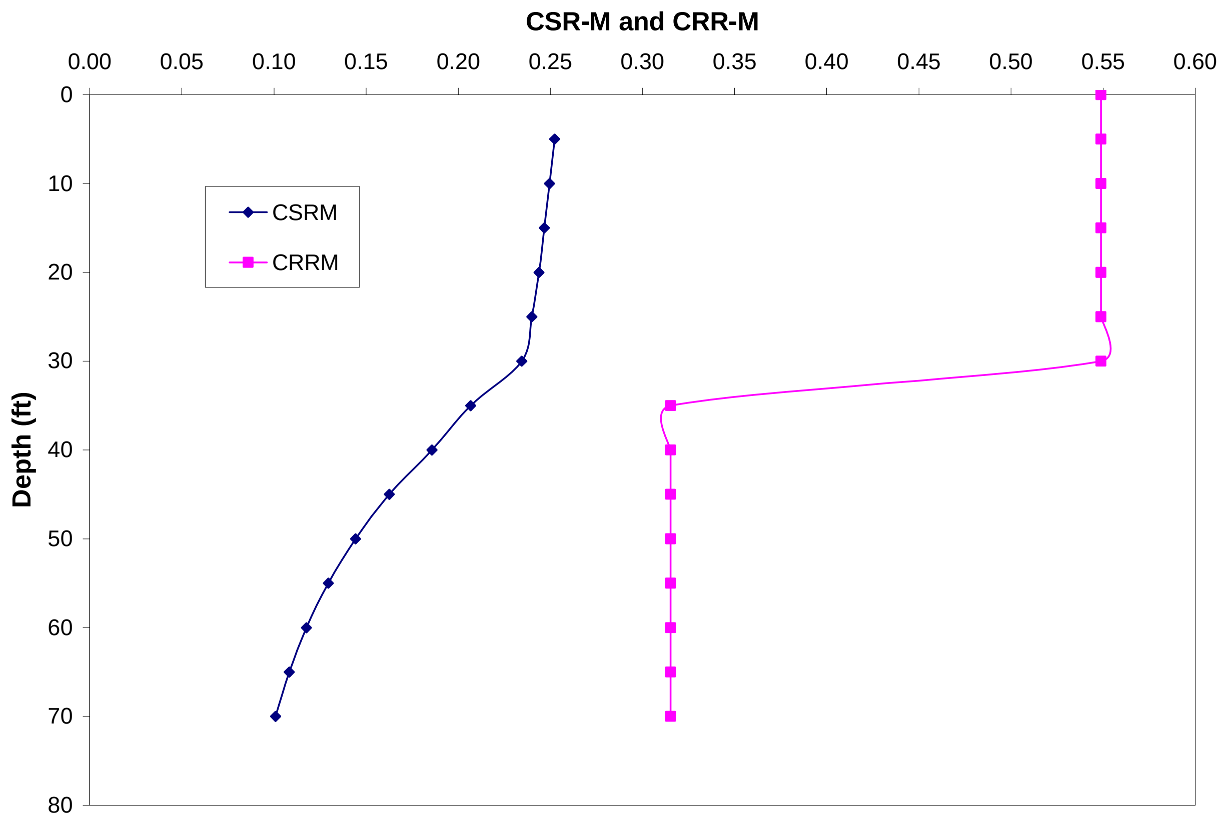

Figure 12. Calculated CSR and CRR values for SOLW marl, and silt and clay.

Note: ***The discontinuity in CRR_M values occurred at 30 ft because it is the interface between Solvay waste and Marl. Solvay waste shear strengths were modeled with an OCR of 2, and Marl was modeled with an OCR of 1. These OCR values are conservative.***

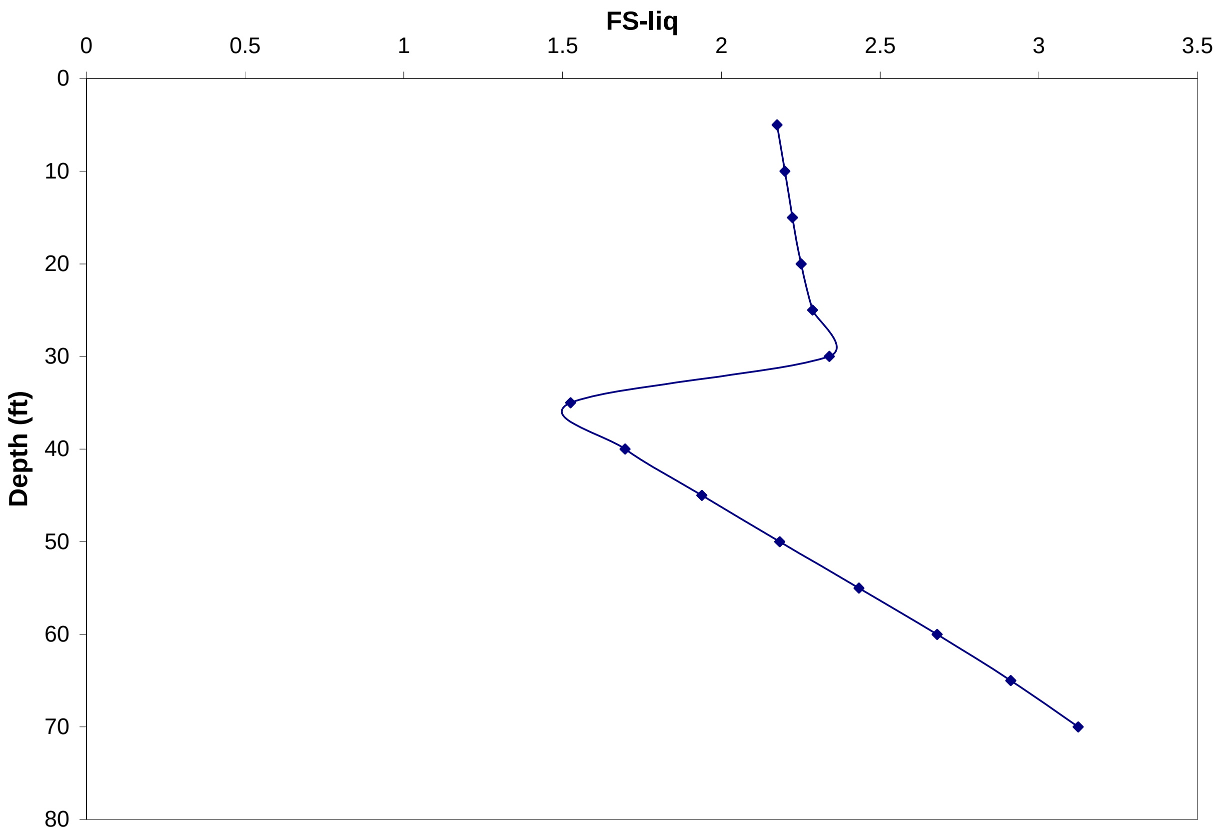

Figure 13. Calculated FS_{liq} values for SOLW marl, and silt and clay.

Note: ***The discontinuity in FS-liq values occurred at 30 ft because it is the interface between Solvay waste and Marl. Solvay waste shear strengths were modeled with an OCR of 2, and Marl was modeled with an OCR of 1. These OCR values are conservative.***

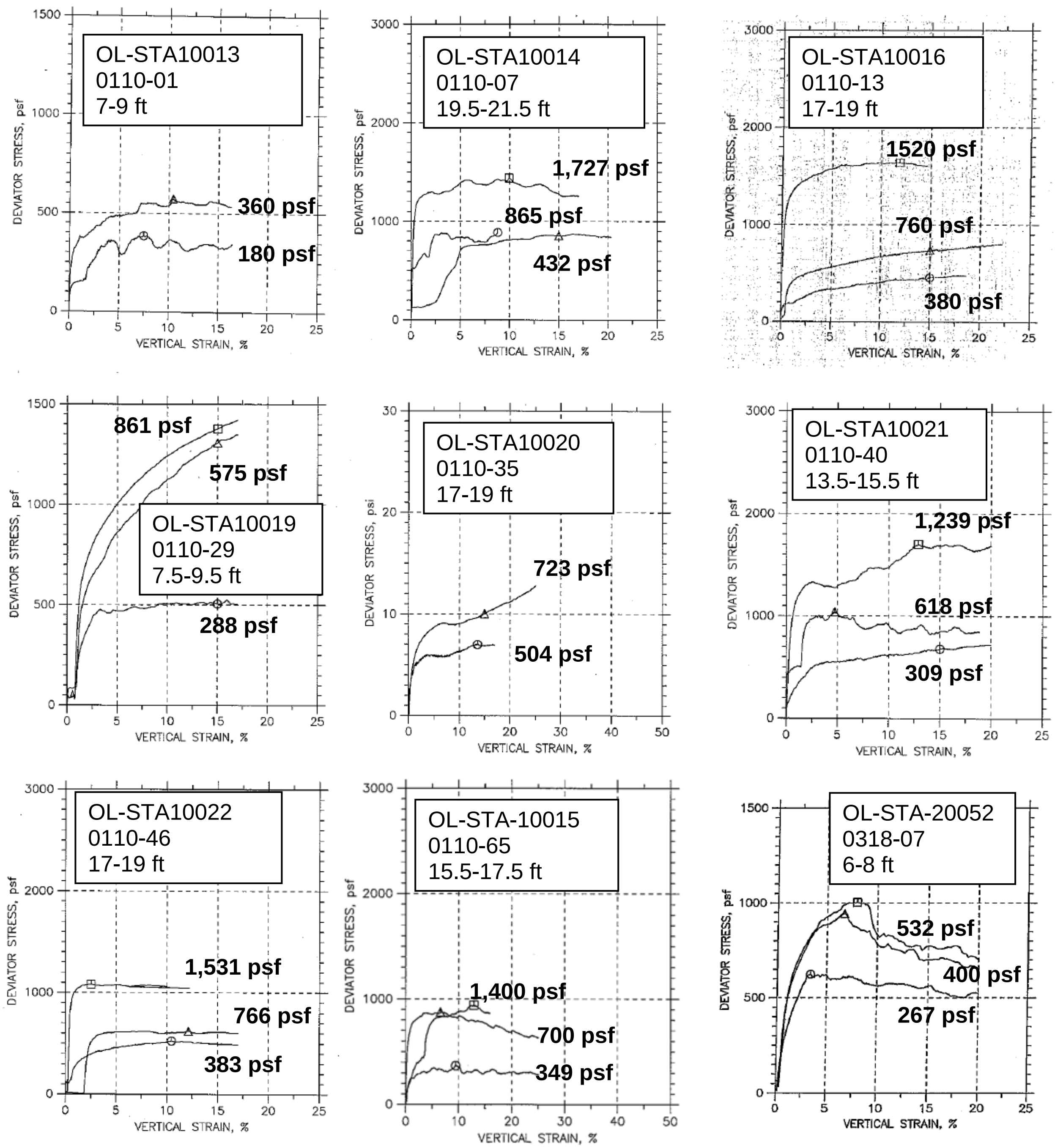

Figure 14. Consolidated undrained monotonic triaxial test stress-strain paths for SOLW (note that effective confining stresses applied in the lab are marked for each test).

