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 2017 AASHTO LRFD Bridge Design Specifications and various research. The phenomenon of liquefaction and how it should be evaluated continues to be the subject of considerable study and debate. It is expected that enhancements will evolve and modify how liquefaction should be evaluated and accounted for in design. This design guide outlines the Department's current recommended procedure for identifying potentially liquefiable soils. Also included are recommendations for characterizing the properties and behavior of liquefiable soils so that substructure stiffness and embankment response to seismic loading can be modeled.

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 cyclically loaded, resulting in a decrease in vertical effective stress and shear strength. If the shear strength drops below the applied cyclic shear loadings, the layer is expected to transition to a semi fluid state until the excess pore-water pressure dissipates.

Embankments and foundations are particularly susceptible to damage, depending on the location and extent of the liquefied soil layers. Such soils may adequately carry everyday loadings, however once liquefied, retain insufficient capacity for such loads or additional seismic forces. Substructure foundations shall either be designed to withstand the liquefaction or ground improvement techniques shall be used to achieve the IDOT performance objectives of no loss of life or loss of span. End slopes and roadway embankments on liquefiable soils require an analysis to determine the likely extent of pavement/slope damage so that the cost of ground improvement techniques can be compared to alternatives such as re-routing traffic around the damaged lanes or quickly effecting emergency repairs.

The stiffness of liquefiable soils supporting foundations is anticipated to degrade over the duration of the seismic event and reduces the lateral stiffness of the substructure. The reduced stiffness results in increased deflection and moment arm, concern for buckling, and potentially additional loading on adjacent substructures. The lateral stiffness, moments and forces carried by such foundations supported by liquefiable soils is best determined using programs such as COM624 or LPILE. The liquefied soil layers can be modeled in these programs with reduced strength parameters or the p-y curves can be modified to reflect the residual strength of the liquefied layers. Note that the estimated fixity depths indicated in Design Guide 3.15 (Seismic Design) should not be used for analyzing substructures with liquefiable soils.

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. 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 when the liquefiable layers are overlain by non-liquefiable soils. Geotechnical losses from liquefaction and any liquefaction induced NSF loadings shall only be considered with the Extreme Event I limit state group loading, since the strength limit state group loadings represent the conditions prior to, not after a seismic event.

Since liquefaction may or may not fully occur while the peak seismic bridge loadings are applied, structures at sites where liquefaction is anticipated must be analyzed and designed to resist the seismic loadings with nonliquefied conditions as well as a configuration that reflects the locations, extent and reduced strength of the liquefiable layers. However, the design spectra used for both configurations shall be the spectra determined for the nonliquefied configuration.

Embankments and bridge cones are susceptible to lateral movements in addition to vertical settlement during a seismic event. When the seismic slope stability factor of safety drops below 1.0, slope deformations become likely and when liquefaction is expected, these movements can be substantial. The ability of embankments and bridge cones to resist such failures when liquefiable soils are present should be investigated using the slope geometry and static stresses along with residual strength properties for the liquefied soils as described later in the design guide.

Liquefaction Analysis Criteria

All bridges located in Seismic Performance Zones (SPZ) 3 and 4 as well as 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, require liquefaction analysis. The exception to this is when the all liquefaction susceptible soils at a site have corrected standard penetration test (SPT) blow counts $(N_1)_{60}$ above 25 blows/ft. or the anticipated groundwater is not within 50 ft of the ground surface. The groundwater elevation used in the analysis should represent the higher likely groundwater considering the possible fluctuations that over the various seasons. The department recommends starting with the groundwater shown in the borings and using the date it was drilled, (say October), increasing the elevation to a wetter month (say April). This increase could be calculated similar to how the "estimated water surface elevation" (EWSE) is calculated.

