Transportation Consortium of South-Central States

Solving Emerging Transportation Resiliency, Sustainability, and Economic Challenges through the Use of Innovative Materials and Construction Methods: From Research to Implementation

Seismic Ground Motion Data Analyses for North-East Arkansas

Project No. 19GTASU01

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Final Report

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16. Abstract

The New Madrid Fault Zone (NMFZ) passes through the middle of northeast Arkansas (NEA), which is located in the western part of the upper Mississippi Embayment (ME) area. With the history of major earthquakes and the merge of ME and NMFZ in the region, the understanding of the nature of ground responses and prospects of liquefaction hazards in NEAR has been a complex problem. According to the American Association of State Highway Officials (AASHTO), the Arkansas Geological Survey (AGS), and the Federal Emergency Management Agency (FEA), the NEA area is under Site Class F and most of the area has liquefaction susceptibility ranging from high to very high. Furthermore, this area has a history of sand boiling. Thus, the Site Specific Ground Response Analysis (SSGRA) and Cyclic Liquefaction Analysis (CFA) are the basic requirements for the Arkansas Department of Transportation (ARDOT) and other agencies before building any structures in the area. As part of a recent ARDOT's Transportation Research Committee (TRC) project, researchers have conducted geophysical investigations several construction sites in NEA over the past twelve years. The current study gathered the previously reported test data and estimated seismic site coefficients for five locations near the city of Jonesboro and cyclic liquefaction analysis for 131 sites in northeast Arkansas. The analysis results will guide the bridge design engineers to estimate suitable site factors for designing bridges near Jonesboro, AR. The developed liquefaction risk maps will also help the engineers (bridge, utility, and foundation) in estimating liquefaction potentials and associated risks in northeast Arkansas.

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SI* (MODERN METRIC) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
LENGTH				
in	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km
AREA				
in^2	square inches	645.2	square millimeters	mm^2
ft^2	square feet	0.093	square meters	m^2
yd^2	square yard	0.836	square meters	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	square kilometers	km^2
VOLUME				
fl oz	fluid ounces	29.57	milliliters	mL
gal	gallons	3.785	liters	L
ft^3	cubic feet	0.028	cubic meters	m^3
yd^3	cubic yards	0.765	cubic meters	m^3
NOTE: volumes greater than 1000 L shall be shown in m^3				
MASS				
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams (or "metric ton")	Mg (or "t")
TEMPERATURE (exact degrees)				
^oF	Fahrenheit	5 (F-32)/9 or (F-32)/1.8	Celsius	^oC
ILLUMINATION				
fc	foot-candles	10.76	lux	lx
fl	foot-Lamberts	3.426	candela/m^2	cd/m^2
FORCE and PRESSURE or STRESS				
lbf	poundforce	4.45	newtons	N
lbf/in^2	poundforce per square inch	6.89	kilopascals	kPa

APPROXIMATE CONVERSIONS FROM SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
LENGTH				
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
AREA				
mm^2	square millimeters	0.0016	square inches	in^2
m^2	square meters	10.764	square feet	ft^2
m^2	square meters	1.195	square yards	yd^2
ha	hectares	2.47	acres	ac
km^2	square kilometers	0.386	square miles	mi^2
VOLUME				
mL	milliliters	0.034	fluid ounces	fl oz
L	liters	0.264	gallons	gal
m^3	cubic meters	35.314	cubic feet	ft^3
m^3	cubic meters	1.307	cubic yards	yd^3
MASS				
g	grams	0.035	ounces	oz
kg	kilograms	2.202	pounds	lb
Mg (or "t")	megagrams (or "metric ton")	1.103	short tons (2000 lb)	T
TEMPERATURE (exact degrees)				
^oC	Celsius	1.8C+32	Fahrenheit	^oF
ILLUMINATION				
lx	lux	0.0929	foot-candles	fc
cd/m^2	candela/m^2	0.2919	foot-Lamberts	fl
FORCE and PRESSURE or STRESS				
N	newtons	0.225	poundforce	lbf
kPa	kilopascals	0.145	poundforce per square inch	lbf/in^2

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ACRONYMS, ABBREVIATIONS, AND SYMBOLS

