LRFD Liquefaction Analysis

This design guide illustrates the Department’s recommended procedures for analyzing the liquefaction potential of soil during a seismic event considering Article 10.5.4.2 of the 2009 Interim Revisions for the AASHTO LRFD Bridge Design Specifications and varying research. The phenomenon of liquefaction and how it should be evaluated is considered to be a complex issue that continues to be the subject of considerable research. It is fully anticipated that future research will further enhance and evolve how liquefaction should be quantified and evaluated. This design guide outlines the Department’s recommended procedure for identifying potentially liquefiable soils by estimating the seismic demands on soils and the capacity of soils to resist liquefaction. Also included are recommendations for characterizing the properties and behavior of liquefiable soils when conducting a structural seismic analysis.

Liquefaction Description and Design

Saturated loose to medium dense cohesionless soils and low plasticity silts tend to densify and consolidate when subjected to cyclic shear deformations inherent with large seismic ground motions. Pore-water pressures within such layers increase as the soils are consolidating resulting in a decrease in effective vertical pressure and loss of soil shear strength and stiffness. This loss in strength can result in large shear deformations and substantial vertical settlements may occur as the excess pore-water pressure dissipates and the volume/thickness of the liquefied layer decreases.

Embankments and foundations are susceptible to excessive deflection, damage, or failure if liquefiable soils are present. Such soils in their existing condition may have more than adequate capacity to carry everyday loadings. However, these same soils may have insufficient shear strength to resist the same service loads, much less any additional seismic forces, once liquefaction commences. The occurrence of liquefaction can dramatically affect foundations and embankments depending on the location of the liquefied soil layers. Deep foundations can sometimes be designed to withstand the effects of liquefaction depending upon the magnitude of liquefaction.
The seismic response and behavior of a structure and locations of potential plastic hinges will likely be different prior to and after liquefaction occurs. As such, structures located at sites where liquefaction is anticipated to occur shall be analyzed and designed for a nonliquefied configuration using site soil conditions in a nonliquefied state and for a liquefied configuration that reflects reduced or residual soil strengths and stiffness for the liquefied layers. The design spectra used for both configurations shall be the spectra determined for the nonliquefied configuration and the Site Class Definition shall be determined in accordance with the design guide for AGMU Memo 09.1. Liquefaction will typically decrease the stiffness of a substructure unit potentially increasing the shear demand on adjacent, stiffer substructure units.

Vertical ground settlement should be expected to occur following liquefaction. As such, spread footings should not be specified at sites expected to liquefy unless ground improvement techniques are employed to mitigate liquefaction concerns. For driven pile and drilled shaft foundations, the vertical settlement will result in a loss of skin friction capacity and an added negative skin friction (NSF) downdrag load, particularly when the liquefiable layers are overlain by competent compacted soil. Piles and shafts should be detailed to develop the required axial capacity below the anticipated liquefiable layers and should be proportioned accordingly to resist estimated NSF loads. NSF loads due to liquefaction settlement should be presented in the Structural Geotechnical Report (SGR).

Liquefaction is considered to be a function of the duration and magnitude of a seismic event and full liquefaction is not anticipated to occur simultaneously with the peak ground motion but rather after such motion has significantly subsided. When liquefiable layers are overlain by competent compacted material, dissipation of the excess pore-water pressures induced by the liquefaction is not likely to be instantaneous and is anticipated to occur over a short period of time. For such sites, NSF downdrag loads are not required to be considered simultaneously with the inertia forces obtained from the seismic analysis. Engineers should design the foundations to resist the additional NSF downdrag loads following a seismic event.

The lateral strength and resistance of liquefiable soils for pile and shaft foundations is anticipated to degrade at the onset of a seismic event. This is anticipated to have a significant effect on the lateral stiffness of the structure and the forces induced into pile and shaft foundations and may result in excessive deflection, bending, or buckling of the foundation. Therefore the lateral stiffness and forces induced into such foundations should be determined
using p-y curves and programs such as COM624 or LPILE. Modeling the effects of liquefaction is beyond the applicability of the estimated fixity depths indicated in Design Guide 3.15 (Seismic Design) for steel and concrete piles and should not be used for analyzing structures located at sites with liquefiable soils. The liquefied soil layers should be assumed to have a residual strength and may be modeled as a weak cohesive soil.