Low plasticity silts and clays may experience pore-water pressure increases, softening, and strength loss during earthquake shaking similar to cohesionless soils. Fine-grained 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 liquefaction analysis. While PI is regularly investigated for pavement subgrades, it has rarely been considered in the past for structure soil borings. However, in order to investigate liquefaction susceptibility of fine-grained soils, the plasticity of such soils should be examined when conducting structure soil borings. Drillers should inspect and describe the plasticity of fine-grained soil samples. Low plasticity fine-grained soils, particularly loams and silty loams, should be retained for the 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 simplified liquefaction analysis procedure described below. If the liquefaction analysis indicates that the factor of safety (FS) against liquefaction is greater than or equal to 1.0, no further concern for liquefaction is necessary. However, if multiple soil layers are present indicating a FS substantially less than 1.0, the potential for these layers to liquefy and the effect on the slope or foundation must but be further evaluated.

Liquefaction Analysis Procedure

The method described below is provided to assist Geotechnical Engineers in facilitating liquefaction analysis for typical or routine projects. For simplicity, numerical expressions or directions are provided for determining values of the variables necessary to conduct the liquefaction analysis for such projects. Non-linear site response analysis programs can be used to determine more exacting values for some of the variables, however this should only be considered necessary for large or unique projects where a more refined liquefaction analysis is desired.

The "Simplified Method" described by Youd et al. (2001) as well as refinements suggested by Cetin et al. (2004) shall be used to estimate liquefaction potential as contained herein. The simplified method compares the resistance of a soil layer against liquefaction (Cyclic Resistance Ratio, CRR) to the seismic demand on a soil layer (Cyclic Stress Ratio, CSR) to estimate the FS of a given soil layer against triggering liquefaction. The FS for each soil sample should be computed to allow thin, isolated layers indicating a FS < 1.0 to be discounted and the specific locations and extent of those determined liquefiable to be indicated in the SGR and accounted for in design.

An Excel spreadsheet that performs these calculations 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 $= CRR_{7.5} K_{\sigma} K_{\alpha} MSF$

CSR $= 0.65 A_S \left(\frac{\sigma_{vo}}{\sigma'_{vo}} \right) r_d$

$CRR_{7.5}$ = cyclic resistance ratio for magnitude 7.5 earthquake

$$= \frac{1}{34 - (N_1)_{60cs}} + \frac{(N_1)_{60cs}}{135} + \frac{50}{\left[10(N_1)_{60cs} + 45\right]^2} - \frac{1}{200}$$

K_{σ} = overburden correction factor

$$= \left(\frac{\sigma'_{vo}}{2.12} \right)^{(f-1)} \text{ and } 9^{(f-1)} \le K_{\sigma} \le 1.5$$

f = soil relative density factor

$$= 0.831 - \frac{(N_1)_{60cs}}{160} \text{ and } 0.6 \le f \le 0.8$$

K_{α} = sloping ground correction factor

= 1.0 for generally level ground surfaces or slopes flatter than 6 degrees. See the following discussions for liquefaction evaluation of slopes and embankments.

MSF = magnitude scaling factor

$= 87.2(M_w)^{-2.215}$

M_w = earthquake moment magnitude.

A_S = peak horizontal acceleration coefficient at the ground surface

= $F_{pga} PGA$

F_{pga} = site amplification factor for zero-period spectral acceleration (LRFD Article 3.10.3.2)

PGA = peak seismic ground acceleration on rock.

σ_{vof} = total vertical soil pressure for final condition (ksf)

σ'_{vof} = effective vertical soil pressure for final condition (ksf)

σ_{vof}, σ'_{vof}, and σ'_{voi} may be calculated using the following correlations for estimating the unit weight of soil (kcf):

Above water table: $\gamma_{granular} = 0.095 N_m^{0.095}$

$\gamma_{cohesive} = 0.1215 Q_u^{0.095}$

Below water table: $\gamma_{granular} = 0.105 N_m^{0.07} - 0.0624$

$\gamma_{cohesive} = 0.1215 Q_u^{0.095} - 0.0624$

Fill soils being modeled for the final condition may be assumed to have unit weights of 0.120 kcf and 0.058 kcf above and below the water table.