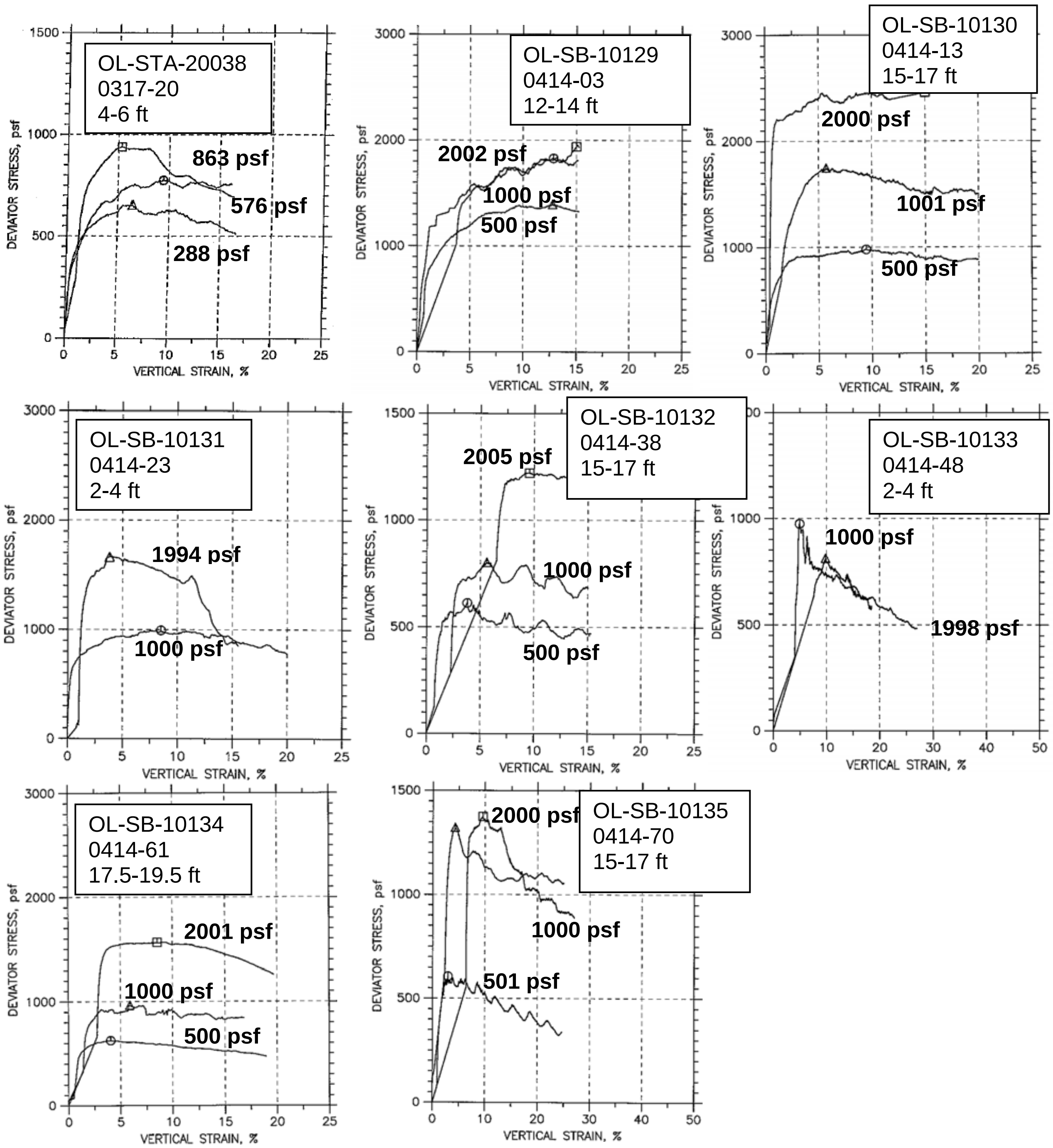

Figure 14. Consolidated undrained monotonic triaxial test stress-strain paths for SOLW (note that effective confining stresses applied in the lab are marked for each test) (continued).

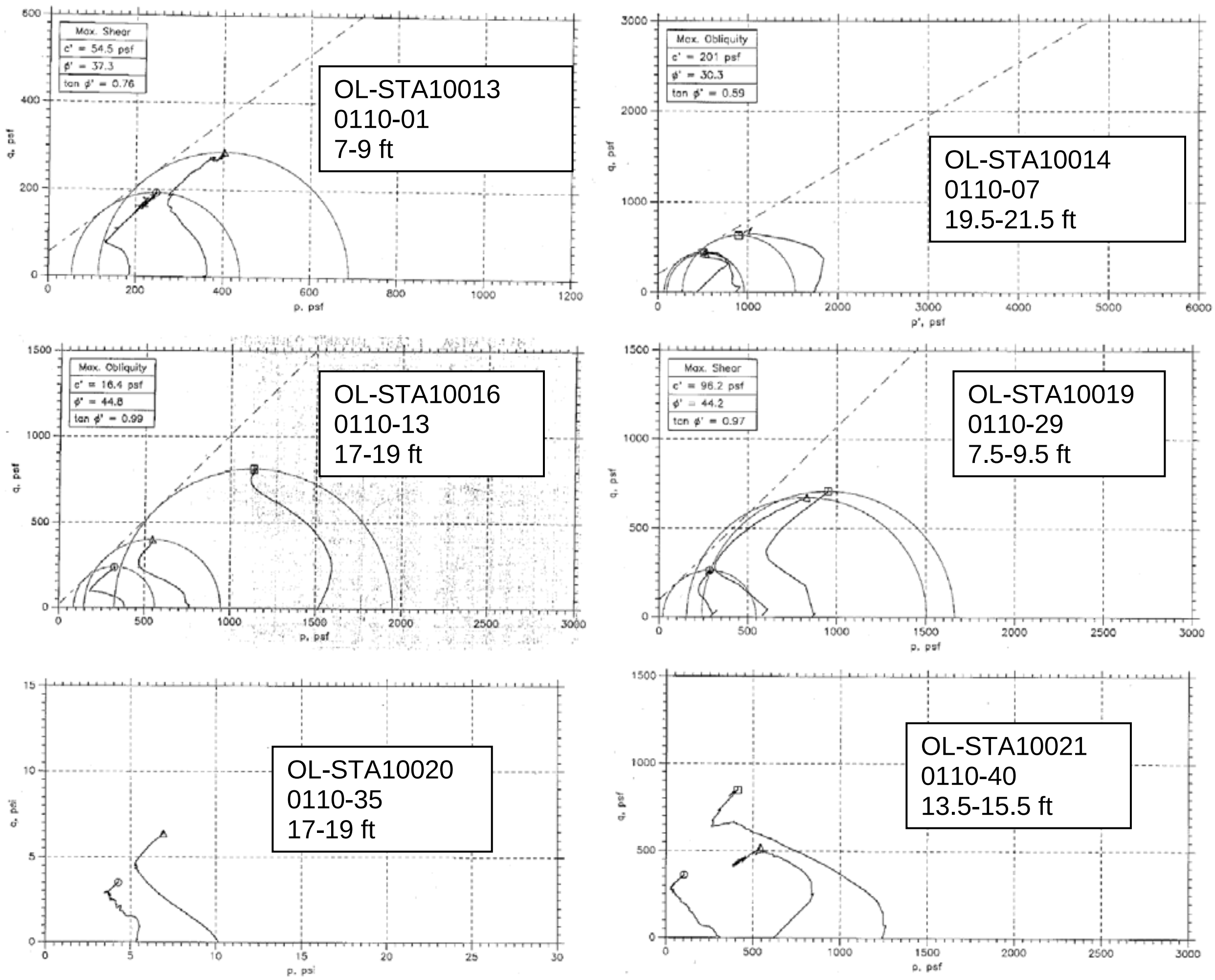

Figure 15. Consolidated undrained monotonic triaxial test q-p' stress paths for SOLW.

Written by: **Ming Zhu** Date: **3/27/2008** Reviewed by: **R. Kulasingam/Jay Beech** Date: **3/28/2008**

Client: **Honeywell** Project: **Onondaga Lake SCA 30% Design** Project/ Proposal No.: **GD3944** Task No.: **09**