Embankments and bridge cones are susceptible to lateral movement in addition to vertical settlement during a seismic event. Lateral stability of embankments and bridge cones should be analyzed against seismic loading and liquefaction using recognized slope stability software. A flow or slope failure of an embankment or bridge cone may occur due to liquefaction. Such behavior is not anticipated to occur simultaneously with peak ground motion and should be decoupled similar to that previously mentioned for NSF downdrag loads. As such, the ability of embankments and bridge cones to resist such failures should be investigated for the existing static stresses using residual strength properties for the liquefied soil layers as described later on in the design guide. The lateral stability of embankments and bridge cones during peak ground motion need only be assessed using non-liquefied soil properties. AGMU Memo 10._ (New Slope Stability Design Criteria for Bridges and Roadways) provides further guidance on the seismic analysis of embankments when only non-liquefied soil properties are being considered.

Liquefaction Analysis Criteria

The need to conduct a liquefaction analysis depends upon the location of the site and geotechnical conditions present. All sites located in Seismic Performance Zones (SPZ) 3 and 4 and sites located in SPZ 2 with a peak seismic ground surface acceleration, $A_S$ (PGA modified by the zero-period site factor, $F_{pga}$), equal to or greater than 0.15 shall require consideration of the geotechnical conditions present and potential for liquefaction. If the geotechnical conditions for these sites are characterized by an anticipated groundwater level within 50 ft of either the existing or final ground surface (whichever is lower) and low plasticity silts or cohesionless soils with a corrected standard penetration test (SPT) blow count ($N_1$) less than or equal to 25 blows/ft are present within the upper 60 ft of the geotechnical profile, a liquefaction analysis shall be conducted.
The groundwater elevation considered should be reflective of the seasonally averaged groundwater elevation for the site. The SGR author should assess and use an adjusted groundwater elevation if the groundwater elevation encountered during the soil boring drilling is not considered reflective of a seasonally averaged groundwater elevation for the site.

Low plasticity silts and clays represent cohesive or fine-grain soils that reflect liquefaction tendencies similar to cohesionless soils. Fine-grain soils with a plasticity index (PI) less than 12 and water content (w_c) to liquid limit (LL) ratio greater than 0.85 are considered potentially liquefiable and require a liquefaction analysis. Cohesionless soils with an (N)_{60} greater than 25 blows/ft are not considered susceptible to liquefaction.

While PI is regularly investigated for pavement subgrades, it has rarely been considered in the past for structure soil borings. With the implementation of using PI to classify fine-grain soils potentially susceptible to liquefaction, the plasticity of such soils should be examined when conducting structure soil borings. Drillers should inspect and describe the plasticity of fine-grain soil samples. Low plasticity fine-grain soils, particularly loams and silty loams, should be retained for the necessary Atterberg Limit testing with the results indicated on the soil boring log.

For typical projects, liquefaction analysis shall be limited to the upper 60 ft of the geotechnical profile measured from the existing or final ground surface (whichever is lower). This depth encompasses a significant number of past liquefaction observations used to develop the following simplified liquefaction analysis procedure. On certain projects involving critical routes, the Department may elect to assess liquefaction potential at greater depths using more refined procedures. These projects will be identified by the Department and handled on a case-by-case basis.

If the liquefaction analysis indicates that the factor of safety (FS) against liquefaction is greater than or equal to 1.0 for all soil layers within the upper 60 feet of the geotechnical profile, no further consideration of liquefaction is necessary. If the analysis identifies soil layers with a FS less than 1.0 against liquefaction, the potential effects of liquefaction on the performance of a structure must be considered and/or potential ground modifications to mitigate potential effects must be investigated. Ground modification techniques to improve liquefaction resistance and their cost to benefit ratio will be assessed by the Department on a case-by-case basis.
Following is a Liquefaction Analysis Procedure illustrating how the soil layers should generally be analyzed to determine the FS against liquefaction.

