UTAH DIVISION OF RADIATION CONTROL: ENERGY*SOLUTIONS'* CLIVE LLRW DISPOSAL FACILITY;

LICENSE AMENDMENT REQUEST; CLASS A WEST EMBANKMENT

ROUND 3 Interrogatories

1. DISTORTION/STRAIN

INTERROGATORY CAW R313-25-7(3)-04/3: SPECIFIC TECHNICAL INFORMATION – DESIGN CRITERION FOR DISTORTION OF LINER AND CLAY COVER COMPONENTS

PRELIMINARY FINDING:

Refer to R313-25-7. The application shall include certain technical information….
(3) Descriptions of the principal design criteria and their relationship to the performance objectives.

INTERROGATORY STATEMENT:

1. *Please demonstrate that the maximum tensile strain associated an angular distortion of 0.02 is sufficiently less than the strain which causes a compacted clay liner (CCL) to crack, considering the properties of the soils in question and the results/findings from recent investigations of compacted clay layer deformation behavior. In particular, the demonstration should consider the effects of both localized axial lengthening and bending when relating tensile strain to angular distortion.*
2. *Please revise Table 3 of the response to include only references and cases that are directly applicable to assessing the strain at which a CCL might crack. As part of this revision, assure that each cited strain level is that associated with cracking and not simply the maximum tensile strain reached at the end of the experiment, well after the onset of cracking.*
3. *Please consider and explain whether results from controlled, shorter-term laboratory tests as cited by the Licensee from technical literature are directly transferable to the long-term performance of clayey soils where soil creep and/or moisture changes can potential affect the soil behavior.*
4. *Please justify the margin between the tensile strain at cracking and the allowable maximum tensile strain selected for design.*

5. *Prepare and submit results of technical analyses that demonstrate that maximum allowable strains determined as described in Items 1 through 4 above are not exceeded under conditions expected to exist within the proposed CAW Embankment.*

BASIS FOR INTERROGATORY

EnergySolutions' response to Interrogatory CAW R313-25-7(3)-04/2C, Item 4 does not sufficiently demonstrate the adequacy of the 0.02 value for the maximum allowable angular distortion used as a design criterion for the liner and clay components of the CAW Embankment cover. In particular, the Licensee has attempted to demonstrate adequacy of the 0.02 distortion value by first establishing, by a literature review, the maximum tensile strain of clay before cracking, then correlating that strain to a corresponding angular distortion, and finally comparing that angular distortion with the design criterion.

From the information provided in Table 3 of the Licensee's response, it appears that the Licensee is suggesting that for the clays at the Clive site, the limiting tensile strain before cracking is in the range of 0.3% to more than 0.9% (although this is open to some interpretation since a succinct statement of what the tensile strain at cracking is expected to be has not been explicitly provided). Next, the Licensee's response states that based on the tensile strain versus angular distortion relationship presented by LaGatta et al. (1997) and referenced in EPA (2004) [Licensee needs to provide complete reference/citation information for final version of said latter document], a maximum allowable angular distortion of 0.02 will produce a strain much lower than 0.2% (because the relationship indicates that an angular distortion of 0.06 equates to a strain of about 0.2%). Unfortunately, deficiencies in this approach exist as indicated in the previous interrogatory, and the subsequent response to that interrogatory still does not adequately address these deficiencies and other issues identified in the interrogatory.

Of particular concern is the tensile strain versus angular distortion relationship used by the Licensee. This relationship presented in LaGatta et al. (1997) is a simple mathematical model in which strain is calculated from the axial lengthening of the hypotenuse of a right triangle (representing the cover) as the length of one of the other sides of the triangle (representing the amount of vertical settlement) increases. As such, the tensile strain represented by this model is only an <u>average</u> axial strain which does not account for bending-induced strains away from the neutral axis or any localized lengthening and/or bending effects. The recent study by Rajesh and Viswanadham (2010), cited by the Licensee, shows more realistic tensile strain versus angular distortion relationships based on numerical- and measurement-based subsidence profiles which include both axial and bending effects. Figure 1 presents a comparison of tensile strain as a function of angular distortion based on both Rajesh and Viswanadham and LaGatta et al. It can readily be seen that inclusion of localized bending and/or lengthening of the cover can result in much greater tensile strains for a given amount of angular distortion. The Licensee should consider the effects of localized bending and lengthening (whether using the data of Rajesh and Viswanadham or other data from published sources which the Licensee may deem more appropriate) when relating angular distortion to tensile strain.