AASHTO - American Association of State Highway and Transportation Officials

AGS – Arkansas Geological Society

ARDOT – Arkansas Department of Transportation

CPT - Cone Penetration Test

FEMA - Federal Emergency Management Agency

FHWA – Federal Highway Administration

GMPE - Ground Motion Prediction Equation

GMRA – Ground Motion Response Analysis

MAM - Micrometer Array Measurements

MASW - Multi-channel Analysis of Surface Waves

ME - Mississippi Embayment

NEA – North-East Arkansas

NEHRP - National Earthquake Hazards Reduction Program

NMFZ – New Madrid Fault Zone

ReMi – Refraction Microtremor Survey

SCPT – Seismic Cone Penetration Test

SPT – Standard Penetration Test

SSGMRA - Site Specific Ground Motion Response Analysis

SWVP - Shear Wave Velocity Profiles

TRC – Transportation Research Committee

PEER - Pacific Earthquake Engineering Research

PSHA - Probabilistic Seismic Hazard Analysis

UHRS - Uniform Hazard Response Spectrum

USGS – United States Geological Survey

LPI -Liquefaction Potential Index

LSN - Liquefaction Severity Number

FS - Factor of Safety

FC - Fines content

CRR - Cyclic Resistance Ratio

CSR - Cyclic Stress Ratio

EXECUTIVE SUMMARY

Northeast Arkansas (NEA) is part of the New Madrid Fault Zone (NMFZ). State highway agency, local city authorities, business owners, and general peoples in this region are in a need of a better understanding of seismic response characteristics to take precautionary measures before designing highways, bridges, and buildings to reduce losses due to the earthquake, as well as to optimize the construction costs. According to the American Association of State Highway Officials (AASHTO), estimation of liquefaction resistance and site-specific design response curves are key elements in the assessment of potential earthquake ground shaking and damage of existing and bridge sites. The Arkansas Department of Transportation (ARDOT) follows these requirements established by AASHTO, and the agency needs extensive ground motion behaviors of different earthquake-prone areas. As part of ARDOT's Transportation Research Committee (TRC) Projects 0803, 1603, and 190, researchers have conducted the geophysical investigations for 51 different construction sites in northeast Arkansas over the twelve years. However, these sites are not enough to cover the entire northeast region of the state. In particular, city areas require more detailed analysis for estimating design base shear forces for the design of the bridges and other critical structures and estimate the liquefaction hazards.

The purpose of this study is to investigate the usefulness of performing site-specific ground motion response analyses (SSGMRA) for the seismic design of transportation infrastructures such as bridges in Northeast Arkansas (NEA) as a means of reducing short-period design ground motions. The NEA lies within the Mississippi Embayment (ME), and it has a deep deposition of soft soil (ranging from 200m~1000m) overlying the bedrock. Besides, lying in the New Madrid Seismic Zone (NMSZ), this region is subjected to significant seismic hazards. In this project, results of geophysical and geotechnical investigation tests such as standard penetration test (SPT) results, cone penetration test (CPT) available in historical, current, and future construction projects have been used to conduct SSGRA and liquefaction potential analysis for different parts of northeast Arkansas. The SSGRA of five locations around the city of Jonesboro has been performed by following the AASHTO guideline. From the analysis results, it has been found the SGMRA can be beneficial over the typical AASHTO recommended spectral acceleration values. These five sites show reduced peak ground acceleration (PGA) as well as short period spectral acceleration (S_S) compared to that of AASHTO. The average reduction of the PGA values of all five sites was about 33%, indicating huge cost savings if the SSGRA results are considered in the design.

Additionally, an extensive CPT-based liquefaction analysis for 131 selected sites was conducted using the maximum design magnitude of earthquake and peak ground acceleration. The CPT data were analyzed to estimate liquefaction potential index, liquefaction severity numbers, seismic induced settlements, later displacements, the overall probability of liquefaction, and factor of safety against liquefaction. The findings of the current study are presented in tabular and graphical formats. The generated Geographical Information System (GIS)-based maps show the level (severity) of liquefaction risks in NEA. The most critical and comparatively safe zones in this zone are also shown in the generated maps. The findings will help engineers (bridge, highway, foundation, and other infrastructures such as utilities and pipelines) to select locations and alignments cost-effectively.