r_d = soil shear mass participation factor

$$= \frac{\left[1 + \dfrac{-23.013 - 2.949 A_S + 0.999 M_w + 0.016 V^*_{s,40'}}{16.258 + 0.201 e^{0.104\left(-d + 0.0785 V^*_{s,40'} + 24.888\right)}}\right]}{\left[1 + \dfrac{-23.013 - 2.949 A_S + 0.999 M_w + 0.016 V^*_{s,40'}}{16.258 + 0.201 e^{0.104\left(0.0785 V^*_{s,40'} + 24.888\right)}}\right]} \text{ for } d < 65 \text{ ft}$$

$$= \frac{\left[1 + \dfrac{-23.013 - 2.949 A_S + 0.999 M_w + 0.016 V^*_{s,40'}}{16.258 + 0.201 e^{0.104\left(-65 + 0.0785 V^*_{s,40'} + 24.888\right)}}\right]}{\left[1 + \dfrac{-23.013 - 2.949 A_S + 0.999 M_w + 0.016 V^*_{s,40'}}{16.258 + 0.201 e^{0.104\left(0.0785 V^*_{s,40'} + 24.888\right)}}\right]} - 0.0014\left(d - 65\right) \text{ for } d \geq 65 \text{ ft}$$

$V^*_{s,40'}$ = average shear wave velocity within the top 40 ft of the finished grade (ft/sec).

$$= \frac{40}{\sum_{i=1}^{n} \dfrac{d_i}{v_{si}}}$$

v_{si} = shear wave velocity of individual soil layer (ft/sec)

$= 169N_m^{0.516}$

Fill soils may be assumed to have a shear wave velocity of 600 ft/sec.

d_i = thickness of individual soil layer (ft)

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

$(N_1)_{60cs}$ = $(N_1)_{60}$ adjusted to an equivalent clean sand value (blows/ft)

$= \alpha + \beta(N_1)_{60}$

α = clean sand adjustment factor coefficient

= 0 for $FC \leq 5\%$

$= e^{\left(1.76 - \frac{190}{FC^2}\right)}$ for $5\% < FC < 35\%$

= 5 for $FC \geq 35\%$

β = clean sand adjustment 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

$(N_1)_{60}$ = corrected SPT blow count (blows/ft)

$= N_m C_N C_E C_B C_R C_S$

N_m = field measured SPT blow count recorded on the boring logs (blows/ft)

C_N = overburden correction factor

$$= \frac{2.2}{\left(1.2 + \frac{\sigma'_{voi}}{2.12}\right)} \leq 1.7$$

σ'_{voi} = effective vertical soil pressure during drilling (ksf)

C_E = hammer energy rating correction factor

$= \frac{ER}{60}$; ER = 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

$$= (-2.1033\times10^{-11})\lambda^6 + (7.9025\times10^{-9})\lambda^5 - (1.2008\times10^{-6})\lambda^4 + (9.4538\times10^{-5})\lambda^3 - (4.0911\times10^{-3})\lambda^2 + (9.3996\times10^{-2})\lambda + 0.0615 \text{ and } 0.75 \le C_R \le 1.0$$

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 $1.1 \le C_S \le 1.3$

ER = hammer efficiency rating (%)

Unless more exacting information is available, use 73% for automatic type hammers and 60% for conventional drop type hammers.

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

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 4½ inch diameter borehole. The diameter and methods of advancing the borehole can vary between Districts and Consultants performing soils explorations for IDOT. As such, it is recommended that the borehole diameter 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 (C_B) should inquire with the soils exploration provider if the borehole diameter is not provided.

SPT tests are generally conducted in accordance with AASHTO T 206 and the split-spoon samplers are designed to accept a metal or plastic liner for collecting and transporting soil samples to the laboratory. Omitting the liner provides an enlarged internal barrel diameter that reduces friction between the soil sample and interior of the sampler, resulting in a reduced SPT blow count. Past experience indicates that interior liners are seldom used and the AASHTO T 206 specification indicates that the use of liners is to be noted on the penetration record. Thus, it shall be assumed in the calculation of the split-spoon sampler lining correction factor (C_S) that liners were not used unless otherwise indicated the soil boring log.