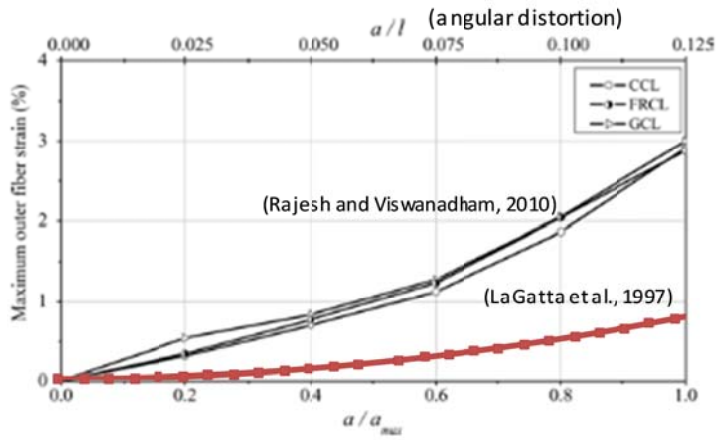

Figure 1. Variation of tensile outer fiber strain with angular distortion based on Rajesh and Viswanadham (2010) and LaGatta et al. (1997).

Additionally, the Licensee should review the data provided in Table 3 of its previous response. In the case of Le (2009), the data appears to be from compression rather than tensile tests, and as such are not directly relevant and should be removed from the table. In also appears that in some cases, the Licensee is confusing the strain level at which tensile cracking occurred with the maximum strain level induced during the test (which may have been reached late in a series of tests, well after cracking occurred). This appears to be the case for strain level data cited from Viswanadham [and Mahesh?] (2002) [which are an order of magnitude larger than the other data] and Rajesh and Viswanadham (2010).

When establishing the allowable maximum tensile strain as related to angular distortion and the data for maximum tensile strain at cracking, the Licensee should consider and explain whether results from these controlled, shorter-term laboratory tests are directly transferable to the long-term performance of clayey soils where soil creep and/or moisture changes can potential affect the soil behavior. The margin between the tensile strain at cracking and the allowable maximum tensile strain selected for design should be justified.

REFERENCES

LaGatta, M.D., Boardman, B.T., Cooley, B.H., and Daniel, D.E. (1997). "Geosynthetic Clay Liners Subjected to Differential Settlement," Journal of Geotechnical and Geoenvironmental Engineering, Vol. 123, No. 5, pp. 402-410.

Rajesh, S. and Viswanadham, B.V.S. (2010). "Performance Assessment of Deformation Behavior of Landfill Barriers at the Onset of Differential Settlement," International Journal of Environmental Engineering, Vol. 2.1, pp. 269-289.

Viswanadham, B.V.S. and Mahesh, K.V. (2002). "Modeling Deformation Behavior of Clay Liners in a Small Centrifuge," Canadian Geotechnical Journal, Vol. 39, pp. 1406-1418.

2. PROBABLE MAXIMUM PRECIPITATION (PMP)

INTERROGATORY CAW R313-25-8(4)-11/3 TECHNICAL ANALYSIS - ROCK COVER DESIGN AND ROCK COVER DESIGN CALCULATIONS/ ANALYSES

PRELIMINARY FINDING:

Refer to R313-25-8(4). Analyses of the long-term stability of the disposal site shall be based upon analyses of active natural processes including erosion, mass wasting, slope failure, settlement of wastes and backfill, infiltration through covers over disposal areas and adjacent soils, and surface drainage of the disposal site. The analyses shall provide reasonable assurance that there will not be a need for ongoing active maintenance of the disposal site following closure.

INTERROGATORY STATEMENT:

Refer to Attachment 10 of Energy*Solutions* 2012: *To further confirm the appropriateness and currency of the calculated Probable Maximum Precipitation (PMP) value and as used, for example, in the rock cover design calculations, please provide the following:*

1. *A revised PMP calculation updating the PMP distribution that incorporates information from the following documents, in addition to HMR 49 (NOAA 1984):*

    - *"2002 Update for Probable Maximum Precipitation, Utah 72 Hour Estimates to 5,000 sq. mi". – March 2003 Jensen 2003); and*
    - *"Probable Maximum Precipitation Estimates for Short Duration, Small Area Storms in Utah" – October 1995 (Jensen 1995)*

2. *Provide a revised Rock Cover Design calculation package (revised Attachment 10 to the CAW LAR) reflecting the updated PMP information, in accordance with the revised PMP information, as appropriate.*