1. INTRODUCTION

1.1. Problem Statement

In the early 1800s, a series of powerful earthquakes in the magnitude of around 7.5 rattled the people and establishments in northeast Arkansas. These earthquakes originated in the New Madrid fault zone (NMFZ), which has extended from Cairo, Illinois to Marked Tree, Arkansas (Figure 1). This active fault system has high possibilities of generating major earthquakes that can strike the region again. At present, the Arkansas Department of Transportation (ARDOT) does not have any geotechnical design manuals and/or specifications for field practitioners for seismic design of different structures. Again, because of the proximity of the New Madrid Fault Zone, the Northeast Arkansas (NEA) region of the state is vulnerable to seismic hazards, which warrant the ARDOT and other agencies some detailed geotechnical design guidelines (1-4). The estimation of shear velocities and development of generalized delineated design response spectrum and analysis of liquefaction potential due to possible earthquakes before the structural design and rehabilitation of any structure are key elements for their seismic resistance. Based on the provisions set up by the Federal Emergency Management Authority (FEMA) and the AASHTO, the ARDOT requires site specific ground motion response analysis (SSGMRA) for most of the bridge projects in northeast Arkansas. Thus, almost all parts of ARDOT Districts 01, 02, 10, and 07 require the SSGMRA (Figure 1), which is very tedious and highly complex. Thus, rigorous area-specific analysis and seismic hazard maps become handy and useful to avoid expensive SSGMRAs.
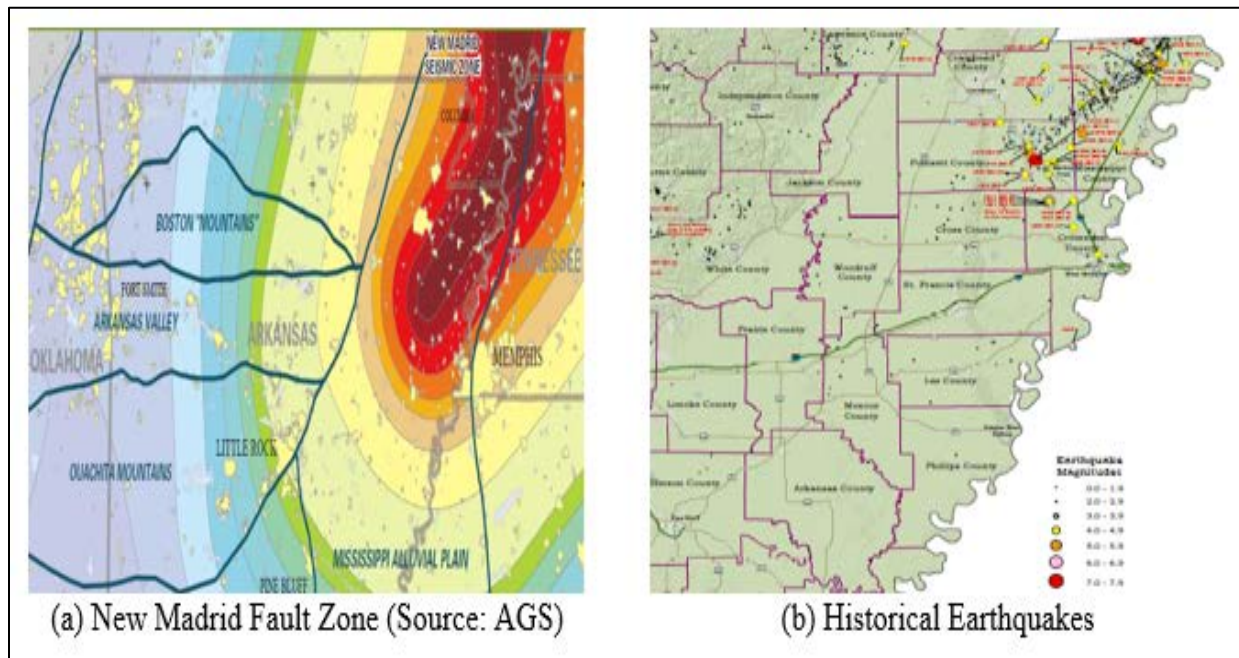
Figure 1. (a) New Madrid Fault Zone and (b) Historical Earthquakes in NEA Zone *(5).*

1.2. Importance of the study

According to the studies of the Federal Emergency Management Agency (FEMA), 34 counties in east Arkansas have been identified as vulnerable in the case of severe earthquakes. Out of all of these counties, Mississippi, Poinsett, Craighead**,** Cross, and Crittenden counties will be severally affected by any major earthquakes *(6, 7)*. These studies analyzed the possibility of damages to

different kinds of infrastructures, including highways, bridges, and culverts in the case of a hypothetical earthquake of 7.7 magnitude occurred near the New Madrid Fault line. The probable damage scenarios and statistics of bridges and segments, liquefaction prospects, and peak ground acceleration (earthquake intensity) related to the hypothetical earthquake are shown in Figure 2. However, according to a research of the Mid-American Earthquake Center, the probability of occurrence of an earthquake of magnitude from 7.0 to 8.0 in the New Madrid Fault Zone (NMFZ) is 7-10 % in the next 50 years, and the probability is from 25 to 40 % for a magnitude of 6.0+ earthquake *(7)*. Including a significant number of schools in the NEA region, Arkansas State University also lies within the vulnerable zone. However, no active research is currently being done other than the current study for reducing impacts or increasing precautions for this area.

In general, the geotechnical seismic design for a foundation of bridges or structures starts with an initial assessment of seismic hazard. The seismic foundation design requires ground motion data, seismic site classification, site characterization, site response analysis, and liquefaction potential analysis and effects evaluation. In the proposed research, different aspects of these steps will be assessed for the NEA area. Finally, a detailed guideline will be developed.

According to the United States Census Bureau, the population of the city of Jonesboro in Arkansas in 2017 was 78,394, which is growing at a significant rate *(8)*. In 1992, the Arkansas Office of Emergency Services estimated several scary scenarios in the event there is an earthquake of a magnitude of above 7 (Richter scale), which is very alarming for the dwellers of Jonesboro as well as surrounding areas. Several other relevant studies also drew similar conclusions. Out of 432 existing bridges in Craighead County, 46 bridges are in poor to structurally deficit conditions *(9)*. Overall, 25% of the bridges in northeast Arkansas are in poor condition *(9)*. Several new bridges are being proposed for fulfilling the demand of new highways and repairs. With the existing vulnerability due to earthquakes and higher seismic design force requirements, future investments for bridge repair and construction are critical. Setting up design criteria in a safe, efficient, and cost-effective manner is a challenge for this region. These criteria can hinder the economic development of the region as well.