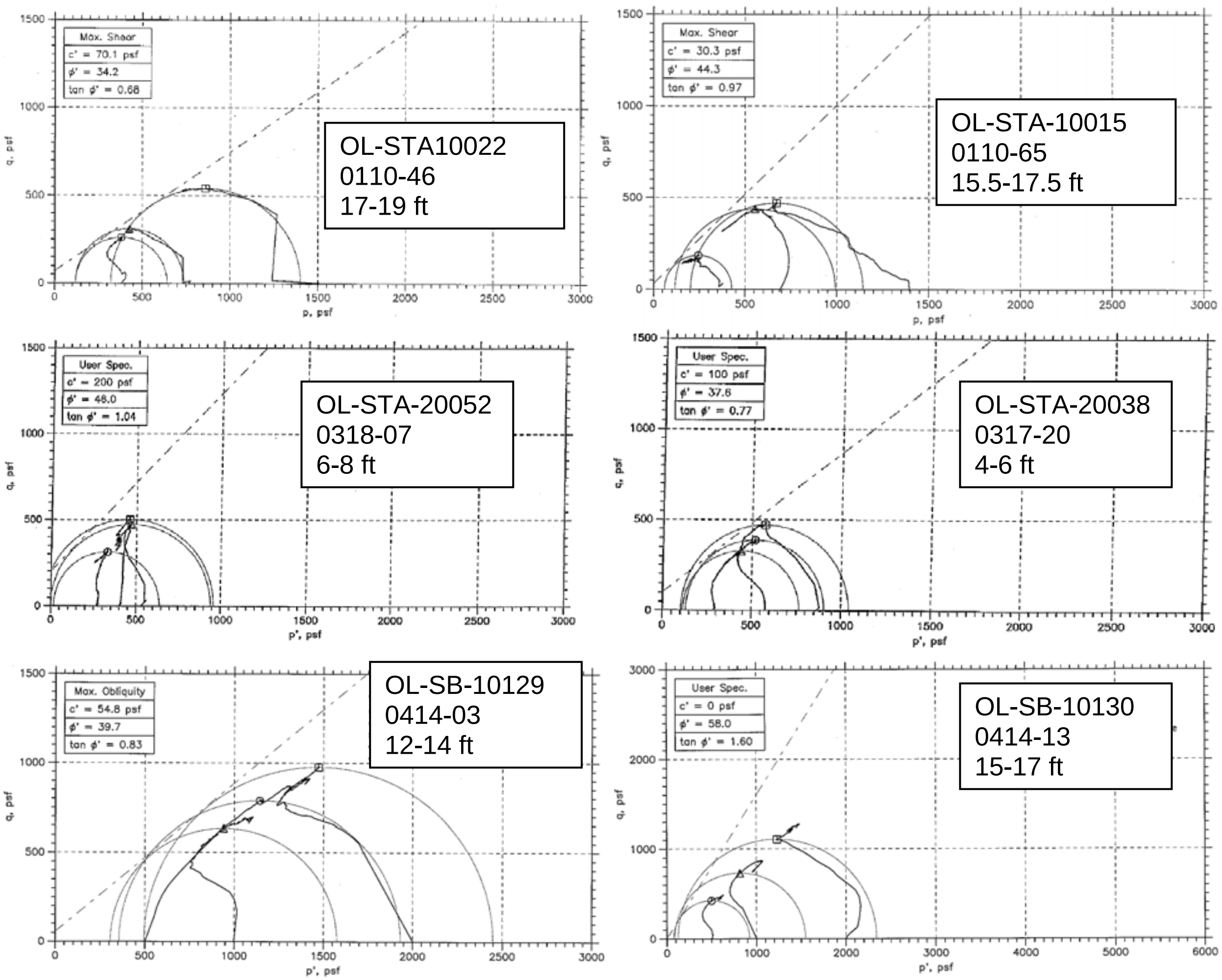

Figure 15. Consolidated undrained monotonic triaxial test q-p' stress paths for SOLW (continued).

Written by: **Ming Zhu** Date: **3/27/2008** Reviewed by: **R. Kulasingam/Jay Beech** Date: **3/28/2008**

Client: **Honeywell** Project: **Onondaga Lake SCA 30% Design** Project/ Proposal No.: **GD3944** Task No.: **09**

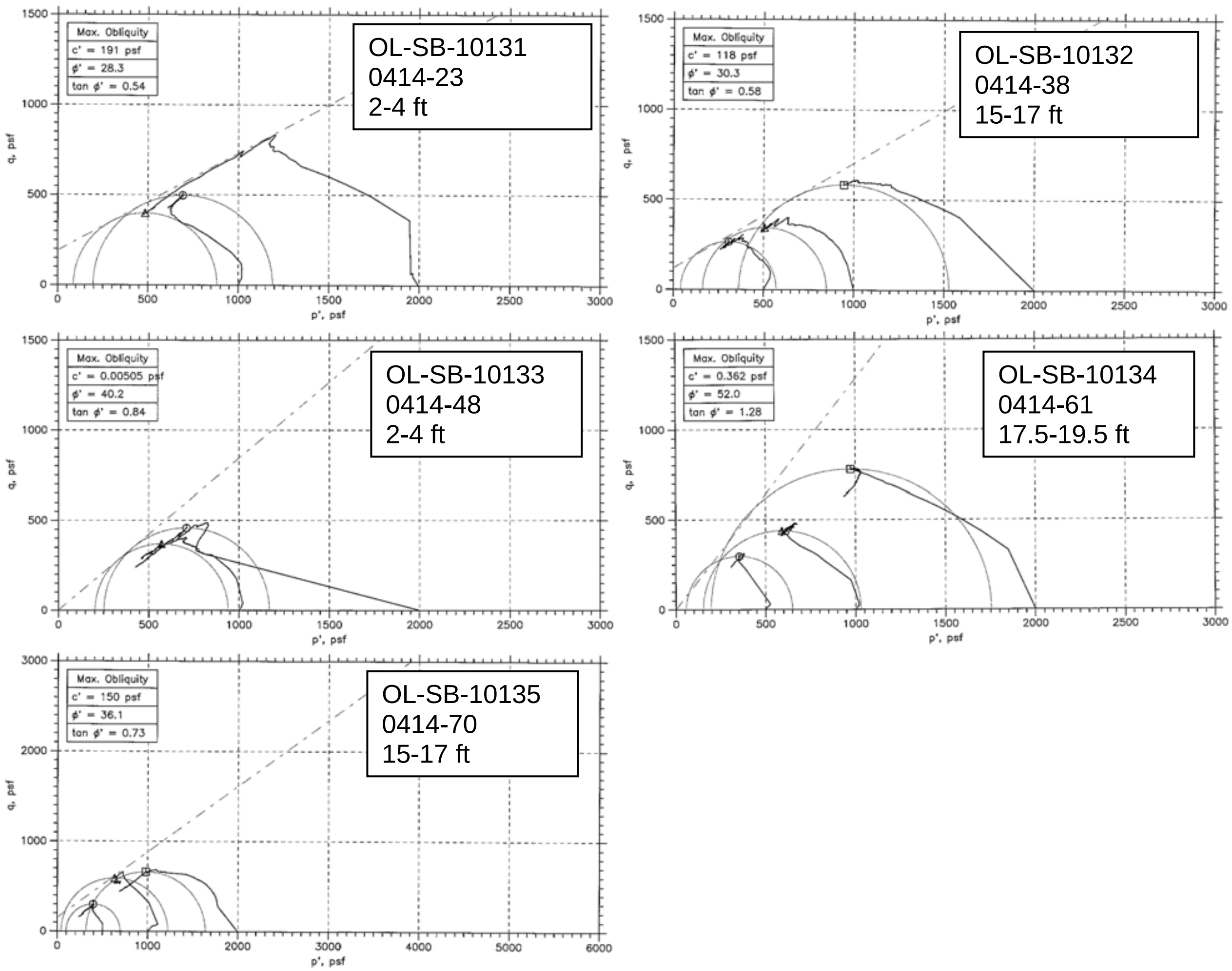

Figure 15. Consolidated undrained monotonic triaxial test q-p' stress paths for SOLW (continued).

Written by:	Ming Zhu	Date:	3/27/2008	Reviewed by:	R. Kulasingam/Jay Beech	Date:	3/28/2008
Client:	Honeywell	Project:	Onondaga Lake SCA 30% Design	Project/ Proposal No.:	GD3944	Task No.:	09

Attachment 1

Deaggregated Seismic Hazard

This attachment presents the deaggregated seismic hazard for the 10% probability of exceedance in a 50-year event, which is based on the 2002 United States Geological Survey (USGS) maps. The following paragraphs describe how a 5.3 moment magnitude was selected for the liquefaction analyses based on the deaggregation.

Conventional engineering methods for evaluation of soil liquefaction potential are deterministic. Required (deterministic) input parameters for evaluation of soil liquefaction include design earthquake magnitude, M (which is a proxy for duration of strong ground shaking) and free-field (zero-period) maximum ground surface acceleration (a_{max}).

In the Central and Eastern U.S., seismic hazard (and therefore a_{max}) is typically governed by multiple seismic sources at various distances. Probabilistic seismic hazard analysis is conducted in order to account for these multiple seismic source – distance pairs. The result of the probabilistic seismic hazard analysis is the a_{max} for a given return period. The contribution of each seismic source in evaluation of the a_{max} in a probabilistic seismic hazard analysis can be assessed by a process called deaggregation.

Deaggregation does not result in a single earthquake magnitude–distance pair. The result of deaggregation is a series of seismic hazard matrices (usually 3 to 5 matrices, each for a different period of oscillation, as available at the United States Geological Survey, USGS web site). For an evaluation of soil liquefaction potential, a_{max} is required. The value of a_{max} can be defined as the peak spectral acceleration (PSA) corresponding to a period of zero seconds. The PSA corresponding to a period of zero seconds is very close to the PSA corresponding to a period of 0.1 seconds. Hence, a 0.1-second matrix (matrix with the lowest period available from the USGS web page) is considered for this site. This is consistent with standard practice in seismic hazard evaluation.

A review of the 0.1-second matrix reveals the following three candidate magnitude-distance pairs:

M 4.81 at 36.3 km (Epsilon = 4.53)
M 5.25 at 115.3 km (Epsilon = 3.39)
M 5.72 at 164.7 km (Epsilon = 3.27)

These above-listed magnitude-distance pairs have the largest Epsilon and hence, dominate the a_{max} estimate for this site (zero Epsilon corresponds to the median motion; Epsilon = 2 corresponds to median plus one standard deviation motion; larger Epsilon=larger a_{max}).

An inspection of the above-listed candidate events indicates that the a_{max} evaluated for this site corresponds to M 4.81. However, M 4.81 at 36.3 km pair is associated with a relatively low duration of strong ground shaking (5.9 seconds as opposed to 18.5 seconds from the M 5.25 event at a distance of 115.3 km). Therefore, Geosyntec selected the highest evaluated a_{max} (i.e., a_{max} that corresponds to an M 4.81 event at 36.3 km) and duration (i.e., duration that corresponds to M 5.25 event) for an evaluation of soil liquefaction at this site. This is a conservative approach.

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*** Deaggregation of Seismic Hazard at Three Periods of Spectral Accel. ***
*** Data from U.S.G.S. National Seismic Hazards Mapping Project, 1996 version ***
PSHA Deaggregation. %contributions. site: Lake_Onondaga long: 76.20000 W., lat: 43.0600 N.
Return period: 475yrs. 2.00 s. PSA =0.0127989g. Computed annual rate=.21076E-02