**Liquefaction Analysis Procedure**

For typical projects, the “Simplified Method” developed by Youd et al. (2001) shall generally be used to investigate liquefaction potential. As advancements in research have been made in the recent years, some refinements suggested by Cetin et al. (2004) have been adopted by the Department and are contained herein. The simplified method compares the cyclic resistance ratio (CRR), the resistance of a soil layer against liquefaction, to the cyclic stress ratio (CSR), the seismic demand on a soil layer, to estimate the FS of a given soil layer against liquefaction. When investigating liquefaction, a FS should be assessed for each soil sample within the previously mentioned 60 ft depth.

The FS against liquefaction for a given soil layer shall be analyzed using the following procedure. An excel spreadsheet that follows the procedure indicated below has been prepared to assist Geotechnical Engineers with conducting a liquefaction analysis and may be downloaded from IDOT’s website.

\[
FS = \frac{CRR}{CSR}
\]

Where:

\[
CRR = CRR_{7.5}K_{\sigma}K_{MSF}
\]

\[
CSR = 0.65A_s\left(\frac{\sigma_{vo}}{\sigma_{vo}}\right)^{r_d}
\]

\[
CRR_{7.5} = \text{cyclic resistance ratio for magnitude 7.5 earthquake}
\]

\[
= \frac{1}{34 - (N_{1})_{60cs}} + \frac{(N_{1})_{60cs}}{135} + \frac{50}{[10(N_{1})_{60cs} + 45]^2} - \frac{1}{200}
\]

\[
K_{\sigma} = \text{overburden correction factor}
\]

\[
= \left(\frac{\sigma_{vo}}{2.12}\right)^{t-1} \text{ and } 1.5 \leq K_{\sigma} \leq 9^{t-1}
\]
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\( f = \) soil relative density factor

\[ f = 0.831 - \frac{(N)}{160} \text{ and } 0.6 \leq f \leq 0.8 \]

\( K_g = \) sloping ground correction factor

= 1.0 for generally level ground surfaces. See the following discussions for liquefaction investigation of slopes and embankments.

\( \text{MSF} = \) magnitude scaling factor

= 87.2\((M_w)^{-2.21}\)

\( M_w = \) earthquake magnitude. A description of the \( M_w \) values to be used for the liquefaction analysis is provided below.

\( A_s = \) peak horizontal acceleration coefficient at the ground surface

= \( F_{pga} \) PGA

\( F_{pga} = \) site factor at zero-period on acceleration response spectrum

(LRFD Article 3.10.3.2)

\( \text{PGA} = \) peak seismic ground acceleration on rock (Site Class B). A description of the PGA values to be used for the liquefaction analysis is provided below.

\( \sigma_{vo} = \) total vertical soil pressure for final condition (ksf)

\( \sigma'_{vo} = \) effective vertical soil pressure for final condition (ksf)

\( \sigma_{vo} \) and \( \sigma'_{vo} \) may be calculated using the following correlations for estimating the unit weight of soil (kcf):

Above water table:

\[
\begin{align*}
\gamma_{\text{granular}} &= 0.095N_m^{0.095} \\
\gamma_{\text{cohesive}} &= 0.1215Q_u^{0.095}
\end{align*}
\]

Below water table:

\[
\begin{align*}
\gamma_{\text{granular}} &= 0.105N_m^{0.07} - 0.0624 \\
\gamma_{\text{cohesive}} &= 0.1215Q_u^{0.095} - 0.0624
\end{align*}
\]

\( r_d = \) soil shear mass participation factor

\[
\begin{align*}
r_d &= \frac{1 + \frac{-23.013 - 2.949A_s + 0.999M_w + 0.016V_{s,40}'}{16.258 + 0.201e^{0.104[0.0785V_{s,40} + 24.888]}}}{1 + \frac{-23.013 - 2.949A_s + 0.999M_w + 0.016V_{s,40}'}{16.258 + 0.201e^{0.104[0.0785V_{s,40} + 24.888]}}} 
\end{align*}
\]

for \( d < 65 \text{ ft} \)
The shear wave velocity (ft/sec) of soils may be estimated using the following relationship, \( V_s = 169N_{m}^{0.516} \). Fill soils may be assumed to have a shear wave velocity of 600 ft/sec unless more exacting data is available.