Developing design criteria for earthquake resistant design for different structures requires an extensive study. It starts with extensive geophysical and geological studies. For the NEA, several agencies including the ARDOT took multiple initiatives to start developing seismic design criteria. Extensive geophysical investigations through field testing were carried out by the ARDOT and its contractors. Deep shear wave velocity profiles for 51 different sites were developed. On top of these, the ARDOT regularly undertakes geotechnical investigations for different construction jobs within the state. For instance, the Standard Penetration tests are carried out all the time, and they are also important inputs while developing criteria for earthquake resistant design. However, the analyses of all of these site-specific data have not been done by any research group. Significant efforts are still in need of setting up some efficient and cost-effective criteria. Setting up site classification, developing site-specific and region-specific delineated design spectrums for different locations and liquefaction hazard analysis are the key parts of the design criteria. In this project, site specific design spectra have been developed for five selected locations near the city of Jonesboro, and liquefaction hazards were analyzed for the NEA. The analysis results are summarized, and they will be a major part of the baseline design criteria for bridges and other critical structures in the region.
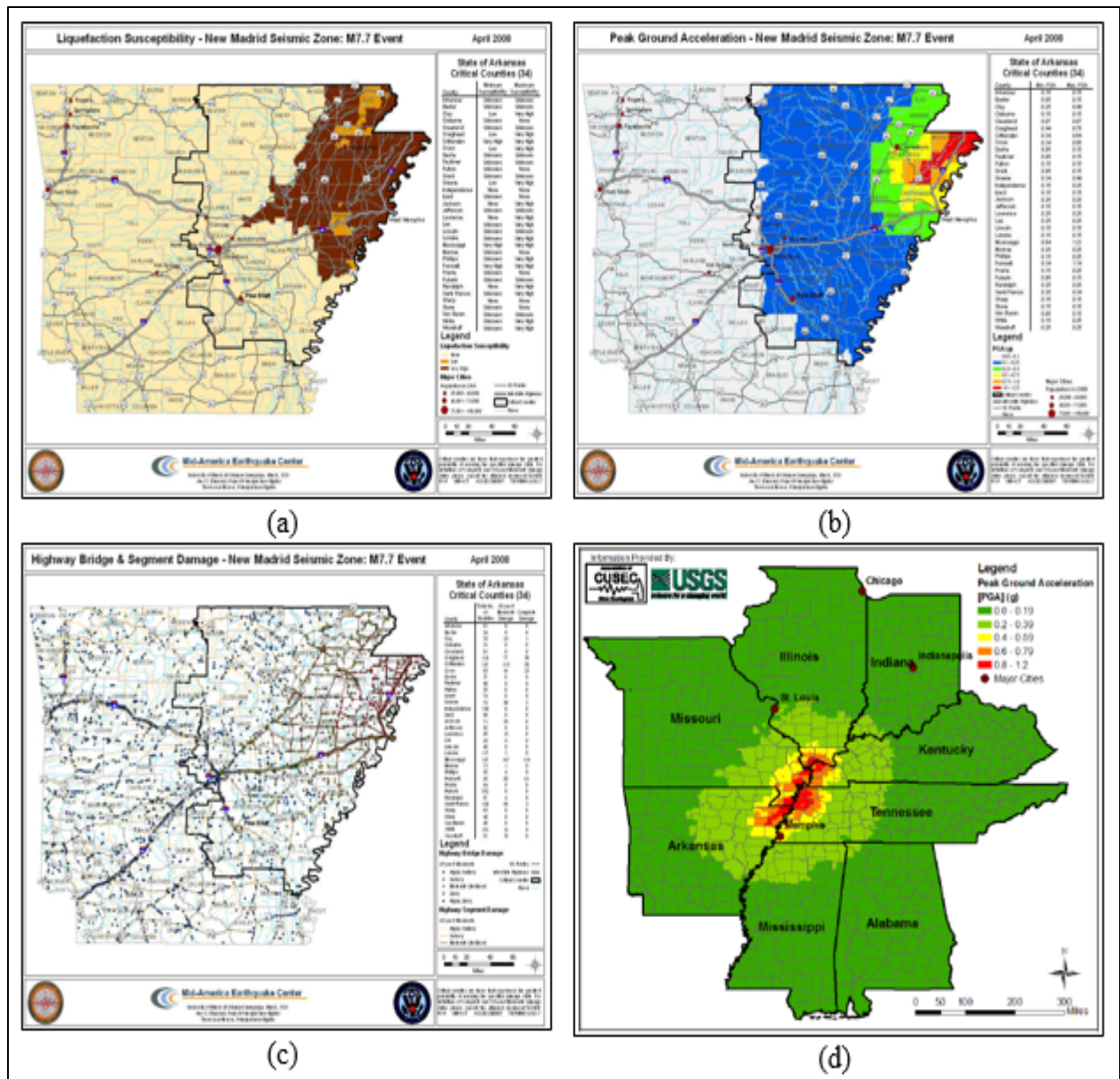
Figure 2. Probable Earthquake Damages and Impacts in NMSZ: (a) Liquefaction Hazard Area in NEA, (b) Peak Ground Acceleration in NEA, (c) Prospective Highway and Bridge Damage, and (d) Peak Ground Acceleration in NMFZ (6)(7).

2. OBJECTIVES

The primary objective of this proposed research project is to gather the SSGMRA data of northeast Arkansas reported in the recent ARDOT TRC projects *(3)(2)*. An extensive literature review has been conducted to retrieve related data and techniques available in the public domain. Besides the SSGMRA data, geotechnical properties (e.g., soil type, SPT Resistance, and CPT) of the selected and nearby locations were collected from the ARDOT sponsored previous construction and research projects. Based on available shear wave velocity profiles, SPT and CPT data, the seismic site coefficients of these sites have been estimated. The available data are then used to analyze liquefaction potentials of different and constructions sites in northeast Arkansas. Specific objectives of this project are enlisted here.