DIST(KM)	MAG(MW)	ALL-EPS	EPSILON>2	1<EPS<2	0<EPS<1	-1<EPS<0	-2<EPS<-1	EPS<-2
13.1	4.87	0.121	0.037	0.065	0.019	0.001	0.000	0.000
13.7	5.27	0.250	0.022	0.091	0.106	0.029	0.001	0.000
35.4	5.30	0.262	0.044	0.134	0.082	0.002	0.000	0.000
64.1	5.33	0.163	0.062	0.094	0.007	0.000	0.000	0.000
89.6	5.34	0.090	0.054	0.036	0.000	0.000	0.000	0.000
114.5	5.35	0.115	0.087	0.028	0.000	0.000	0.000	0.000
138.7	5.35	0.101	0.090	0.011	0.000	0.000	0.000	0.000
164.8	5.36	0.133	0.130	0.003	0.000	0.000	0.000	0.000
190.1	5.36	0.082	0.082	0.000	0.000	0.000	0.000	0.000
210.1	5.36	0.076	0.076	0.000	0.000	0.000	0.000	0.000
234.6	5.37	0.093	0.093	0.000	0.000	0.000	0.000	0.000
259.7	5.37	0.057	0.057	0.000	0.000	0.000	0.000	0.000
284.5	5.37	0.067	0.067	0.000	0.000	0.000	0.000	0.000
13.6	5.71	0.212	0.006	0.037	0.091	0.067	0.010	0.000
36.1	5.74	0.522	0.027	0.155	0.255	0.082	0.002	0.000
65.2	5.76	0.514	0.051	0.226	0.219	0.018	0.000	0.000
90.0	5.77	0.418	0.060	0.233	0.125	0.000	0.000	0.000
110.3	5.78	0.474	0.083	0.292	0.099	0.000	0.000	0.000
135.6	5.78	0.974	0.210	0.633	0.130	0.000	0.000	0.000
165.3	5.79	1.135	0.311	0.760	0.064	0.000	0.000	0.000
190.3	5.79	0.816	0.275	0.521	0.021	0.000	0.000	0.000
210.3	5.80	0.841	0.330	0.510	0.002	0.000	0.000	0.000
234.9	5.80	1.164	0.525	0.639	0.000	0.000	0.000	0.000
259.8	5.81	0.805	0.413	0.392	0.000	0.000	0.000	0.000
284.8	5.81	1.048	0.621	0.427	0.000	0.000	0.000	0.000
309.9	5.81	0.662	0.434	0.228	0.000	0.000	0.000	0.000
334.7	5.82	0.828	0.593	0.235	0.000	0.000	0.000	0.000
364.3	5.82	0.662	0.536	0.126	0.000	0.000	0.000	0.000
389.7	5.82	0.361	0.318	0.043	0.000	0.000	0.000	0.000
410.0	5.82	0.278	0.251	0.026	0.000	0.000	0.000	0.000
434.5	5.83	0.315	0.296	0.019	0.000	0.000	0.000	0.000
464.3	5.83	0.207	0.204	0.002	0.000	0.000	0.000	0.000
489.2	5.83	0.102	0.102	0.000	0.000	0.000	0.000	0.000
509.7	5.84	0.072	0.072	0.000	0.000	0.000	0.000	0.000
534.1	5.84	0.073	0.073	0.000	0.000	0.000	0.000	0.000
13.9	6.21	0.111	0.003	0.016	0.039	0.039	0.014	0.001
37.2	6.23	0.414	0.011	0.068	0.170	0.142	0.022	0.000
65.9	6.24	0.596	0.021	0.127	0.297	0.145	0.005	0.000
90.3	6.25	0.608	0.025	0.150	0.331	0.102	0.000	0.000
110.4	6.25	0.777	0.035	0.207	0.439	0.096	0.000	0.000

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136.0	6.25	1.791	0.089	0.529	1.026	0.147	0.000	0.000
165.6	6.26	2.422	0.140	0.835	1.348	0.100	0.000	0.000
190.4	6.26	1.960	0.129	0.768	1.030	0.033	0.000	0.000
210.4	6.27	2.199	0.160	0.959	1.074	0.007	0.000	0.000
235.2	6.27	3.347	0.280	1.626	1.442	0.000	0.000	0.000
259.9	6.27	2.536	0.242	1.358	0.937	0.000	0.000	0.000
285.1	6.28	3.612	0.394	2.098	1.119	0.000	0.000	0.000
309.9	6.28	2.485	0.309	1.527	0.650	0.000	0.000	0.000
334.9	6.29	3.373	0.480	2.182	0.712	0.000	0.000	0.000
364.5	6.29	2.964	0.495	2.012	0.458	0.000	0.000	0.000
389.8	6.29	1.748	0.334	1.215	0.199	0.000	0.000	0.000
410.1	6.30	1.434	0.306	1.009	0.119	0.000	0.000	0.000
434.7	6.30	1.757	0.429	1.236	0.092	0.000	0.000	0.000
464.5	6.30	1.264	0.362	0.884	0.018	0.000	0.000	0.000
489.3	6.30	0.670	0.218	0.452	0.000	0.000	0.000	0.000
509.8	6.31	0.500	0.178	0.323	0.000	0.000	0.000	0.000
534.3	6.31	0.541	0.214	0.327	0.000	0.000	0.000	0.000
559.8	6.31	0.299	0.132	0.167	0.000	0.000	0.000	0.000
584.9	6.31	0.410	0.199	0.210	0.000	0.000	0.000	0.000
37.6	6.72	0.167	0.004	0.023	0.059	0.059	0.021	0.001
66.3	6.72	0.289	0.007	0.044	0.110	0.108	0.020	0.000
90.4	6.72	0.324	0.009	0.052	0.130	0.122	0.012	0.000
110.5	6.72	0.435	0.012	0.071	0.179	0.164	0.009	0.000
136.2	6.73	1.060	0.030	0.181	0.454	0.386	0.009	0.000
165.5	6.73	1.360	0.042	0.248	0.623	0.447	0.000	0.000
190.5	6.73	1.127	0.037	0.218	0.548	0.324	0.000	0.000
210.5	6.73	1.348	0.046	0.275	0.691	0.336	0.000	0.000
235.5	6.73	2.217	0.081	0.486	1.220	0.430	0.000	0.000
260.0	6.73	1.857	0.073	0.436	1.087	0.260	0.000	0.000
285.4	6.73	2.937	0.124	0.743	1.771	0.299	0.000	0.000
310.1	6.74	2.182	0.099	0.593	1.346	0.143	0.000	0.000
333.9	6.74	2.479	0.122	0.726	1.533	0.099	0.000	0.000
364.3	6.74	1.834	0.099	0.591	1.113	0.031	0.000	0.000
389.9	6.74	1.049	0.061	0.367	0.620	0.001	0.000	0.000
414.4	6.74	1.174	0.074	0.444	0.655	0.000	0.000	0.000
439.8	6.74	0.693	0.048	0.285	0.360	0.000	0.000	0.000
464.8	6.75	0.874	0.066	0.391	0.418	0.000	0.000	0.000
489.8	6.75	0.500	0.041	0.243	0.216	0.000	0.000	0.000
509.7	6.75	0.387	0.034	0.201	0.152	0.000	0.000	0.000
534.3	6.75	0.409	0.039	0.233	0.136	0.000	0.000	0.000
559.9	6.75	0.248	0.026	0.154	0.068	0.000	0.000	0.000
585.3	6.75	0.354	0.041	0.232	0.082	0.000	0.000	0.000
37.7	7.20	0.100	0.002	0.014	0.034	0.034	0.014	0.002
66.4	7.21	0.184	0.004	0.026	0.064	0.064	0.024	0.001
90.4	7.21	0.214	0.005	0.030	0.076	0.076	0.026	0.000

Written by: **Ming Zhu** Date: **3/27/2008** Reviewed by: **R. Kulasingam/Jay Beech** Date: **3/28/2008**

Client: **Honeywell** Project: **Onondaga Lake SCA 30% Design** Project/ Proposal No.: **GD3944** Task No.: **09**

110.5	7.21	0.291	0.007	0.042	0.105	0.105	0.033	0.000
136.3	7.21	0.729	0.018	0.106	0.266	0.266	0.074	0.000
165.5	7.21	0.975	0.024	0.145	0.365	0.365	0.076	0.000
190.6	7.21	0.836	0.021	0.128	0.321	0.320	0.046	0.000
210.6	7.21	1.031	0.027	0.161	0.405	0.395	0.042	0.000
235.6	7.21	1.764	0.048	0.284	0.714	0.671	0.047	0.000
260.0	7.22	1.535	0.043	0.255	0.641	0.569	0.026	0.000
285.5	7.22	2.524	0.073	0.435	1.093	0.897	0.026	0.000
310.1	7.22	1.945	0.058	0.347	0.872	0.656	0.011	0.000
334.0	7.22	2.292	0.071	0.425	1.067	0.722	0.007	0.000
364.4	7.22	1.776	0.058	0.346	0.869	0.502	0.001	0.000
390.0	7.22	1.043	0.036	0.212	0.533	0.262	0.000	0.000
414.5	7.22	1.202	0.043	0.256	0.642	0.261	0.000	0.000
439.9	7.23	0.741	0.028	0.165	0.415	0.134	0.000	0.000
464.9	7.23	0.976	0.038	0.228	0.564	0.145	0.000	0.000
489.8	7.23	0.579	0.024	0.142	0.342	0.071	0.000	0.000
509.6	7.23	0.454	0.019	0.116	0.271	0.047	0.000	0.000
534.3	7.22	0.470	0.021	0.127	0.285	0.037	0.000	0.000
560.0	7.22	0.296	0.014	0.085	0.180	0.017	0.000	0.000
585.5	7.23	0.459	0.023	0.139	0.274	0.023	0.000	0.000

Summary statistics for above 2.0s PSA deaggregation, R=distance, e=epsilon:
Mean src-site R= 289.7 km; M= 6.53; e0= 0.26; e= 1.05 for all sources.
Modal src-site R= 285.1 km; M= 6.28; e0= 0.71 from peak (R,M) bin
Primary distance metric: EPICENTRAL
MODE R*= 335.0km; M*= 6.29; EPS.INTERVAL: 1 to 2 sigma % CONTRIB.= 2.182

Principal sources (faults, subduction, random seismicity having >10% contribution)

Source:	% contr.	R(km)	M	epsilon0 (mean values)
CEUS gridded seismicity,Frankel	57.12	295.0	6.58	0.18
CEUS gridded seismicity,Toro att	42.88	282.7	6.45	0.36

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PSHA Deaggregation. %contributions. site: Lake_Onondaga long: 76.20000 W., lat: 43.0600 N.
Return period: 475yrs. 0.50 s. PSA =0.0482607g. Computed annual rate=.21104E-02

DIST(KM)	MAG(MW)	ALL-EPS	EPSILON>2	1<EPS<2	0<EPS<1	-1<EPS<0	-2<EPS<-1	EPS<-2
12.9	4.83	0.834	0.034	0.205	0.429	0.161	0.005	0.000
34.1	4.85	1.073	0.150	0.614	0.305	0.005	0.000	0.000
63.5	4.87	0.444	0.224	0.220	0.000	0.000	0.000	0.000
89.6	4.88	0.219	0.178	0.041	0.000	0.000	0.000	0.000
115.1	4.89	0.296	0.279	0.017	0.000	0.000	0.000	0.000
139.9	4.89	0.185	0.185	0.000	0.000	0.000	0.000	0.000
164.1	4.90	0.217	0.217	0.000	0.000	0.000	0.000	0.000
189.8	4.90	0.105	0.105	0.000	0.000	0.000	0.000	0.000
209.8	4.90	0.079	0.079	0.000	0.000	0.000	0.000	0.000
233.8	4.91	0.073	0.073	0.000	0.000	0.000	0.000	0.000

Written by: **Ming Zhu** Date: **3/27/2008** Reviewed by: **R. Kulasingam/Jay Beech** Date: **3/28/2008**

Client: **Honeywell** Project: **Onondaga Lake SCA 30% Design** Project/ Proposal No.: **GD3944** Task No.: **09**