\[
\begin{align*}
V_{s,40}^- &= \frac{1 + \frac{-23.013 - 2.949A_S + 0.999M_w + 0.016V_{s,40}^-}{16.258 + 0.201e^{0.104(0.465 + 0.0785V_{s,40}^- + 24.888)}}}{1 + \frac{-23.013 - 2.949A_S + 0.999M_w + 0.016V_{s,40}^-}{16.258 + 0.201e^{0.104(0.0785V_{s,40}^- + 24.888)}}} - 0.0014(d - 65) \text{ for } d \geq 65 \text{ ft}
\end{align*}
\]

\( V_{s,40}^- \) = average shear wave velocity within the top 40 ft of the finished grade (ft/sec).

\( d \) = depth of the soil sample below the finished grade (ft)

\( (N_i)_{60cs} \) = corrected \( (N_i)_{60} \) to an equivalent clean sand value (blows/ft)

\( \alpha \) = clean sand correction factor coefficient

- 0 for FC \leq 5%
- \( e^{\left(1.76\, \frac{190}{FC^2}\right)} \) for 5% < FC < 35%
- 5 for FC \geq 35%

\( \beta \) = clean sand correction factor coefficient

- 1.0 for FC \leq 5%
- \( 0.99 + \frac{FC^{1.5}}{1000} \) for 5% < FC < 35%
- 1.2 for FC \geq 35%

FC = % passing No. 200 sieve

\( (N_i)_{60} \) = corrected SPT blow count (blows/ft)

\( N_m C_N C_E C_o C_R C_S \) = field measured SPT blow count recorded on the boring logs (blows/ft)

\( C_N \) = overburden correction factor

\( C_E \) = hammer energy rating correction factor

\[ C_E = \frac{ER}{60}; \ ER = \text{hammer efficiency rating} \]
C_B = borehole diameter correction factor

= 1.0 for boreholes approximately 2\frac{1}{2} to 4\frac{1}{2} inches in diameter

= 1.05 for boreholes approximately 6 inches in diameter

= 1.15 for boreholes approximately 8 inches in diameter

C_R = rod length correction factor

= \frac{3.54659611 \times 10^{-5}}{101.4538} \frac{10^{-5}}{1025.7} \frac{10^{-5}}{1033.2} \frac{10^{-5}}{10615} \frac{10^{-5}}{3996.9} \frac{10^{-5}}{911.4} \frac{22}{3}

C_S = split-spoon sampler lining correction factor

= 1.0 for samplers with liners

= 1 + \frac{C_N N_m}{100} for samplers without liners where 1.1 \leq C_S \leq 1.3

ER = hammer efficiency rating (%)

\ell = drill rod length (ft) measured from the point of hammer impact to tip of sampler.

\ell \: may \: be \: estimated \: as \: the \: depth \: below \: the \: top \: of \: boring \: for \: the \: soil \: sample under \: consideration \: plus \: 5 \: ft \: to \: account \: for \: protrusion \: of \: the \: drill \: rod \: above \: the top \: of \: borehole.

SPT tests are generally conducted in accordance with AASHTO T 206. For soils explorations conducted by IDOT, boreholes are typically advanced using hollow stem augers that are 8 inches in diameter or using wash boring methods with a cutting bit that results in approximately a 4\frac{1}{2} inch diameter borehole. The diameter and methods of advancing the borehole can vary between Districts and Consultants who are performing soils explorations for IDOT. As such, it is recommended that the diameter of the borehole be included on the soil boring log in addition to the drilling procedure (hollow stem auger, mud rotary, etc.). Geotechnical engineers conducting a liquefaction analysis and calculating the borehole diameter correction factor (Q_B) should inquire with the engineer conducting the soils exploration if the diameter of the borehole is unknown.