1. Gathering shear wave velocity profile data from previous projects from Arkansas and nearby states within the Mississippi Embayment (ME) area.
2. Collecting bore log data and other soil properties from test results such as standard penetration test (SPT) from ARDOT, cone penetration test (CPT) from USGS, and nearby area.
3. Estimating the variability of shear wave velocity profiles from the previously collected logs and geotechnical test results.
4. Predicting design response spectrum parameters (including seismic site coefficients) of selected locations.
5. Updating/evaluating/estimating the dynamic soil properties of the NEA region.
6. Analyzing liquefaction potentials for different locations within NEA based on different site-specific ground response analysis results.
7. Generating liquefaction hazard related maps for NEA.

The project is limited to gathering relevant geotechnical test results and information available in the public domain and analyze them using various tools and techniques to enrich the knowledge and data in predicting seismic hazards in northeast Arkansas.

3. LITERATURE REVIEW

3.1. Background

ARDOT Districts 01, 02, 07, and 10 are in the Mississippi Embayment (ME), which is a trough-shaped depression along the axis of the Mississippi River. Deposition layers of sediments are extended up to a depth of 500 to 1000 meters and mainly consist of clay, silt, sand, and gravels[10]. The ARDOT has numerous bridges within this region, and they are considered vulnerable in the case of major earthquakes. And, most of the embayment areas are susceptible to liquefaction. Multiple research groups worked on the shear wave velocity profiling, liquefaction potential analysis, and SSGMRA for different locations within the ME and inside NEA. Major previous and ongoing ARDOT sponsored research projects include TRC 0803, TRC 1603, and TRC 1901. TRC 1502, TRC 1204, MBTC 3017, MBTC 3032, and TRC 1803, are some of the other projects run by the state transportation organization. In addition to those, several other researchers also worked on the shear wave velocity profiling, liquefaction potential analysis, and site-specific ground motion analysis for different locations within ME, inside of NEA, and within NMFZ [2, 6, 7].

3.2. Site Classification Information

The Arkansas Geological Survey (AGS) has an extensive array of maps related to geohazard prospects for the state. The geohazard maps delineate liquefaction-prone areas (Figure 3a) and the National Earthquake Hazards Reduction Program (NEHRP) soil classification sites within the state (Figure 3b). NEHRP categorized most of the area of northeast Arkansas (NEA) as Site Class F type, according to AASHTO seismic design criteria, which requires site specific response analysis for geotechnical design [11, 12]. Soil sites are classified as Site Class F based on the average shear wave velocity of the upper 100 ft, average standard penetration test (SPT) blow counts, average undrained shear strength, plasticity index, moisture content of the soil, and thickness of the soil layer over bedrock. The analysis also recommends site specific investigation requirements for the area because of the expected variability in the possible amount of amplification in bedrock ground motion. According to the analysis, bedrock ground motion can amplify as high as 10X (AGS 2019) [13, 14, 15]. In another study, Romero et al (2005) [16], studied the ground motion amplification of soils in the upper Mississippi Embayment (ME). Soil deposits in ME may amplify the ground motion by factors of 2 or greater. In a recent study on the ME, Sedaghadi et al. (2018) [17] analyzed the site amplification factor can vary within a range of 3 to 7. So, SSGMRA is important for this area, in general, as well. However, site specific ground motion response analysis (SSGMRA) requires extensive efforts, time, and expertise. The total area under these criteria is also significantly large and the results obtained from the analysis are very generic. According to AASHTO (2017) [11], before designing any highway structures in this region, SSGMRA is recommended to establish the seismic design loads and estimation of design response spectra [13, 14, 15, 18]. The precise mapping of surface response acceleration, frequency, duration, and amplification parameters are helpful for a better understanding and developing appropriate guidelines.