13.7	5.22	0.575	0.015	0.089	0.224	0.211	0.035	0.000
35.8	5.24	1.420	0.065	0.390	0.780	0.184	0.001	0.000
64.6	5.26	1.108	0.123	0.642	0.342	0.001	0.000	0.000
89.9	5.27	0.768	0.145	0.552	0.072	0.000	0.000	0.000
112.3	5.28	0.923	0.254	0.650	0.020	0.000	0.000	0.000
136.3	5.28	1.225	0.442	0.783	0.000	0.000	0.000	0.000
164.6	5.29	1.264	0.691	0.573	0.000	0.000	0.000	0.000
190.0	5.29	0.730	0.525	0.205	0.000	0.000	0.000	0.000
210.0	5.30	0.627	0.528	0.099	0.000	0.000	0.000	0.000
234.3	5.30	0.691	0.655	0.036	0.000	0.000	0.000	0.000
260.8	5.31	0.414	0.414	0.000	0.000	0.000	0.000	0.000
285.1	5.31	0.345	0.345	0.000	0.000	0.000	0.000	0.000
309.5	5.31	0.188	0.188	0.000	0.000	0.000	0.000	0.000
333.9	5.32	0.184	0.184	0.000	0.000	0.000	0.000	0.000
363.6	5.32	0.109	0.109	0.000	0.000	0.000	0.000	0.000
13.9	5.70	0.268	0.006	0.037	0.093	0.093	0.035	0.002
37.0	5.70	0.955	0.027	0.162	0.408	0.328	0.029	0.000
65.4	5.72	1.170	0.051	0.305	0.672	0.142	0.000	0.000
90.1	5.73	1.029	0.060	0.360	0.590	0.019	0.000	0.000
110.2	5.73	1.184	0.083	0.495	0.606	0.000	0.000	0.000
135.6	5.73	2.368	0.212	1.248	0.908	0.000	0.000	0.000
165.1	5.74	2.650	0.335	1.736	0.579	0.000	0.000	0.000
190.2	5.75	1.781	0.308	1.287	0.186	0.000	0.000	0.000
210.2	5.75	1.717	0.384	1.270	0.063	0.000	0.000	0.000
234.6	5.75	2.164	0.670	1.490	0.004	0.000	0.000	0.000
259.6	5.76	1.349	0.571	0.779	0.000	0.000	0.000	0.000
284.5	5.76	1.564	0.865	0.699	0.000	0.000	0.000	0.000
309.7	5.77	0.877	0.594	0.283	0.000	0.000	0.000	0.000
334.2	5.77	0.956	0.764	0.192	0.000	0.000	0.000	0.000
363.9	5.77	0.640	0.594	0.046	0.000	0.000	0.000	0.000
389.5	5.78	0.299	0.297	0.001	0.000	0.000	0.000	0.000
409.8	5.78	0.202	0.202	0.000	0.000	0.000	0.000	0.000
434.1	5.78	0.195	0.195	0.000	0.000	0.000	0.000	0.000
463.9	5.79	0.105	0.105	0.000	0.000	0.000	0.000	0.000
14.0	6.21	0.114	0.003	0.015	0.039	0.039	0.015	0.002
37.5	6.22	0.471	0.011	0.068	0.170	0.170	0.051	0.001
66.0	6.22	0.749	0.021	0.127	0.319	0.265	0.016	0.000
90.3	6.23	0.774	0.025	0.150	0.377	0.221	0.001	0.000
110.4	6.23	0.974	0.035	0.206	0.518	0.214	0.000	0.000
135.9	6.23	2.181	0.088	0.529	1.274	0.291	0.000	0.000
165.5	6.24	2.808	0.140	0.833	1.693	0.142	0.000	0.000
190.3	6.24	2.146	0.128	0.767	1.226	0.025	0.000	0.000
210.3	6.25	2.286	0.160	0.957	1.168	0.001	0.000	0.000
235.0	6.25	3.243	0.279	1.663	1.300	0.000	0.000	0.000
259.8	6.25	2.269	0.242	1.384	0.644	0.000	0.000	0.000

Written by: **Ming Zhu** Date: **3/27/2008** Reviewed by: **R. Kulasingam/Jay Beech** Date: **3/28/2008**

Client: **Honeywell** Project: **Onondaga Lake SCA 30% Design** Project/ Proposal No.: **GD3944** Task No.: **09**

284.8	6.26	2.944	0.394	2.012	0.539	0.000	0.000	0.000
309.8	6.26	1.837	0.308	1.339	0.190	0.000	0.000	0.000
334.6	6.27	2.229	0.479	1.658	0.092	0.000	0.000	0.000
364.1	6.27	1.696	0.495	1.195	0.007	0.000	0.000	0.000
389.6	6.28	0.880	0.333	0.547	0.000	0.000	0.000	0.000
409.9	6.28	0.646	0.298	0.348	0.000	0.000	0.000	0.000
434.3	6.28	0.686	0.393	0.294	0.000	0.000	0.000	0.000
464.1	6.29	0.414	0.289	0.125	0.000	0.000	0.000	0.000
489.2	6.29	0.189	0.149	0.040	0.000	0.000	0.000	0.000
509.6	6.29	0.124	0.107	0.016	0.000	0.000	0.000	0.000
533.8	6.29	0.113	0.105	0.008	0.000	0.000	0.000	0.000
560.9	6.30	0.057	0.057	0.001	0.000	0.000	0.000	0.000
585.2	6.30	0.055	0.055	0.000	0.000	0.000	0.000	0.000
37.7	6.72	0.169	0.004	0.023	0.059	0.058	0.023	0.002
66.3	6.72	0.301	0.007	0.044	0.110	0.110	0.030	0.000
90.4	6.72	0.337	0.009	0.052	0.130	0.129	0.018	0.000
110.5	6.72	0.446	0.012	0.071	0.178	0.173	0.012	0.000
136.1	6.72	1.065	0.030	0.181	0.454	0.393	0.008	0.000
165.4	6.73	1.321	0.042	0.248	0.622	0.410	0.000	0.000
190.4	6.73	1.048	0.036	0.218	0.548	0.246	0.000	0.000
210.5	6.73	1.207	0.046	0.275	0.687	0.199	0.000	0.000
235.4	6.73	1.890	0.081	0.485	1.156	0.168	0.000	0.000
264.7	6.73	2.197	0.110	0.660	1.356	0.071	0.000	0.000
290.0	6.73	1.484	0.087	0.518	0.869	0.010	0.000	0.000
310.0	6.74	1.499	0.099	0.592	0.807	0.000	0.000	0.000
333.7	6.74	1.562	0.121	0.725	0.715	0.000	0.000	0.000
363.9	6.74	1.025	0.099	0.583	0.343	0.000	0.000	0.000
389.8	6.74	0.527	0.061	0.347	0.118	0.000	0.000	0.000
414.1	6.74	0.528	0.074	0.378	0.075	0.000	0.000	0.000
439.7	6.75	0.276	0.048	0.208	0.020	0.000	0.000	0.000
464.5	6.75	0.307	0.065	0.232	0.010	0.000	0.000	0.000
489.6	6.75	0.155	0.041	0.114	0.000	0.000	0.000	0.000
509.5	6.74	0.108	0.034	0.074	0.000	0.000	0.000	0.000
533.9	6.75	0.099	0.039	0.060	0.000	0.000	0.000	0.000
559.7	6.75	0.052	0.025	0.027	0.000	0.000	0.000	0.000
584.8	6.75	0.064	0.037	0.026	0.000	0.000	0.000	0.000
37.7	7.20	0.100	0.002	0.014	0.034	0.034	0.014	0.002
66.4	7.21	0.185	0.004	0.026	0.064	0.064	0.025	0.001
90.4	7.21	0.214	0.005	0.030	0.076	0.076	0.027	0.000
110.5	7.21	0.290	0.007	0.042	0.104	0.104	0.033	0.000
136.2	7.21	0.718	0.018	0.106	0.266	0.266	0.063	0.000
165.5	7.21	0.942	0.024	0.145	0.364	0.359	0.049	0.000
190.5	7.21	0.789	0.021	0.128	0.321	0.297	0.022	0.000
210.5	7.21	0.949	0.027	0.161	0.404	0.341	0.016	0.000
235.5	7.22	1.568	0.048	0.284	0.713	0.511	0.011	0.000

261.9	7.22	1.501	0.050	0.301	0.755	0.393	0.002	0.000
286.4	7.22	1.859	0.065	0.389	0.969	0.436	0.000	0.000
310.0	7.22	1.498	0.058	0.347	0.840	0.253	0.000	0.000
333.8	7.23	1.663	0.071	0.424	0.969	0.199	0.000	0.000
364.2	7.23	1.184	0.058	0.346	0.692	0.089	0.000	0.000
389.9	7.23	0.645	0.035	0.212	0.367	0.031	0.000	0.000
414.3	7.24	0.687	0.043	0.255	0.372	0.017	0.000	0.000
439.8	7.24	0.388	0.028	0.165	0.192	0.003	0.000	0.000
464.7	7.24	0.467	0.038	0.223	0.207	0.000	0.000	0.000
489.7	7.25	0.253	0.024	0.132	0.097	0.000	0.000	0.000
509.5	7.25	0.184	0.019	0.103	0.061	0.000	0.000	0.000
533.9	7.25	0.171	0.021	0.105	0.045	0.000	0.000	0.000
559.8	7.25	0.096	0.014	0.063	0.020	0.000	0.000	0.000
585.2	7.26	0.132	0.023	0.089	0.020	0.000	0.000	0.000

Summary statistics for above 0.5s PSA deaggregation, R=distance, e=epsilon:
Mean src-site R= 229.7 km; M= 6.23; e0= 0.49; e= 1.22 for all sources.
Modal src-site R= 235.0 km; M= 6.25; e0= 0.58 from peak (R,M) bin
Primary distance metric: EPICENTRAL
MODE R*= 285.1km; M*= 6.26; EPS.INTERVAL: 1 to 2 sigma % CONTRIB.= 2.012

Principal sources (faults, subduction, random seismicity having >10% contribution)

Source:	% contr.	R(km)	M	epsilon0 (mean values)
CEUS gridded seismicity,Frankel	66.19	237.1	6.24	0.47
CEUS gridded seismicity,Toro att	33.81	215.3	6.22	0.53

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PSHA Deaggregation. %contributions. site: Lake_Onondaga long: 76.20000 W., lat: 43.0600 N.
Return period: 475yrs. 0.10 s. PSA =0.0709094g. Computed annual rate=.21080E-02