The barrel of split-spoon samplers conforming to AASHTO T 206 are designed to accept a metal or plastic liner for collecting and transporting soil samples to the laboratory. Omitting the liners results in an enlarged internal barrel diameter that reduces the friction between the soil sample and interior of the sampler resulting in a reduction in measured penetration resistance. Past experience indicates that interior liners are seldom used and the AASHTO T 206
The split-spoon sampler lining correction factor ($C_L$) shall be calculated for samplers without liners unless otherwise indicated in the recorded soil boring data.

The field measured SPT blow count values obtained in Illinois commonly use an automatic type hammer which typically offer hammer efficiency (ER) values greater than the standard 60% associated with drop type hammers. For soils exploration conducted with automatic type hammers, an ER of 73% may be assumed unless more exacting information is available.

Also, research suggests that liquefaction resistance of cohesionless soils improves with increased fines content. As such, sieve analysis should be conducted for generally cohesionless soil layers present within the upper 60 ft of the geotechnical profile, with an $\langle N_{60} \rangle$ less than or equal to 25 blows/ft, and below the anticipated groundwater elevation to determine the representative % fines (FC) passing a No. 200 sieve. This data should be included in the SGR and/or reflected on the soil boring log.

### $M_w$ and PGA Values for Liquefaction Analysis

PGA and spectral acceleration values typically specified on structure plans and commonly obtained using U.S. Geological Survey (USGS) tools are the result of what is known as a probabilistic seismic hazard analysis (PSHA). PSHA is a statistical culmination of multiple potential seismic events taking into consideration a wide range of magnitudes, site to source distances, and estimated rates of occurrence for each potential earthquake scenario. The hazard (i.e., return period or probability of exceedance) affects the rate at which these scenarios are combined to determine the above mentioned PGA or spectral acceleration values (referred to hereafter as the PSHA PGA). These values are typically the only values necessary for a structural engineer to conduct a dynamic seismic analysis for a structure.

Unlike the dynamic seismic analysis of a structure, a liquefaction analysis is dependent upon two ground motion parameters, PGA and $M_w$. Duration of the ground motion is a key component in analyzing liquefaction potential and is essentially quantified as a function of $M_w$. In the past, IDOT has typically conducted liquefaction analysis using the PSHA PGA and by setting the $M_w$ equal to the value reported by the USGS as the Mean Earthquake Moment Magnitude ($\bar{M_w}$). The PSHA PGA and $\bar{M_w}$ are determined independently and typically not
considered to be a likely combination for a seismic event. Also, use of only $M_w$ may not fully capture the liquefaction hazard.

The PGA and $M_w$ pairs to be used for the liquefaction analysis should be determined from the deaggregation data of the seismic hazard for a given site. The deaggregation data provides a summary of the contribution of the various earthquake scenarios to the hazard and may be accessed at the following USGS web site: http://eqint.cr.usgs.gov/deaggint/2008/.

Portions of southern Illinois may possess ground motion characteristics that are considered multi-modal meaning that there are multiple earthquake scenarios that have a significant contribution to the hazard. Such locations require that liquefaction analysis be performed for multiple PGA and $M_w$ pairs. In Illinois, the multi-modal relationship is often characterized by a far source-site capable of producing a seismic event with a large $M_w$ and smaller PGA and a near source-site capable of producing a seismic event with a smaller $M_w$ but larger PGA. The far source-site will almost always be controlled by the New Madrid Seismic Zone (NMSZ). The near source-site will typically be controlled by “background seismicity” sources gridded by the USGS or the Wabash Fault Zone for locations in the southeastern part of Illinois. As locations progress towards the southern tip of Illinois, the multi-modal liquefaction hazard dissipates and the liquefaction hazard will be controlled solely by the NMSZ.

The deaggregation data contains data for a Modal source-site. The Modal source-site is considered to be the source-site with the largest contribution to the hazard. This data should be used to determine the PGA and $M_w$ pair that is characteristic of the NMSZ or far source-site. The PGA for the Modal source-site shall be determined using the $M_w$ and the source-to-site distance ($R$, km) with the ground motion prediction equations (GMPE’s) established by the USGS for the NMSZ. The USGS uses a weighted average of 8 different ground GMPE’s for the NMSZ. Because of their complexity, the GMPE’s are not presented herein but rather the PGA values should be determined using the NMSZ GMPE’s included with IDOT’s Liquefaction Analysis excel spreadsheet.