BASIS FOR INTERROGATORY:

When determining the PMP for High Hazard and Moderate Hazard dams, the State of Utah currently requires the use of HMR 49, which Energy Solutions has used in Attachment 10 to the CAW LAR (Rock Cover Calculations dated December 27, 2011 in EnergySolutions 2012), but also in conjunction with the use of two other reports: (1) the "2002 Update for Probable Maximum Precipitation, Utah 72 Hour Estimates to 5,000 sq. mi. – March 2003" and (2) "Probable Maximum Precipitation Estimates for Short Duration, Small Area Storms in Utah – October 1995." Although these two methods were developed (by the Utah Climate Center) for estimating PMF conditions for design of dams, these methods are considered to be more representative of actual conditions in Utah. The use of the two additional reports may not have been required in 1996 when the Bingham Environmental (Bingham Environmental, Inc. 1996)

PMP calculations referenced and used in Attachment 10 were done. The Rock Cover Calculations need to be revised to reflect the revised PMP determination findings, as appropriate, to demonstrate that the proposed rock cover design remains appropriate from an erosion protection point of view (or a new set of erosion protection calculations submitted to support an alternative cover design such as one incorporating a rock-soil matrix layer).

REFERENCES:

Bingham Environmental, Inc., David Cline – Project Manager, HEC-1 and HEC-2 Analysis, LARW Application for License Renewal, Envirocare Disposal Facility, Clive, Utah. November 26, 1996 (Appendix KK of the Envirocare LARW License Renewal Application).

EnergySolutions, LLC. 2012. Radioactive Material License # UT 2300249 and Ground Water Quality Discharge Permit No. UGW450005. Amendment and Modification Request – Class A West Embankment: Response to Round 2 Interrogatory. Letter to Mr. Rusty Lundberg at Utah Division of Radiation Control dated January 12, 2012, with Response and Attachments.

Jensen, D. 1995. 2002 Update for Probable Maximum Precipitation, Utah 72 Hour Estimates to 5,000 sq. mi. - March 2003. Utah Climate Center.

Jensen, D. 2003. Probable Maximum Precipitation Estimates for Short Duration, Small Area Storms in Utah - October 1995. Utah Climate Center.

3. SEISMIC

INTERROGATORY CAW R313-25-8(4)-16/3: SEISMIC HAZARD EVALUATION / SEISMIC STABILITY ANALYSIS UPDATE

PRELIMINARY FINDING:

Refer to R313-25-8(5). Analyses of the long-term stability of the disposal site shall be based upon analyses of active natural processes including erosion, mass wasting, slope failure, settlement of wastes and backfill, infiltration through covers over disposal areas and adjacent soils, and surface drainage of the disposal site. The analyses shall provide reasonable assurance that there will not be a need for ongoing active maintenance of the disposal site following closure.

INTERROGATORY STATEMENT:

SEISMIC HAZARD ANALYSIS

The responses to Interrogatory CAW R313-25-8(4)-16/2A are inadequate and do not address the requests stated in the interrogatory. The Round 2 interrogatory is, therefore, restated and extended below:

1. *Perform and submit results of a probabilistic seismic hazard analysis (PSHA). Utilize a corrected version of EZ-FRISK or other accepted software for conducting PSHAs. Prepare the PSHA without using the Pankow and Pechmann (2004) ground motion model and giving full weight to the NGA models. Remove foreshocks and aftershocks from the earthquake history by de-clustering the earthquake catalog as done in standard probabilistic approaches and as was done by Pechmann and Arabasz (1995). Use the resulting earthquake catalog in any ground motion calculations.*
2. *Correctly label the vertical axis in Figure 3 and correctly interpret the information presented in Figure 3 (cumulative frequency plot).*

SOIL LIQUEFACTION/CYCLIC SOFTENING

3. *The Applicant's response to Round 2 CAW interrogatories does not include a response to Interrogatory CAW R313-25-8(4)-16/2C. Please respond to Interrogatory CAW R313-25-8(4)-16/2C, repeated and extended below:*

   Please demonstrate that the potential effects of soil liquefaction and/or cyclic softening phenomena in native soils at the Clive Facility have been adequately accounted for in the geotechnical analyses supporting the design of the proposed CAW Embankment. In doing so, clearly justify the selection of soil parameters and any design assumptions by comparison of such with correlations, field test results, and/or laboratory test results (including cyclic shear testing) consistent with the guidance given by developers of current, published analytical methods. Also, evaluate and document the effects on previous liquefaction and embankment stability assessments and cyclic softening resulting from changes to the seismic hazard analysis addressed in Items 1 and 2 of this interrogatory.