(a) Liquefaction Susceptibility   (b) Site Classification Map
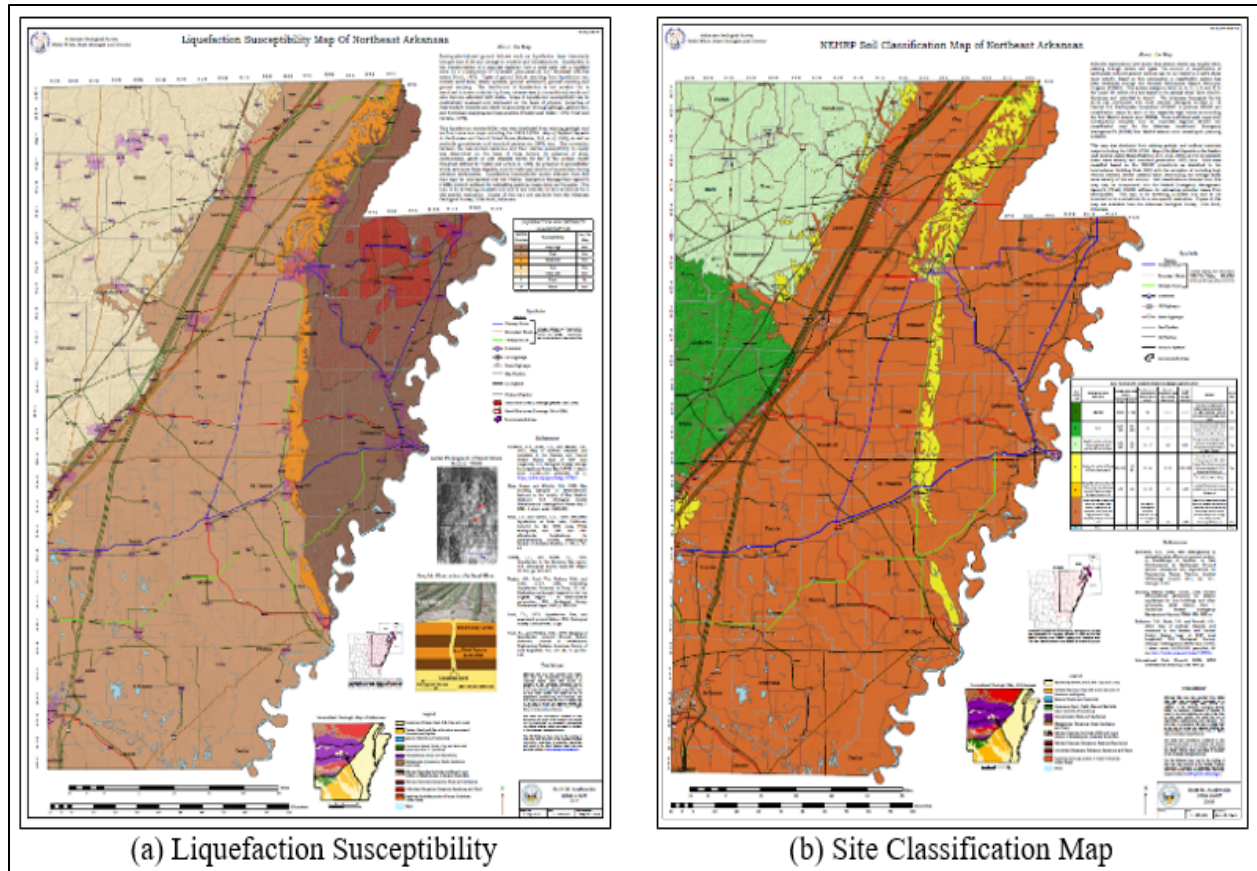
Figure 3. Geohazard Maps of NEA: (a) Liquefaction Susceptibility (19), and (b) Site Classification (15).

3.3. Previous Geophysical Investigations

Site specific analysis, ground motion amplification analysis, and liquefaction potential analysis require the shear wave velocity profiles (SWVP) of the project site. Several invasive and non-invasive methods are available for obtaining SWVPs. Downhole and cross-hole seismic surveys are the most useful methods for obtaining SWVPs, and USGS used these methods for analyzing site characteristics in NEA. Standard penetration test values (SPT-N), cone penetration test (CPT) results, seismic cone penetration test (SCPT) results can also be used for predicting SWVPs, but the variability in prediction is usually high in the case of SPT-N related methods *(20*. Several non-invasive techniques like Multi-channel Analysis of Surface Waves (MASW), ReMi, and Micrometer Array Measurements (MAM) are also common and used by several researchers and consulting companies. Usually, the data obtained using these techniques require the processing of time series spectrum data. Dispersion curves are developed using spectrum analyzers. Finally, the inversion of the dispersion curves gives SWVP. Initial data are usually collected through generating Raleigh and Surface waves from an active-source linear array surface energy source and receiving the reflected waves using geophones. A combination of two methods, e.g., MASW and ReMi altogether, help in developing better shear-wave velocity profiles. The ARDOT and other state agencies use these methods in different research projects to develop SWVPs for NEA.

Rosenbolt et al. (2010) *(21)* performed eleven low frequency, active-source surface wave velocity measurements to develop shear wave velocity profiles of deep unconsolidated soils in the Mississippi embayment. The eleven locations were selected along the top of the New Madrid Fault Line, starting from the north of New Madrid, Missouri to Memphis, Tennessee. Four of the test locations are within the Arkansas state boundary, four within the state of Missouri, and three locations fall within the state of