DIST(KM)	MAG(MW)	ALL-EPS	EPSILON>2	1<EPS<2	0<EPS<1	-1<EPS<0	-2<EPS<-1	EPS<-2
13.9	4.81	1.461	0.034	0.206	0.516	0.516	0.180	0.009
36.3	4.81	4.513	0.150	0.898	2.228	1.182	0.055	0.000
64.4	4.82	3.813	0.282	1.679	1.826	0.026	0.000	0.000
89.6	4.83	2.239	0.333	1.618	0.288	0.000	0.000	0.000
114.6	4.84	2.696	0.774	1.903	0.020	0.000	0.000	0.000
139.7	4.85	1.451	0.733	0.719	0.000	0.000	0.000	0.000
163.5	4.86	1.444	1.142	0.302	0.000	0.000	0.000	0.000
189.5	4.86	0.555	0.552	0.003	0.000	0.000	0.000	0.000
209.4	4.87	0.339	0.339	0.000	0.000	0.000	0.000	0.000
233.0	4.87	0.242	0.242	0.000	0.000	0.000	0.000	0.000
262.2	4.88	0.103	0.103	0.000	0.000	0.000	0.000	0.000
14.0	5.22	0.653	0.015	0.089	0.224	0.224	0.089	0.011
37.1	5.22	2.464	0.065	0.390	0.980	0.902	0.126	0.001
65.2	5.23	2.919	0.123	0.733	1.686	0.377	0.000	0.000
89.9	5.24	2.241	0.145	0.864	1.229	0.004	0.000	0.000
115.2	5.25	3.397	0.339	1.960	1.098	0.000	0.000	0.000

Written by: **Ming Zhu** Date: **3/27/2008** Reviewed by: **R. Kulasingam/Jay Beech** Date: **3/28/2008**

Client: **Honeywell** Project: **Onondaga Lake SCA 30% Design** Project/ Proposal No.: **GD3944** Task No.: **09**

140.0	5.25	2.241	0.370	1.643	0.229	0.000	0.000	0.000
164.1	5.26	2.694	0.802	1.863	0.028	0.000	0.000	0.000
189.8	5.26	1.277	0.693	0.584	0.000	0.000	0.000	0.000
209.7	5.27	0.910	0.699	0.211	0.000	0.000	0.000	0.000
233.5	5.28	0.774	0.734	0.040	0.000	0.000	0.000	0.000
262.7	5.28	0.407	0.407	0.000	0.000	0.000	0.000	0.000
289.0	5.29	0.132	0.132	0.000	0.000	0.000	0.000	0.000
309.0	5.29	0.077	0.077	0.000	0.000	0.000	0.000	0.000
332.7	5.30	0.051	0.051	0.000	0.000	0.000	0.000	0.000
14.0	5.70	0.274	0.006	0.037	0.093	0.093	0.037	0.006
37.5	5.70	1.141	0.027	0.163	0.408	0.408	0.130	0.004
65.8	5.70	1.728	0.051	0.305	0.767	0.585	0.020	0.000
90.1	5.71	1.613	0.060	0.360	0.901	0.291	0.000	0.000
112.6	5.72	2.169	0.106	0.633	1.303	0.128	0.000	0.000
136.5	5.71	3.005	0.190	1.132	1.642	0.041	0.000	0.000
164.7	5.72	3.267	0.335	1.881	1.052	0.000	0.000	0.000
189.9	5.73	1.872	0.308	1.348	0.217	0.000	0.000	0.000
209.9	5.73	1.549	0.385	1.125	0.039	0.000	0.000	0.000
234.0	5.74	1.578	0.650	0.929	0.000	0.000	0.000	0.000
263.2	5.75	1.025	0.694	0.331	0.000	0.000	0.000	0.000
289.3	5.75	0.399	0.352	0.046	0.000	0.000	0.000	0.000
309.2	5.76	0.268	0.264	0.004	0.000	0.000	0.000	0.000
333.2	5.76	0.208	0.208	0.000	0.000	0.000	0.000	0.000
362.8	5.77	0.089	0.089	0.000	0.000	0.000	0.000	0.000
14.0	6.21	0.114	0.003	0.016	0.039	0.039	0.016	0.003
37.7	6.21	0.493	0.011	0.068	0.170	0.170	0.067	0.006
66.2	6.22	0.859	0.021	0.127	0.320	0.319	0.071	0.000
90.3	6.22	0.913	0.025	0.150	0.377	0.344	0.017	0.000
110.3	6.22	1.133	0.035	0.207	0.519	0.369	0.004	0.000
135.8	6.23	2.414	0.089	0.529	1.296	0.500	0.000	0.000
165.2	6.23	2.835	0.140	0.834	1.666	0.195	0.000	0.000
190.2	6.24	1.938	0.128	0.768	1.022	0.020	0.000	0.000
210.1	6.24	1.842	0.160	0.925	0.757	0.000	0.000	0.000
234.5	6.25	2.210	0.280	1.398	0.532	0.000	0.000	0.000
260.6	6.26	1.372	0.290	0.952	0.130	0.000	0.000	0.000
284.9	6.25	1.198	0.342	0.833	0.023	0.000	0.000	0.000
309.5	6.26	0.629	0.283	0.346	0.000	0.000	0.000	0.000
333.7	6.27	0.579	0.371	0.208	0.000	0.000	0.000	0.000
363.3	6.28	0.309	0.259	0.050	0.000	0.000	0.000	0.000
389.2	6.29	0.117	0.114	0.003	0.000	0.000	0.000	0.000
409.5	6.29	0.066	0.066	0.000	0.000	0.000	0.000	0.000
433.4	6.30	0.051	0.051	0.000	0.000	0.000	0.000	0.000
37.7	6.72	0.171	0.004	0.023	0.059	0.059	0.023	0.004
66.4	6.72	0.315	0.007	0.044	0.110	0.110	0.042	0.002
90.4	6.72	0.358	0.009	0.052	0.130	0.130	0.038	0.000

Written by: **Ming Zhu** Date: **3/27/2008** Reviewed by: **R. Kulasingam/Jay Beech** Date: **3/28/2008**

Client: **Honeywell** Project: **Onondaga Lake SCA 30% Design** Project/ Proposal No.: **GD3944** Task No.: **09**

110.4	6.72	0.470	0.012	0.071	0.179	0.177	0.031	0.000
136.0	6.72	1.092	0.030	0.181	0.454	0.400	0.027	0.000
165.2	6.72	1.281	0.042	0.248	0.622	0.367	0.002	0.000
190.3	6.73	0.945	0.037	0.218	0.518	0.172	0.000	0.000
210.3	6.73	1.004	0.046	0.275	0.581	0.102	0.000	0.000
235.0	6.73	1.385	0.081	0.486	0.779	0.039	0.000	0.000
264.2	6.73	1.345	0.111	0.636	0.599	0.000	0.000	0.000
289.8	6.73	0.758	0.087	0.442	0.230	0.000	0.000	0.000
309.8	6.73	0.656	0.099	0.439	0.118	0.000	0.000	0.000
333.0	6.74	0.557	0.121	0.400	0.035	0.000	0.000	0.000
363.3	6.74	0.274	0.094	0.180	0.000	0.000	0.000	0.000
389.5	6.75	0.109	0.051	0.058	0.000	0.000	0.000	0.000
413.3	6.75	0.084	0.054	0.031	0.000	0.000	0.000	0.000
37.7	7.20	0.100	0.002	0.014	0.034	0.034	0.014	0.002
66.4	7.20	0.188	0.004	0.026	0.064	0.064	0.026	0.003
90.4	7.21	0.219	0.005	0.030	0.076	0.076	0.030	0.001
110.5	7.21	0.296	0.007	0.042	0.105	0.105	0.037	0.001
136.2	7.21	0.723	0.018	0.106	0.266	0.265	0.068	0.000
165.4	7.21	0.919	0.024	0.145	0.365	0.336	0.049	0.000
190.5	7.21	0.738	0.021	0.128	0.321	0.250	0.017	0.000
210.4	7.22	0.846	0.027	0.161	0.403	0.248	0.007	0.000
235.3	7.22	1.291	0.048	0.284	0.664	0.295	0.001	0.000
264.5	7.23	1.431	0.065	0.387	0.780	0.200	0.000	0.000
289.9	7.23	0.914	0.051	0.304	0.499	0.061	0.000	0.000
309.9	7.24	0.874	0.058	0.338	0.454	0.023	0.000	0.000
333.4	7.24	0.838	0.071	0.377	0.388	0.002	0.000	0.000
363.6	7.25	0.485	0.058	0.265	0.161	0.000	0.000	0.000
389.6	7.26	0.219	0.036	0.138	0.045	0.000	0.000	0.000
413.7	7.26	0.192	0.041	0.131	0.020	0.000	0.000	0.000
439.5	7.27	0.087	0.024	0.061	0.002	0.000	0.000	0.000
464.1	7.28	0.085	0.030	0.055	0.000	0.000	0.000	0.000

Summary statistics for above 0.1s PSA deaggregation, R=distance, e=epsilon:
Mean src-site R= 158.8 km; M= 5.82; e0= 0.29; e= 1.13 for all sources.
Modal src-site R= 36.3 km; M= 4.81; e0= -0.61 from peak (R,M) bin
Primary distance metric: EPICENTRAL
MODE R*= 37.5km; M*= 4.81; EPS.INTERVAL: 0 to 1 sigma % CONTRIB.= 2.228

Principal sources (faults, subduction, random seismicity having >10% contribution)

Source:	% contr.	R(km)	M	epsilon0 (mean values)
CEUS gridded seismicity,Frankel	67.06	171.9	5.90	0.30
CEUS gridded seismicity,Toro att	32.95	132.0	5.68	0.29

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Written by:	**R. Kulasingam**	Date:	**01/04/2011**	Reviewed by:	**Ming Zhu/Jay Beech**	Date:	**01/04/2011**
Client:	**Honeywell**	Project:	**Onondaga Lake ILWD Stability**	Project/ Proposal No.:	**GD4014**	Task No.:	**02**

ADDENDUM TO THE PACKAGE TITLED "LIQUEFACTION POTENTIAL ANALYSES"

INTRODUCTION

The purpose of this addendum is to supplement the calculation package titled "*Liquefaction Potential Analyses*" (original calculation package) by presenting the liquefaction potential analyses for the ILWD area subsurface materials that include the weight of the cap. Only the existing conditions were analyzed in the original calculation package because that was considered to be a conservative approach. The methodology used in this addendum is the same as what was used in the original calculation package. A 5-ft thick sediment cap with an estimated average unit weight of 120 pcf was modeled to evaluate the influence of the cap on the liquefaction potential of the subsurface materials. An evaluation of the cap liquefaction potential is also included in this addendum as Attachment 1.