BASIS FOR INTERROGATORY:

SEISMIC HAZARD ANALYSIS:

As requested in Interrogatory CAW R313-25-8(4)-16/2A, the Licensee did not justify the use of the semi-probabilistic approach for addressing the hazard from background earthquakes. The semi-probabilistic approach is not "standard practice for earth embankments" as stated in the response. For example, Utah Dam Safety and the Utah Division of Water Resources use a 5000-year return period probabilistic hazard map for the State produced by the U.S. Geological Survey to address background earthquakes in Utah. Utah Dam Safety recognized the deficiencies in semi-probabilistic approaches and decided to take a probabilistic seismic hazard

analysis (PSHA) approach to address background earthquake hazard, as do the majority of seismic hazard evaluation practitioners currently. The Licensee did not address the criticisms of Pechmann and Arabasz (1995) of the semi-probabilistic approach as described in the Basis for Interrogatory. Although the Licensee did perform a PSHA in lieu of the semi-probabilistic approach as requested in the Interrogatory, the following comments must be addressed to produce an acceptable PSHA.

It appears that the Licensee used the semi-probabilistic approach to address the hazard from background earthquakes because they did not have a PSHA code that they thought was suitable for use for the Clive site. The Licensee should not constrain their response by their earlier seismic hazard evaluation (October 25, 2011 letter) simply because they lack a suitable PSHA code. Also the Licensee is using a version of the proprietary code EZ-FRISK that contains an error in the hanging wall term of the Abrahamson and Silva (2008) NGA ground motion prediction model. The Licensee states that they requested Risk Engineering in 2009, the developer of EZ-FRISK, to make the correction but that they are still waiting for a corrected version of EZ-FRISK. It is suggested that the Licensee switch to another available PSHA code. Alternatively, Bob Youngs of AMEC Geomatrix is a nationally recognized practitioner in PSHAs who has developed his own codes.

The reviewer is puzzled why the ground motion prediction model by Pankow and Pechmann (2004) was used in the response (Addendum) to Interrogatory 2 when it was not used in their 25 October 2011 analysis, which only used the NGA models. The use of this model by the Licensee in the deterministic hazard analysis and PSHA described in the Addendum is not state-of-the-practice. The USGS did not use the Pankow and Pechmann (2004) model in the National Seismic Hazard Maps because the NGA models were available. Both Drs. Pankow and Pechmann at the University of Utah support the use of the NGA models in lieu of their own model. The Licensee's assertion that the NGA models are based mainly on California data and data from compressional tectonic regimes is incorrect. The NGA models are appropriate for normal faulting regimes (such as Utah), as was recognized by the USGS in their development of the National Seismic Hazard Maps. The database of normal faulting strong motion data used in the NGA models is superior to the database used in the original Spudich et al. (1999) model and subsequently by Pankow and Pechmann (2004). Hence the calculations performed by the Licensee should be repeated without the use of the Pankow and Pechmann (2004) ground motion model and the full weight should be given to the NGA models.

The current state-of-practice in seismic hazard evaluations is to not consider the hazard from foreshocks or aftershocks. In the calculation of historical seismicity recurrence, dependent events such as foreshocks and aftershocks are removed from the historical catalog through the process of declustering. In particular, it is important to remove dependent events when calculating seismicity rates for input into probabilistic seismic hazard analysis (PSHA) because PSHA assumes that earthquakes follow a Poisson process; that is they are independent events. As stated by McGuire (2004), "foreshocks and aftershocks are by definition smaller than the mainshock and thus the associated motions will generally be smaller. As a result, the standard procedure in PSHA is to include only mainshocks in deriving magnitude distributions". We note

the Licensee is not following standard practice by including dependent events in their semi-deterministic approach; however, we agree that their calculated recurrence is conservative. The use of the 84th percentile ground motions is an accepted approach to include conservatism in a deterministic hazard analysis. Hence it should be noted that the Licensee is including additional conservatism over and above the use of 84th percentile ground motions in their hazard analyses.