Tennessee. However, all eleven sites represent similar geological conditions. The researchers analyzed the velocity profiles to depths of over 200 m, and the variability of the velocity profiles for all the sites, and found the average V_S is about 193 m/s for alluvial deposits, 400 m/s for the Upper Claiborne formations, and 685 m/s in the Memphis sand formation zone. This research will be a good source of information for predicting V_S of different layers for the NEA area and selecting new locations of SSGMRA. Selected site locations are shown in Figure 5b.
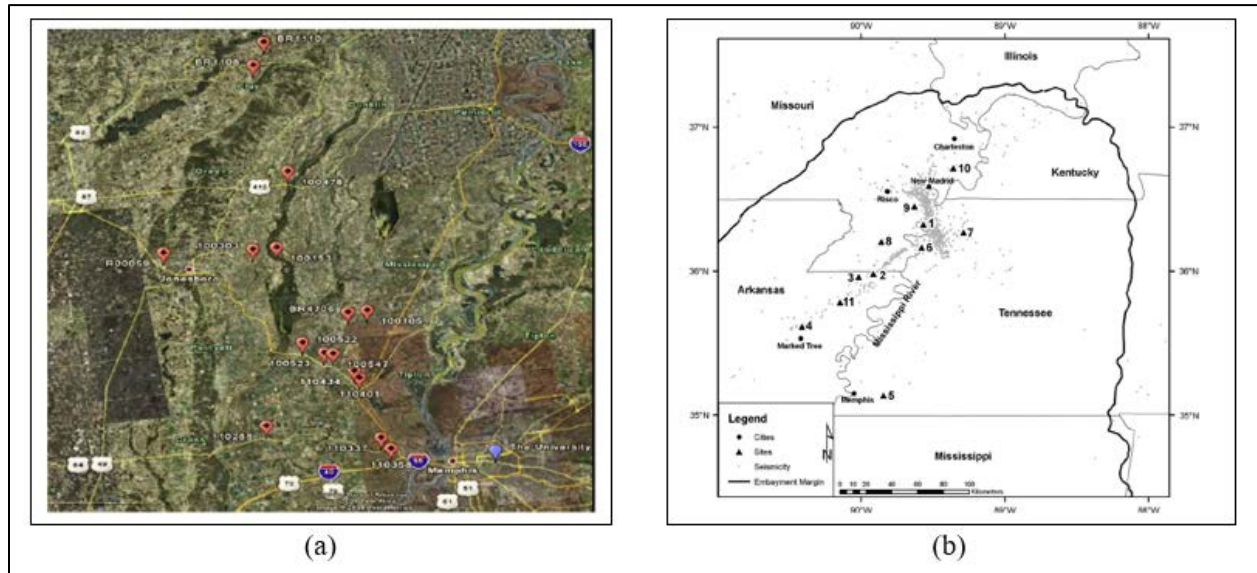
Figure 4. Test Site Locations of Two Studies: (a) TRC0803 (1); and (b) The Rosenbolt et al. (2010) (21).

MBTC3032

Mack-Blackwell Rural Transportation Center (MBTC) *(22)* at the University of Arkansas conducted SSGMRA for a site in Blytheville, AR. These researchers used equivalent-linear and non-linear site response analyses for the site. The results showed the seismic design forces could have been reduced by 33% percent, with respect to the AASHTO guideline requirements. In general, the short period ranges used for designing bridges in this region is 0.1 to 0.5 seconds. For short-period bridges in this region, these researchers expected to lower the seismic design forces, which means lowering the construction costs as well.

TRC1603

Following the cost-benefit analysis results of MBTC 3032 *(22)*, the ARDOT continued its TRC 1603 *(2)* project to develop deep shear wave velocity profiles of 15 sites within the NEA, which is close to the fault line. As part of TRC 1603 *(2)*, the SSGMRA was conducted for one bridge site in Monette, AR and it reported a 7% cost reduction for the overall structure. The shear wave velocity profiles were developed based on the Multi-channel Analysis of Surface Waves (MASW) and Micrometer Array Measurements (MAM). Data presented in the TRC 1603 *(2)*study were used in the current study.

TRC1901

Geophysical investigations were made for twenty more locations for obtaining shear wave velocity profiles under the project TRC 1901 *(3)*. This ongoing project planned to develop a decision tree, which could be used to analyze whether an SSGMRA is required for any site. The project used the existing methods adopted by the AASHTO in most of the cases. The MASW and ReMi were to be used for the SWVPs of the selected project sites. In the current study, the results obtained from TRC 1901 *(3)* have been used extensively.