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 the specific drill rig hammer energy is known.

Liquefaction resistance improves with increased fines content. As such, sieve analysis should be conducted for low plasticity fine-grained loams and silts below the anticipated groundwater elevation and within the upper 60 ft when the $(N_1)_{60}$ is less than or equal to 25 blows/ft to determine percent passing a No. 200 sieve (Fines Content, FC). These data should be included in the SGR and/or reported on the soil boring log.

M_w and PGA Values for Liquefaction Analysis

The spectral accelerations for the 0.0 second, 0.2 second and 1.0 second structure period are typically used by the structural engineer to conduct a pseudo-static seismic analysis and design of the bridge and foundation elements. These are commonly obtained from U.S. Geological Survey (USGS) maps which were developed using a probabilistic seismic hazard analysis (PSHA). PSHA estimates the likelihood that various seismic accelerations will be exceeded at a given site, over a future specific period of time, by analyzing various potential seismic sources, earthquake magnitudes, site to source distances, and estimated rates of occurrence. With this methodology, as the desired probability of exceedance is decreased (or design return period is increased), the corresponding spectral accelerations increase. The 0.0 second spectral acceleration is commonly considered as the PGA (hereafter referred to as the PSHA PGA) for the structure's design return period.

In addition to PGA, duration of shaking is a key factor in triggering liquefaction and is represented in the liquefaction analysis procedure by the earthquake Moment Magnitude (M_w). In the past, IDOT used the PSHA PGA with the Mean Earthquake Moment Magnitude provided by the USGS for the site location and design return period. However, it was determined that this PGA and M_w combination may not properly identify a site's liquefaction potential for the design return period. This is due to portions of Illinois being considered multi-modal, meaning that there are multiple earthquake sources that have a significant contribution to the overall hazard. Thus, the liquefaction potential at a site must be checked for multiple PGA and M_w pairs to determine the controlling values. Multi-modal conditions are characterized by a distant seismic source likely to produce a large M_w but the PGA at the site would be relatively low due to the distance, and a near-site source with a smaller M_w and larger PGA. The distant seismic source will almost always be the New Madrid seismic zone (NMSZ). The near-site source will typically be the “background seismicity” sources gridded by the USGS, although the Wabash Valley seismic zone (WVSZ) will control the near-site source for some sites in southeastern Illinois. Sites near the southern most portion of the state become less multi-modal and are solely controlled the NMSZ. The PGA and M_w values to be checked at a site can be identified by downloading the USGS deaggregation report using the Dynamic Conterminous U.S. 2014 (v4.1.1) data, located at: https://earthquake.usgs.gov/hazards/interactive/. The report provides the distance (rRup) to the numerous potential seismic sources, the M_w of each source and the percent contribution (ALL_ε) that source provides to the overall hazard at a site.

The rRup and M_w values in the report to be investigated will have an “ALL_ε” larger than 5%. The PGA to be used with each selected rRup and M_w pair shall be calculated using the USGS ground motion prediction equations. Scenarios with a source to site distance not extending to the NMSZ can be identified as near-site sources which use the Central Eastern United States (CEUS) GMPE’s equations while distant seismic sources should utilize the “NMSZ” equations. These equations are programmed into the IDOT Liquefaction Analysis spreadsheet to provide the appropriate PGA values for each rRup and M_w pair when either the “CEUS” or the “NMSZ” is selected.

Two examples for using 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 sloping ground and 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_α.

K_α 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_α 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_α. Researchers that developed the Simplified Procedure have indicated that there is a wide range of proposed K_α 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_α not be used by nonspecialists in geotechnical earthquake engineering or in routine engineering practice.