The continued use of the terms "recurrence interval" and "return period" by the Licensee as interchangeable is incorrect. The USGS website cited by the Licensee in their response is incorrect and the USGS has been notified and are in the process of correcting this error (Mark Petersen, USGS, written communication, 2012). As stated by Robin McGuire, the author of the PSHA code EZ-FRISK used by the Licensee, in his book on "Seismic Hazard and Risk Analysis" the two terms are defined as follows:

> *Return Period: the mean (average) time between occurrences of a seismic hazard, for example, a certain ground motion at a site, or a certain level of damage or loss.*
>
> *Recurrence Interval: the mean (average) time between occurrences of a given type of earthquake, for example, an earthquake of a specified magnitude, on a fault or in a region.*

<u>SOIL LIQUEFACTION/CYCLIC SOFTENING:</u>

In recent years, the geotechnical engineering profession's understanding of, and analysis methods for, liquefaction-related phenomena have evolved. In the past, liquefaction was treated as a phenomenon largely associated with the seismic loading of loose, clean sands which could result in significant loss of strength and large deformations. However, the 1999 earthquakes in Kocaeli, Turkey and Chi-Chi, Taiwan both highlighted the potential for significant strength loss and deformation of finer-grained soils – soils previously considered "non-liquefiable." Subsequent research (e.g., Andrews and Martin, 2000; Seed et al., 2003; Boulanger and Idriss, 2004, 2005, and 2006; Bray and Sancio, 2006; and Youd et al., 2009) has generally led to a distinction between "sand-like" soils which undergo liquefaction and "clay-like" soils which undergo cyclic-softening. Both phenomena are generally associated with generation of high pore pressures and strains during shear; however, the distinction between liquefaction and cyclic softening is important in that the methods of analysis and assessment are different for the different types of soil. Also important is that the resulting behaviors can vary.

In previous reports as well as "Geotechnical Update Report" dated February 15, 2011 (Attachment 5 to EnergySolutions, 2011), the Licensee addressed liquefaction susceptibility using site specific data and analyses (see Section 4.5.2, page 19, of referenced document). However, rather than presenting quantitative factors of safety and/or cyclic resistance and cyclic stress ratios, the Licensee qualitatively summarized the results of the analyses thusly:

> *"The 2005 study determined that for the design event, the majority of the soils in the upper 30 to 60 feet of the soil profile consist of cohesive deposits, which have a low probability of liquefaction due to their high clay content. It was also found that the*

interbedded cohesionless silt and silty sand deposits would also be unlikely to liquefy under the design seismic event."

A close reading of this statement reveals that the susceptibility of non-silty or "clean" sands (those which, if loose, are most prone to liquefaction) which may be at the site has not been addressed. Also, from the information provided, it is not clear how the finer-grained soils were treated in the analyses. Similarly, with respect to slope stability and other deformation-related assessments, it is unclear how the shear strengths of finer-grained soils subject to seismic loading conditions were assessed and quantified. Reported fines content, moisture content, and Atterberg limit data suggest that some of the loose/soft soils at the site are "marginal" soils which may or may not experience liquefaction and/or cyclic softening. Published guidance and criteria (e.g., Youd et al., 2001; Boulanger and Idriss, 2008, Bray and Sancio, 2008, Boulanger and Idriss, 2011) currently referenced in the geotechnical engineering profession typically recommend that such soils be examined in greater detail and potentially be subjected to cyclic shear testing.

Stability and deformation calculations for existing embankments may be affected by the particular issues described in this interrogatory. Stability and deformations associated with the proposed CAW Embankment, particularly given the increase in embankment height and longer slopes of this embankment relative to other embankments at the Clive Facility, need to be assessed with consideration given to these issues.

Also, it should be noted that other current/recent interrogatories submitted for the proposed CAW Embankment License Amendment Request focus on further verifying the level of ground acceleration expected at the site. The effect of any revisions to such parameters on previous liquefaction and embankment stability assessments need to evaluated and documented.

REFERENCES

SEISMIC HAZARD ANALYSIS:

Abrahamson, N.A. and Silva, W.J., 2008, Summary of the Abrahamson and Silva NGA ground motion relations: Earthquake Spectra, v. 24, p. 67-97.

McGuire, R.K., 2004, Seismic hazard and risk analysis: Earthquake Engineering Research Institute, MNO-10, 240 p.

Pankow, K.L. and Pechmann, J.C., 2004, The SEA99 ground-motion predictive relations for extensional tectonic regimes: Revisions and a new peak ground velocity relation: Bulletin of the Seismological Society of America, v. 94, p. 341-348.