ANALYSIS RESULTS AND CONCLUSION

Table 1 presents the Cyclic Stress Ratio (CSR) values calculated using the simplified procedure proposed by Seed and Idriss [1971], as described in the original calculation package. The calculated CSR values are plotted with depth in Figure 1. The calculated CSR values generally ranged from 0.09 at a depth of 75 ft (same as the 70-ft deep location for the existing conditions) in the silt and clay unit to 0.15 near the top of the Solvay waste (SOLW), with a maximum CSR value of 0.19 at a depth of 25 to 35 ft below the top of the cap. For existing conditions, the calculated CSR values generally ranged from 0.10 at a depth of 70 ft in the silt and clay unit to 0.25 near the top of the SOLW, as presented in the original calculation package. The difference in the calculated CSR values and distribution with depth was caused by the addition of the cap with a significantly higher unit weight than the subsurface materials. The calculated CSR values for the subsurface materials are less for the case including the cap than for the existing conditions, indicating that the addition of the cap decreases the severity of the seismic loading conditions.

Table 1 also presents the Cyclic Resistance Ratio (CRR) values for cohesive soils calculated using the procedure proposed by Boulanger and Idriss [2007], as described in the original calculation package. Figure 1 shows the distribution of CRR with depth graphically. The same shear strength ratios were used in both this addendum and the original calculation package. The results indicate that the calculated CRR values for the case including the cap are the same as those for the existing conditions. Because the calculated CSR values for the case including the cap are smaller than those for the existing conditions (as discussed in the previous section), the calculated factor of safety against liquefaction (FS_{liq}), which is the ratio between CRR and CSR, increases when considering the cap. As presented in Table 1 and Figure 2, the calculated factors of safety against liquefaction ranged from about 2.9 to 3.6 for the SOLW unit, 1.8 to 2.0 for the marl unit, and 2.3 to 3.4 for the silt and clay unit for the case including the cap analyzed herein. In liquefaction analyses, a calculated factor of safety of 1.0 to 1.2 is generally considered adequate to conclude that adverse effects due to pore pressure buildup are unlikely. Therefore, based on this analysis, the SOLW, marl, and silt and clay units in the ILWD area are not considered to be susceptible to liquefaction during the design seismic event.

The above analyses clearly show that the calculated CSR values decreased in the subsurface materials due to the addition of the cap, thereby indicating less severe seismic loading conditions. It is expected that the calculated CSR values for the deeper soil units such as silt and sand, and sand and gravel will decrease too. The strength of these soil units, expressed as corrected SPT blow counts for the purposes of liquefaction potential evaluation, are not expected to change due to the addition of a few feet thick cap. In addition, the calculations for the existing conditions presented in the original calculation package indicate significantly higher calculated CRR values compared to CSR values for these deeper soil units. Due to the above reasons, the deeper soils units (such as silt and sand, and sand and gravel) are not considered susceptible to liquefaction during the design seismic event.

All other additional evaluations and discussions presented in the original calculation package (e.g. screening criteria, triaxial test stress paths etc.) that contributed to the conclusion that subsurface materials are not considered susceptible to liquefaction during the design seismic event are not affected by the addition of a few feet thick cap, and therefore are not repeated herein.

An evaluation of the potential for cap liquefaction was also performed, as described in Attachment 1. Based on this evaluation, a monitoring and maintenance (as needed) approach is recommended. Additional details will be provided in the Cap Monitoring and Maintenance Plan.

Written by:	**R. Kulasingam**	Date:	**01/04/2011**	Reviewed by:	**Ming Zhu/Jay Beech**	Date:	**01/04/2011**
Client:	**Honeywell**	Project:	**Onondaga Lake ILWD Stability**	Project/ Proposal No.:	**GD4014**	Task No.:	**02**

REFERENCES

Boulanger, R.W. and Idriss, I.M. (2007), "Evaluation of Cyclic Softening in Silts and Clays", Journal of Geotechnical and Geoenvironmental Engineering, ASCE, Volume 133, No. 6, June 2007, pp. 641 - 652.

Seed, H.B. and Idriss, I.M. (1971). "Simplified Procedure for Evaluating Soil Liquefaction Potential", Journal of the Soil Mechanics and Foundations Division, ASCE, Vol. 107, No. SM9, pp. 1249 - 1274.

Tables

Written by: R. Kulasingam Date: 01/04/2011 Reviewed by: Ming Zhu/Jay Beech Date: 01/04/2011

Client: Honeywell Project: Onondaga Lake ILWD Stability Project/ Proposal No.: GD4014 Task No.: 02

Table 1. Calculation of CSR, CRR, and factor of safety against liquefaction

Unit weight of water =	62.4	pcf
Maximum surface acceleration =	0.09	g
K-alpha =	1	
OCR model parameter, m =	0.8	
Earthquake magnitude =	5.3	
Magnitude scaling factor for cohesive soils =	1.13	

Depth (ft)	Depth (m)	Idealized Soil Type	Unit Weight (pcf)	Vertical Effective Stress (psf)	Vertical Total Stress (psf)	Stress Reduction Factor	Equivalent Cyclic Shear Stress (psf)	CSR_M	Cu/Sigv' - NC	OCR	Cu/Sigv' - OC	$CRR_{7.5}$	CRR_M	FSliq
0	0.0	Cap	120	0	0	1.00	0							
5	1.5	Cap	120	288	600	0.99	35							
10	3.0	SOLW	81	381	1,005	0.98	58	0.15	0.35	2.0	0.61	0.49	0.55	3.63
15	4.6	SOLW	81	474	1,410	0.97	80	0.17	0.35	2.0	0.61	0.49	0.55	3.26
20	6.1	SOLW	81	567	1,815	0.96	102	0.18	0.35	2.0	0.61	0.49	0.55	3.06
25	7.6	SOLW	81	660	2,220	0.94	122	0.19	0.35	2.0	0.61	0.49	0.55	2.96
30	9.1	SOLW	81	753	2,625	0.92	141	0.19	0.35	2.0	0.61	0.49	0.55	2.92
35	10.7	SOLW	81	846	3,030	0.89	158	0.19	0.35	2.0	0.61	0.49	0.55	2.94
40	12.2	Marl	98	1,024	3,520	0.85	175	0.17	0.35	1.0	0.35	0.28	0.32	1.84
45	13.7	Marl	98	1,202	4,010	0.80	189	0.16	0.35	1.0	0.35	0.28	0.32	2.01
50	15.2	Silt and Clay	108	1,430	4,550	0.75	200	0.14	0.35	1.0	0.35	0.28	0.32	2.25
55	16.8	Silt and Clay	108	1,658	5,090	0.70	209	0.13	0.35	1.0	0.35	0.28	0.32	2.50
60	18.3	Silt and Clay	108	1,886	5,630	0.66	217	0.12	0.35	1.0	0.35	0.28	0.32	2.74
65	19.8	Silt and Clay	108	2,114	6,170	0.62	225	0.11	0.35	1.0	0.35	0.28	0.32	2.97
70	21.3	Silt and Clay	108	2,342	6,710	0.59	232	0.10	0.35	1.0	0.35	0.28	0.32	3.18
75	22.9	Silt and Clay	108	2,570	7,250	0.57	241	0.09	0.35	1.0	0.35	0.28	0.32	3.36

Figures

Written by:	R. Kulasingam	Date:	01/04/2011	Reviewed by:	Ming Zhu/Jay Beech	Date:	01/04/2011

Client:	Honeywell	Project:	Onondaga Lake ILWD Stability	Project/ Proposal No.:	GD4014	Task No.:	02

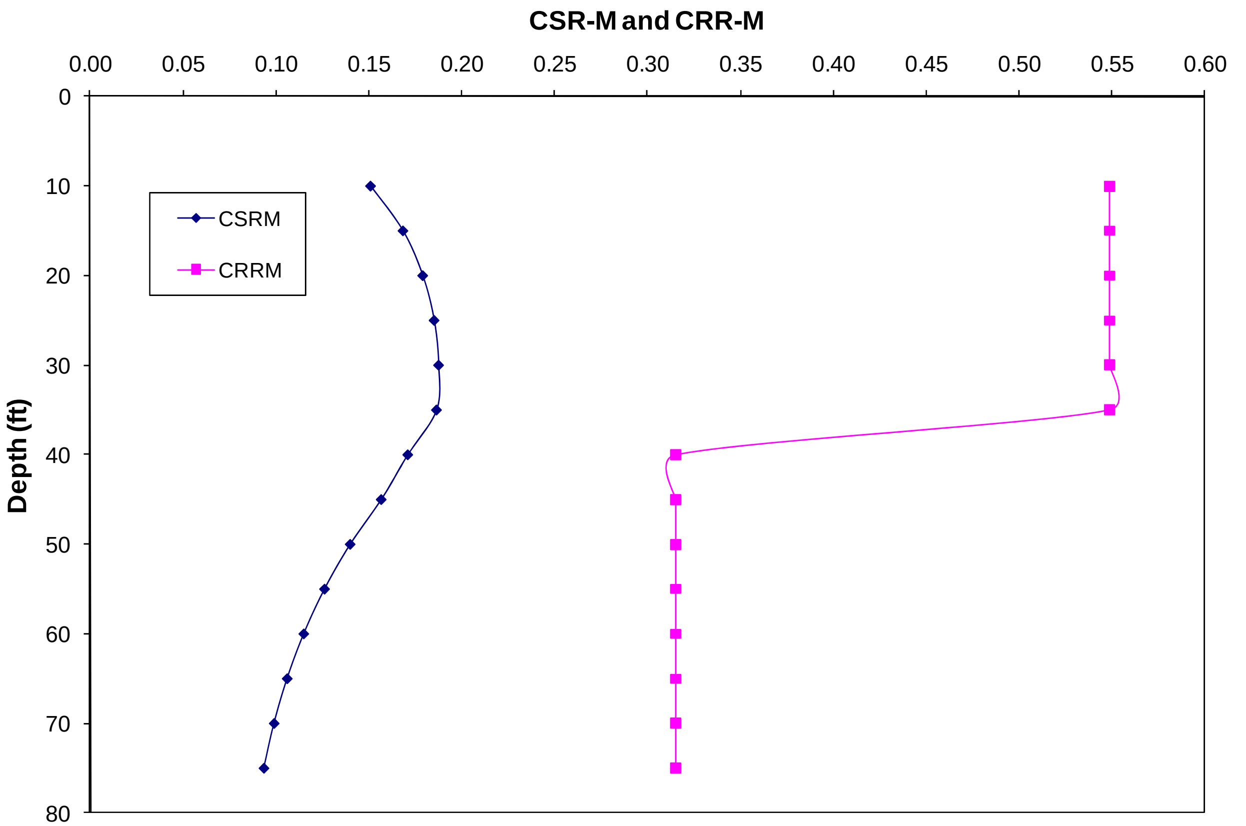

Figure 1. Calculated CSR and CRR values for SOLW, marl, and silt and clay.