Olson and Stark (2003) have presented an alternative approach for analyzing the effects of static shear stress due to sloping ground 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, an 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, and can be readily facilitated with the use of a spreadsheet and data obtained from the slope stability software. 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.

The Department's Liquefaction Analysis spreadsheet that estimates liquefaction resistance of 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.

Examples for Determining the Controlling M_w and PGA Value

The first of the two examples is for a location near Mount Vernon, Illinois. Figure 1 below shows is the deaggregation report placed in a spreadsheet and sorted by descending percent contribution (ALL_ε). In this case, the five earthquake sources highlighted in the figure have a contribution to the total hazard greater than 5% which are highlighted in the lower box below.

*** Deaggregation of Seismic Hazard at One Period of Spectral Acceleration ***
*** Data from Dynamic: Conterminous U.S. 2014 (v4.1.1) ****
PSHA Deaggregation. %contributions.
site: Mount Vernon, IL.
longitude: 88.909°W
latitude: 38.263°E
imt: Peak ground acceleration
vs30 = 760 m/s (B/C boundary)
return period: 975 yrs.
#This deaggregation corresponds to: Total
Summary statistics for PSHA PGA deaggregation, r=distance, ε=epsilon:
Deaggregation targets:
Return period: 975 yrs
Exceedance rate: 0.001025641 yr⁻¹
PGA ground motion: 0.19615166 g
Mode (largest r-m bin):
r: 196.36 km
m: 7.77
εo: 0.94 σ
Contribution: 6.78 %
Mode (largest εo bin):
r: 145.45 km
m: 7.52
εo: 0.71 σ
Contribution: 2.82 %

Closest Distance, rRup (km)	Magnitude (Mw)	ALL_ε	ε=[2.5,∞)	ε=[2,2.5)	ε=[1.5,2)	ε=[1,1.5)	ε=[0.5,1)	ε=(-∞,0.5)	ε=[-0.5,∞)	ε=[-1,-0.5)	ε=[-1.5,-1)
190	7.7	6.783	1.941	2.334	1.57	0.826	0.009	0.101	0	0	0
10	4.9	6.657	0.227	0.219	0.652	0.968	1.102	1.921	1.083	0.343	0.142
150	7.5	6.013	1.967	2.823	0.765	0.249	0.21	0	0	0	0
10	5.1	5.163	0.044	0.156	0.279	0.529	0.922	1.396	1.136	0.431	0.269
110	7.5	5.126	0.927	2.791	1.185	0.056	0.096	0.071	0	0	0
10	4.7	3.939	0.11	0.107	0.318	0.48	0.653	1.04	0.926	0.253	0.052
10	5.3	3.865	0.127	0.111	0.138	0.682	0.666	1.006	0.741	0.305	0.087
130	7.5	3.195	2.129	0.742	0.223	0.001	0.101	0	0	0	0
130	7.3	2.911	0.493	1.773	0.535	0.025	0.085	0	0	0	0
10	5.5	2.801	0.08	0.16	0.176	0.285	0.987	0.437	0.393	0.284	0
190	7.5	2.744	0.801	1.417	0.313	0.178	0.035	0	0	0	0
190	7.7	2.406	1.941	0.465	0	0	0	0	0	0	0
190	7.9	2.076	0.335	0.494	0.798	0.279	0.128	0.034	0.008	0	0

Figure 1. Mount Vernon Illinois Deaggregation Report.

The upper box “Mode (largest r-m bin)” provides more precise rRup and M_w data for the largest contributing source and can be used in lieu of maximum contributor in the lower box.

In the lower box, three of the five sites have source-to-site distances and M_w values indicative of the NMSZ while the remaining two earthquake scenarios are considered near-site sources. Of the two near-site cases, the one with a M_w of 4.9 can be discarded since the other near-site M_w is higher and will control. The NMSZ source with a distance of 150 km can also be neglected since the closer source at 110 km would produce a higher PGA given they have the same magnitude. The remaining three rRup and M_w pairs will need to be checked in the IDOT Liquefaction Analysis spreadsheet, using the appropriate GMPE model, to determine PGA at the site for each case. The largest PGA will generally cause the most liquefaction, but all three cases should be checked since M_w also plays into the analysis.