Pechmann, J.C. and Arabasz, W.J., 1995, The problem of the random earthquake in seismic hazard analysis: Wasatch Front region, Utah, in W. R. Lund (ed.), Environmental and Engineering Geology of the Wasatch Front Region, 1995 Symposium and Field Conference: Utah Geological Association Publication 24, p. 77-93.

Spudich, P., Joyner, W.B., Lindh, A.G., Boore, D.M., Margaris, B.M., and Fletcher, J.B., 1999, SEA99 - A revised ground motion prediction relation for use in extensional tectonic regimes: Bulletin of the Seismological Society of America, v. 89, p. 1156-1170.

SOIL LIQUEFACTION/CYCLIC SOFTENING:

Andrews, D.C.A. and Martin, G.R. (2000).“Criteria for liquefaction of silty soils.” Proc. Twelfth World Conf. on Earthquake Eng., Auckland, New Zealand.

Boulanger, R.W. and Idriss, I.M. (2004). “Evaluating the potential for liquefaction or cyclic failure of silts and clays.” Rep.No.UCD/CGM-04/01, Center for Geotechnical Modeling, Dept. of Civil and Environmental Eng., Univ. of California, Davis ,CA, USA.

Boulanger, R.W. and Idriss, I.M. (2005). “New criteria for distinguishing between silts and clays that are susceptible to liquefaction versus cyclic failure.” Proc. Technologies to Enhance Dam Safety and the Environment, 25th Annual United States Society on Dams Conf., USSD, Denver, pp. 357–366.

Boulanger, R.W. and Idriss, I.M. (2006). “Liquefaction susceptibility criteria for silts and clays.” J. of Geotechnical and Geoenvironmental Eng., ASCE, Vol. 132, No. 11, pp. 1413-1426.

Boulanger, R.W. and Idriss, I.M. (2008). Closure to “Liquefaction susceptibility criteria for silts and clays.” J. of Geotechnical and Geoenvironmental Eng., ASCE, Vol. 134, No. 7, pp. 1027-1028.

Boulanger, R.W. and Idriss, I.M. (2011). “Cyclic failure and liquefaction: Current issues.” Proc. Fifth International Conf. of Earthquake Geotechnical Eng., Santiago, Chile.

Bray, J.D. and Sancio, R.B. (2006). “Assessment of the liquefaction susceptibility of fine grained soils.” J. of Geotechnical and Geoenvironmental Eng., ASCE, Vol. 132, No. 9, pp. 1165-1177.

Bray, J.D. and Sancio, R.B. (2008). Closure to “Assessment of the liquefaction susceptibility of fine grained soils.” J. of Geotechnical and Geoenvironmental Eng., ASCE, Vol. 134, No. 7, pp. 1031-1034.

EnergySolutions (2011). License Amendment Request: Class A West Embankment, with Attachments 1 through 7 and cover letter to Mr. Rusty Lundberg at Utah Division of Radiation Control dated May 2, 2011.

Seed, R.B., Cetin, K.O., Moss, R.E.S., Kammerer, A.M., Wu, J., Pestana, J.M. Riemer, M.F., Sancio, R.B., Bray, J.D., Kayen, R.E., and Faris A. (2003). “Recent advances in soil liquefaction engineering: A unified and consistent framework.” Keynote presentation, 26th Annual ASCE Los Angeles Geotechnical Spring Seminar, H.M.S. Queen Mary, Long Beach, CA, USA, Report EERC-2003–06, Earthquake Engineering Research Institute, Berkeley, CA, USA.

Youd, T.L., DeDen, D.W., Bray, J.D., Sancio, R., Cetin, O., and Gerber, T.M. (2009). "Zero-displacement lateral spreads, 1999 Kocaeli, Turkey Earthquake." J. of Geotechnical and Geoenvironmental Eng., ASCE, Vol. 135, No. 1, pp. 46-61.

Youd, T. L., Idriss, I. M., Andrus, R. D., Arango, I., Castro, G., Christian, J. T., Dobry, R., Finn, W. D. L., Harder, L. F., Jr., Hynes, M. E., Ishihara, K., Koester, J. P., Liao, S. S. C., Marcuson, W. F., III, Martin, G. R., Mitchell, J. K., Moriwaki, Y., Power, M. S., Robertson, P. K., Seed, R. B., and Stokoe, K. H., II. (2001). "Liquefaction resistance of soils: summary report from the 1996 NCEER and 1998 NCEER/NSF workshops on evaluation of liquefaction resistance of soils." J. of Geotechnical and Geoenvironmental Eng., ASCE, Vol. 127, No. 10, pp. 817-833.

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