The deaggregation data also contains an “ALL_EPS” column. This data contained in this column is indicative of the percent contribution of each earthquake scenario to the hazard for a given site. Earthquake scenarios that contribute 5% or more to the hazard and do not have a source-to-site distance indicative of the NMSZ should also be considered in the liquefaction
analysis. These earthquake scenarios are considered representative of “background seismicity” sources or the Wabash Valley Fault in southeastern Illinois. The PGA values for these sites shall be determined using the $M_w$ and the source-to-site distance (R, km) with the ground motion prediction equations established by the USGS for the Central Eastern United States (CEUS). The USGS uses a weighted average of 7 different GMPE’s for the CEUS. Similar to the NMSZ, the GMPE’s for the CEUS are not presented herein but rather the PGA values should be determined using the CEUS GMPE’s included with IDOT’s Liquefaction Analysis excel spreadsheet.

Two examples for interpreting the deaggregation data and determining the PGA and $M_w$ pairs to be used for the liquefaction analysis are included at the end of the design guide.

**Liquefaction Analysis Procedure for Slopes and Embankments**

The liquefaction resistance of dense granular materials under low confining stress (dilative soils) tends to increase with increased static shear stresses. Such static shear stresses are typically the result of ground surface inclinations associated with slopes and embankments. Conversely, the liquefaction resistance of loose soils under high confining stress (contractive soils) tends to decrease with increased static shear stresses. Such soils are susceptible to undrained strain softening. The effects of the sloping ground surface and the static shear stresses on the liquefaction resistance of soils is accounted for in the previously described Simplified Procedure by use of the sloping ground correction factor, $K_\alpha$.

$K_\alpha$ is a function of the static shear stress to effective overburden pressure ratio and relative density of the soil. Graphical curves have been published that correlate $K_\alpha$ with these variables (Harder and Boulanger 1997). With the exception of earth masses of a constant slope, the ratio of the static shear stress to effective overburden pressure will vary at different points under an embankment, and most slopes, making it difficult to determine an appropriate $K_\alpha$ factor when analyzing liquefaction potential at a single location. Researchers that developed the Simplified Procedure have indicated that there is a wide range of proposed $K_\alpha$ values indicating a lack of convergence and need for additional research. It is recommended that the graphical curves that have been published for establishing $K_\alpha$ not be used by nonspecialists in geotechnical earthquake engineering or in routine engineering practice.
Olson and Stark (2003) have presented an alternative analysis method for analyzing the effects of static shear stress due to sloping grounds on the liquefaction resistance of soils. A detailed description of the method is not included herein and Geotechnical Engineers should obtain a copy of the reference document for further information.

The method provides a numerical relationship for determining whether soils are contractive or dilative. If soils are determined to be contractive, additional analysis should be conducted to investigate the effects of static shear stress on the liquefaction resistance of soils. The additional analysis is an extension of a traditional slope stability analysis typically performed with commercial software. The additional analysis outlined in the referenced document can typically be easily facilitated with the use of a spreadsheet and data extrapolated from the software conducting the slope stability analysis. If the additional analysis indicates soil layers with a FS < 1.0 against liquefaction, a post-liquefaction slope stability analysis should be conducted with residual shear strengths assigned to the soil layers expected to liquefy. While Olson and Stark (2003) present one acceptable method for estimating the residual shear strength of liquefied soil layers, there are also a number of other methods presented in various reference documents concerning liquefaction or there are methods that are already embedded in the code of analysis software.

The Liquefaction Analysis excel spreadsheet published by the Department that estimates liquefaction resistance of the soil using the Simplified Method described above also estimates whether soils are contractive or dilative based upon the relationship provided by Olson and Stark (2003). As the classification of contractive or dilative soils is affected by overburden pressure, the presence of such soils should be assessed considering a soil column that starts at the top of the embankment/slope and another soil column that begins at the base of the embankment/slope.