The is a good example of the multi-modal nature of some locations in Illinois. However, there will be many instances where the deaggregation data indicates that only near-site sources or only NMSZ sources contribute more than 5% which is shown in the next example.

The second example is for a location near Cairo, Illinois and the site deaggregation data is provided in below in Figure 2. There are three highlighted earthquake scenarios where the "ALL_ε" contribution is greater than 5%.

*** Data from Dynamic: Conterminous U.S. 2014 (v4.1.1) ****										
PSHA Deaggregation. %contributions.										
site: Cairo, Illinois										
longitude: 89.176°W										
latitude: 37.007°E										
imt: Peak ground acceleration										
vs30 = 760 m/s (B/C boundary)										
return period: 975 yrs.										
#This deaggregation corresponds to: Total										
Summary statistics for PSHA PGA deaggregation, r=distance, ε=epsilon:										
Deaggregation targets:										
Return period: 975 yrs										
Exceedance rate: 0.001025641 yr^{-1}										
PGA ground motion: 0.94946485 g										
Mode (largest r-m bin):										
r: 13.11 km										
m: 7.52										
$ε_o$: -0.5 σ										
Contribution: 37.6 %										
Mode (largest $ε_o$ bin):										
r: 12.68 km										
m: 7.52										
$ε_o$: -1.11 σ										
Contribution: 11.44 %										
Closest Distance, rRup (km)	Magnitude (Mw)	ALL_ε	ε=[2.5,∞)	ε=[2,2.5)	ε=[1.5,2)	ε=[1,1.5)	ε=[0.5,1)	ε=(-∞,0.5)	ε=[-0.5,∞)	ε=[-1,-0.5)
10	7.5	37.603	11.443	10.562	10.114	1.252	1.742	2.134	0.352	0.004
10	6.9	11.897	1.279	3.395	3.181	1.959	1.268	0.648	0.164	0.003
10	7.3	11.822	4.241	1.388	3.712	1.021	1.047	0.368	0.041	0.004
10	7.1	4.516	1.116	1.114	1.087	0.677	0.219	0.236	0.066	0.001
10	7.7	4.213	1.447	1.855	0.192	0.24	0.33	0.141	0.006	0.002
10	6.7	3.925	0.133	1.173	1.1	0.794	0.383	0.199	0.14	0.003
10	4.9	2.477	0.719	0.231	0.978	0.286	0.238	0.023	0	0
10	6.5	2.406	0.646	0.616	0.626	0.272	0.192	0.05	0.004	0.001
50	5.1	2.272	0.76	0.338	0.643	0.397	0.109	0.024	0	0
70	7.7	2.023	1.324	0.687	0.002	0.011	0	0	0	0
10	5.3	2.002	0.582	0.569	0.51	0.269	0.055	0.017	0	0
30	5.5	1.708	0.428	0.2	0.427	0.429	0.164	0.057	0.004	0
10	5.7	1.413	0.16	0.2	0.188	0.464	0.268	0.107	0.026	0
90	4.7	1.29	0.302	0.243	0.42	0.177	0.143	0.006	0	0
130	6.1	1.236	0.245	0.208	0.372	0.245	0.126	0.032	0.007	0
10	5.9	1.135	0.208	0.125	0.21	0.363	0.15	0.061	0.016	0.001

Figure 2. Cairo Deaggregation Data.

By inspection, all three have source-to-site distances and magnitudes indicative of the NMSZ. With the distances being the same, only the highest magnitude source need be checked using the Mode (largest r-m bin) r and m combination.

Like Example #1, the PGA value to be used with this earthquake magnitude must be determined using the IDOT Liquefaction Analysis Excel spreadsheet and the indicated GMPE model.

Relevant References

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