Note: ***The discontinuity in CRR_M values occurred at 35 ft below the top of cap because it is the interface between Solvay waste and Marl. Solvay waste shear strengths were modeled with an OCR of 2, and Marl was modeled with an OCR of 1. These OCR values are conservative. The top 5 ft consists of the cap material.***

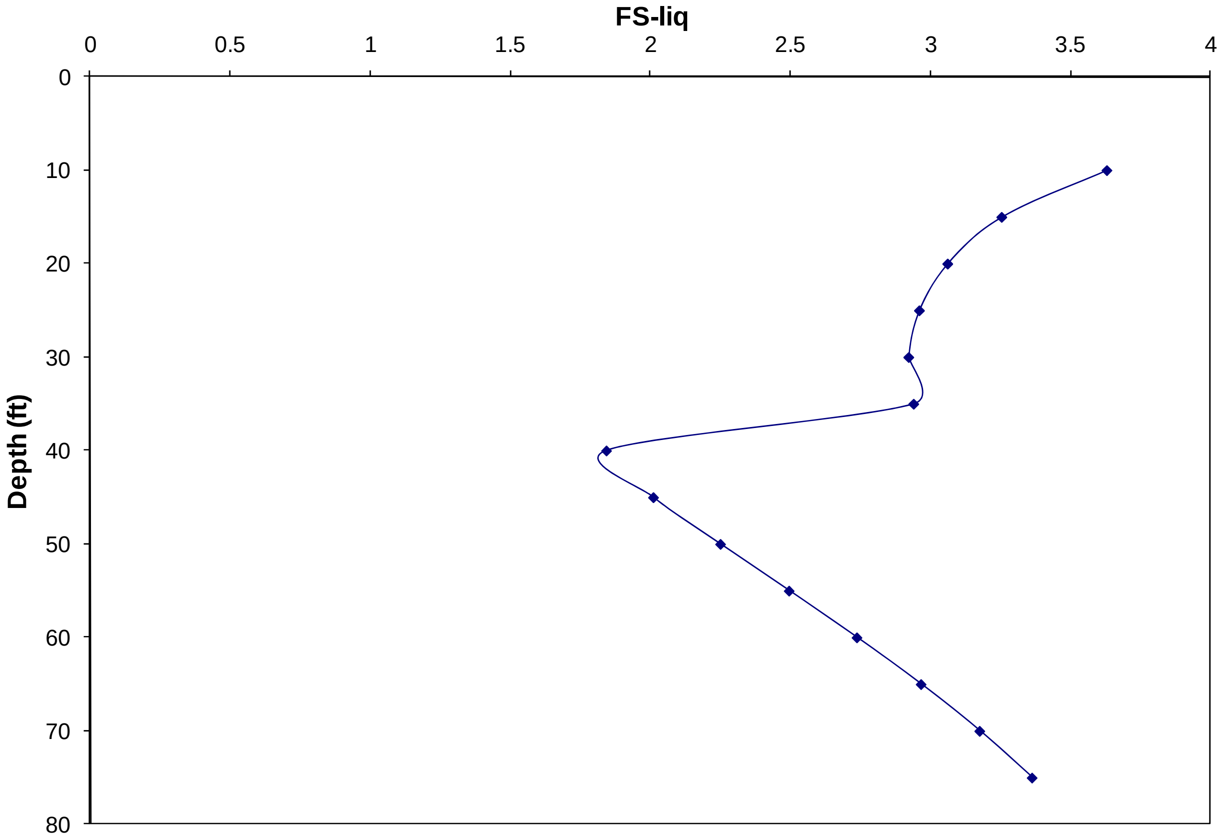

Figure 2. Calculated FS_{liq} values for SOLW, marl, and silt and clay.

Note: ***The discontinuity in FS-liq values occurred at 35 ft below the top of cap because it is the interface between Solvay waste and Marl. Solvay waste shear strengths were modeled with an OCR of 2, and Marl was modeled with an OCR of 1. These OCR values are conservative. The top 5 ft consists of the cap material.***

Attachment 1

LIQUEFACTION POTENTIAL OF THE PROPOSED SEDIMENT CAP

As documented in the main text of the *Capping, Dredging, and Habitat Design*, the sediment cap design for Onondaga Lake consists primarily of medium sand and gravel layers. Mean cap thicknesses will vary depending on water depth, remediation area, and over placement. The sand cap materials will be placed using a hydraulic spreading system, except for in shallow areas near the shoreline where mechanical placement will be necessary. All the coarse gravel materials will be placed mechanically. It is anticipated that the sand placed hydraulically will have a low relative density because of the placement method. Based on experience, low density sand materials typically have more potential for liquefaction than coarser materials; therefore, it is the sand material that is considered in the evaluation presented below.

General screening criteria (e.g., the Chinese criteria [Wang, 1979] or Andrews and Martin [2000], as described in Appendix H.2) were considered to evaluate the liquefaction potential of the sand in the cap. Since the sand will likely have less than 10% clay content and a liquid limit less than 32, these criteria indicate that the capping layer may be susceptible to liquefaction. Other liquefaction evaluation methods that are used in standard practice to estimate liquefaction susceptibility and potential displacements were considered. For a variety of reasons, they were not considered appropriate for this application. Because of this, a more practical approach, as described in the paragraphs that follow, was selected. For completeness, a brief discussion of these other evaluation methods and why they are not considered applicable is provided following the references section below.

Onondaga Lake is not in a seismic impact zone, as defined in RCRA Subtitle D(258) Seismic Guidance for Municipal Solid Waste Landfill Facilities (Richardson et al, 1995), which means there is a less than 10 percent probability that the maximum horizontal acceleration in lithified earth material, as expressed as a percentage of the earth's gravitational pull, will exceed 0.10g in 250 years. As such, the risk and frequency of earthquakes is low. Liquefaction evaluations of the Solvay waste and underlying Marl and Silt and Clay [see Appendix H.2] resulted in acceptable factor of safety values. Therefore, if the sand layer in the cap does liquefy, the effects are expected to be limited to within the cap itself and not affect the underlying materials. These effects (if any) are generally expected to be manifested at the cap surface in the form of cracking, slumping, and/or displacements. In the event of liquefaction impacts from an unlikely seismic event, the cap can be readily repaired as part of the long-term monitoring and maintenance program. The extent of these potential effects has not been estimated because of the large uncertainties inherent in that type of calculation. The potential effects are strongly related to slope and seismic event, with steeper slopes and stronger seismic events showing greater potential for localized displacements. The slopes of the cap are generally very flat (i.e., three to five degrees), therefore, the extent of impact, if any, is expected to be limited. In addition, the steeper slope areas are primarily near the shoreline and are surrounded by flatter slopes.

Monitoring and maintenance (as needed) approaches have already been successfully used for caps installed at sites that are in seismic impact zones (i.e., the Western United States) [Waukeganweb, 2002]. For the Onondaga lake site, cap monitoring is recommended to be performed after: (i) indication of significant damage to structures in the Syracuse metropolitan area due to an earthquake; or (ii) occurrence of a 5.5 or greater magnitude on the Richter scale earthquake within 30 miles. Based on these monitoring events, cap maintenance would be performed, if required. Additional details will be provided in the Cap Monitoring and Maintenance Plan.

REFERENCES

Andrews, D.C.A. and Martin, G.R. (2000). "Criteria for Liquefaction of Silty Soils", 12th World Conference on Earthquake Engineering, Proceedings, Auckland, New Zealand.

Richardson, G., Kavazanjian, E., and Matasovic, N (1995). "RCRA Subtitle D(258) Seismic Design Guidance for Municipal Solid Waste Landfill Facilities", EPA/600/R-95/051.

Wang, W. (1979). "Some Findings in Soil Liquefaction", Water Conservancy and Hydroelectric Power Scientific Research Institute, Beijing, China.

Waukeganweb (2002), "Draft summary of contaminated sediment capping projects – 2/27/2002", **http://www.waukeganweb.net/waukegan%20harbor%20clean%20up/USEPA%20Summary%20of%20Capping%20Projects.pdf**

OTHER LIQUEFACTION EVALUATION METHODS

As indicated above, liquefaction evaluation methods other than the general screening criteria approach were considered and then determined to be inapplicable for the sand cap. Specifically, methods used in standard engineering practice for evaluating liquefaction potential in sands (e.g., Seed and Idriss, 1971; NCEER, 1997) were considered. These methods are based on *in situ* soil testing, such as standard penetration tests (SPTs) and cone penetrometer tests (CPTs), and comparison to case history databases. Since the case histories do not include sand layers placed under water to shallow depths, as is the case for the proposed Onondaga Lake sediment cap, these methods were not considered applicable.

Liquefaction evaluation approaches using laboratory testing were also considered. These approaches have been used in cases of thick sand deposits placed under water (e.g., hydraulic fill dams) and are based on either: (i) undrained cyclic triaxial testing of sand samples prepared in the laboratory; or (ii) determination of the *in situ* void ratio of the hydraulically placed sand and comparing it to the critical state or steady state line obtained in the laboratory. Both these approaches suffer from limitations in terms of measuring or recreating the *in situ* void ratio and structure of the sand, as well as having to assume that fully undrained conditions would prevail during the design earthquake. Small changes in void ratio can have a large effect on the undrained behavior of sand in these tests, and estimating and recreating the *in situ* void ratio can be difficult. The void ratio can also vary with time (i.e., a freshly deposited sand will have a higher void ratio than a sand cap that has been in place for several years). This can lead to under or overestimation of the liquefaction potential. In addition, partial drainage (i.e., free draining boundary conditions above or below the deposit) will likely occur for a thin sand cap; however, this type of testing, which evaluates fully undrained conditions, is more applicable to a fully undrained thick sand deposit. Since this testing cannot take partial drainage into consideration, it will potentially overestimate susceptibility to liquefaction. Because of these limitations, laboratory testing approaches were not considered appropriate to evaluate the liquefaction potential of the proposed sediment cap.

In addition, hybrid approaches for evaluating the liquefaction potential that combine past experience with laboratory testing and case histories were considered. These approaches include estimating relative densities from past laboratory tests and correlating them to SPT blow counts, estimating undrained residual shear strengths from SPT blow counts using case histories, and performing deformation analyses. Typically, the deformation analyses result in large uncertainty in calculated cap displacements over a range of possible relative density values because of the numerous assumptions that are required as part of the analysis. In addition, all the limitations that were mentioned for the laboratory testing and case histories apply to this hybrid approach. From a practical standpoint, calculating a wide range in deformations is not considered useful because it would not influence how potential liquefaction will be addressed in the design; therefore, additional calculations are not recommended. Instead, low earthquake risk for the site, flat slopes, and underlying materials that are not susceptible to liquefaction were considered in developing a monitoring and maintenance, if required, approach for the cap, as discussed above.

REFERENCES

NCEER (1997). "Summary Paper, Proc. of NCEER Workshop on Evaluation of Liquefaction Resistance of Soils", NCEER-97-0022.

Seed, H.B. and Idriss, I.M. (1971). "Simplified Procedure for Evaluating Soil Liquefaction Potential", Journal of the Soil Mechanics and Foundations Division, ASCE, Vol. 107, No. SM9, pp. 1249 - 1274.