Note that the method provided by Olson and Stark (2003) also includes an equation for estimating the seismic shear stress on a soil layer (Eq. 3a in the reference document). The variable $C_M$ included in the referenced equation shall be replaced with the variable MSF and both variables MSF and $r_d$ shall be calculated using the equations outlined above for the Simplified Method. The depth used to calculate $r_d$ should be the average depth of the critical failure surface within the potential liquefaction zone.
Example #1: Determining $M_w$ and PGA Values Near Grayville, IL

Figures 1a and 1b provide the deaggregation data obtained from the USGS website for a site near Grayville, IL.

There are five earthquake scenarios highlighted in the figures where the “ALL_EPS” or contributions to the hazard are greater than or equal to 5%. Three of the five sites have source-to-site distances indicative of the NMSZ. The hazard of these three sites is considered captured by the Modal source-site. Therefore in this particular example, there are three earthquake scenarios that should be considered for the liquefaction analysis as follows.

- EQ Scenario #1, Dist. (R) = 12.1 km, $M_w = 4.80 \rightarrow \text{PGA} = 0.175$ (CEUS Model)
- EQ Scenario #2, Dist. (R) = 12.6 km, $M_w = 5.03 \rightarrow \text{PGA} = 0.209$ (CEUS Model)
- EQ Scenario #3, Dist. (R) = 155.1 km, $M_w = 7.70 \rightarrow \text{PGA} = 0.111$ (NMSZ Model)

The PGA value for these earthquake scenarios have been determined using the IDOT Liquefaction Analysis excel spreadsheet and the indicated GMPE model.

In this instance, it is clear that EQ Scenario #2 will control over EQ Scenario #1. The PGA and $M_w$ pairs for EQ Scenario’s #2 and #3 serve as an example of the potential multi-modal nature of some locations. EQ Scenario #2 is indicative of a near source-site exhibiting a smaller $M_w$ yet is capable of producing a relatively high PGA. EQ Scenario #3 characterizes the far source-site that is typically represented by the NMSZ and has a larger $M_w$ and smaller PGA.

It is worthy of noting that while the NMSZ is capable of producing large PGA’s, the PGA’s associated with the NMSZ for a large portion of southern Illinois are less than the PSHA PGA due to the attenuation of the ground motion that occurs over the source-to-site distance. Recognizing this attenuation and calculating the deterministic PGA for such sites is expected to decrease the number of locations where liquefaction is estimated to occur.

There will be many instances where the deaggregation data indicates that there are no near source-sites or earthquake scenarios contributing at least 5% to the hazard that need to be considered for the liquefaction analysis using a PGA determined from the CEUS GMPE’s. In
such cases, the hazard is considered dominated by the NMSZ and only the Modal source-site needs considered.

**Figure 1a. Grayville Deaggregation Data.**
Example #2: Determining $M_w$ and PGA Values Near Cairo, IL

Figure 2 provides the deaggregation data obtained from the USGS website for a site near Cairo, IL.

There are three earthquake scenarios highlighted in the figure where the “ALL_EPS” or contributions to the hazard are greater than or equal to 5%. By inspection, these three combinations have source-to-site distances indicative of the NMSZ. The hazard of these combinations is considered captured by the Modal source-site. In this particular example, only the hazard defined by the NMSZ needs to be considered for liquefaction analysis as indicated below.

- EQ Scenario #1, Dist. ($R$) = 11.5 km, $M_w$ = 7.70 $\rightarrow$ PGA = 1.528 (NMSZ Model)

Similar to Example #1, the PGA value for the earthquake scenario has been determined using the IDOT Liquefaction Analysis excel spreadsheet and the indicated GMPE model.
In this example the PGA for the earthquake scenario to be used for the liquefaction analysis is larger than the PSHA PGA. This is due to the influence of the additional earthquake scenarios that are taken into consideration for the PSHA and have an estimated PGA less than PGA associated with the Modal source-site. Such results become increasingly apparent as project locations progress toward the extreme southern portion of Illinois.
Relevant References