CENTRAL US, MISSISSIPPI EMBAYMENT (ME) AREA

Several research works have been done on studying the seismic site factors in the ME area. In 2005, Park et al. (2005a *(23)* and 2005b *(24)*) estimated seismic site factors related to dynamic soil properties of deposits and non-linear probabilistic seismic hazard analysis for the embayment. However, these factors are required to be updated before using in NEA and Arkansas. The main objective of the proposed project is to update site coefficients for developing the design response spectrum. Romero et al. (2005) *(16)* enlisted most of the shear wave velocity profiling in the upper ME area (Figure 5). Based on the analysis of these data and the results of the ongoing TRC 1901 *(3)*project, the updating process will be more effective and useful for ARDOT. However, several steps are important in updating site response coefficients of the ground motions and attenuation relationships.
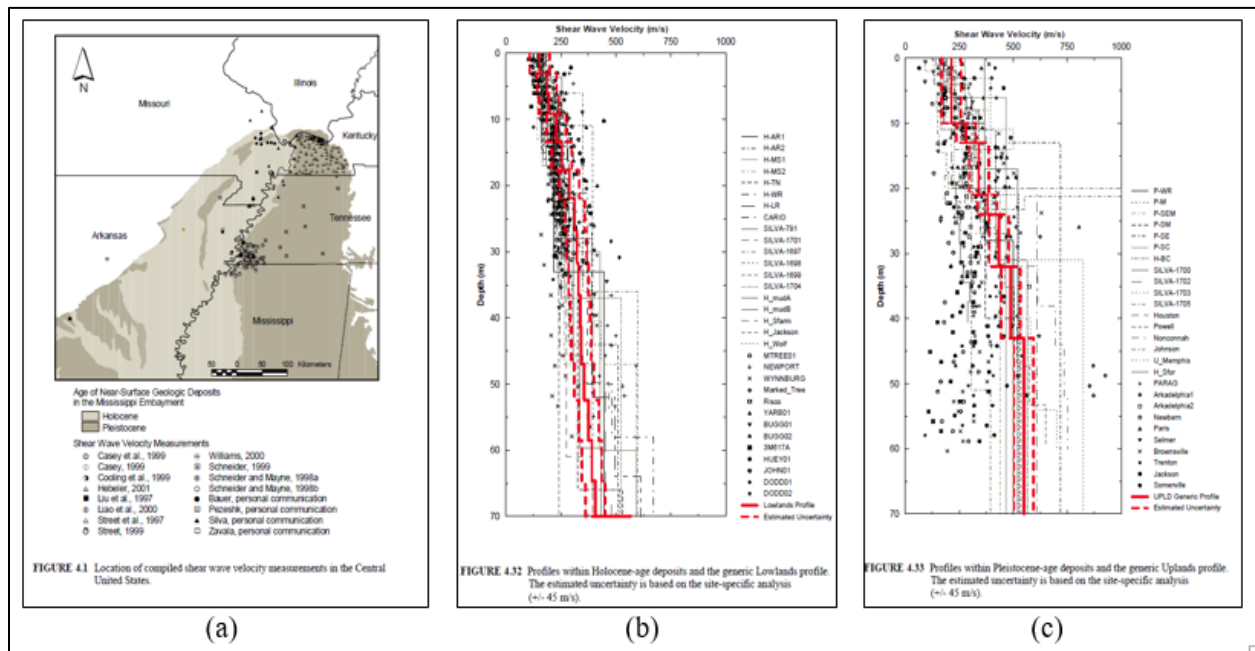
Figure 5. Shear Wave Velocity Studies in the Upper ME: (a) Site Locations, and (b-c) Variability of Shear Wave Velocity Profiles (16).

3.4. Ground Response Analysis in NMSZ and ME/NEA

According to AASHTO (2017) *(11)*, site specific ground response analysis requires probabilistic seismic hazard analysis (PSHA) for estimating the ground motions of the rocks underlying the soft layer. USGS based, Uniform Hazard Response Spectrum (UHRS) at the rock levels is a common method for seismic hazard analysis and deaggregation. While using the deaggregation method, probabilistically consistent magnitude and distance, and ground motion of the bedrock are required. However, the determination of the ground motion is always critical because of the sensitivity of the ground layers under Site Class F soils. The Pacific Earthquake Engineering Research (PEER) strong motion database is an alternate source for finding a suitable ground motion. Particularly, the NGA-East database covers the central U.S., so this database is also useful for Arkansas. Besides the AASHTO, several other guidelines are also useful for the SSGMRA *(12, 25, 26, 27)*.

Uncertainty and variability estimation in the SWVPs, site amplification factors, and variability in the dynamic properties of soils are also important for seismic investigations. Some researchers worked on the variability and uncertainty in those aspects for Arkansas and upper ME regions. The Center for Earthquake and Information (CERI) at Memphis, has done some studies on the uncertainty of NMSZ and ME subsurface profiles. Other researchers *(28)* conducted uncertainty analysis of soil profiles within the ME using blasting methods. Wood et al. (2019) *(2)* summarized the deep V_S profiles of eight seismic stations in the NMFZ area as part of TRC 1603, which is a good source to get an understanding of the variability of deep V_S profiles in the NEA area.

A group of researchers *(29, 30)* estimated dynamic soil properties and seismic site coefficients of the upper ME area. These researchers integrated nonlinear site effects in the probabilistic seismic hazard analysis. Later, Moon et al. (2017) *(31)* updated the site coefficients based on site specific ground response analysis. As part of two ARDOT projects (MBTC 3032 *(22)* and TRC 1603 *(2)*), the SSGMRAs of two bridge sites in Blytheville and Monette were accomplished.

3.5. Liquefaction Potential Analysis in ME

The ARDOT Districts 10, 01, 02, and 07 are located in the Mississippi Embayment. The embayment is a trough-shaped depression along the axis of the Mississippi River. The deposition of sediment layers is extended up to a depth of 500 meters to 1000 meters and mainly consist of clay, silt, sand, and gravels *(10)*. The ARDOT has numerous bridges in this region and they are considered vulnerable in the case of major earthquakes. And, most of the embayment area is susceptible to liquefaction.

Based on the SPT results liquefaction potentials for eight different bridge sites in NEA were evaluated by some researchers at the Mack-Blackwell Rural Transportation Center (MBTC) *(32)*. These researchers evaluated liquefaction triggering susceptibility and residual strength of liquefied soil based on SPT data provided by ARDOT. These researchers used 19 boring logs from eight different bridge sites in eastern Arkansas. The design PGAs of the bridge sites ranged from 0.24g to 1.03g. In a follow-up study *(32)*, these researchers developed general recommendations for liquefaction analysis in NEA.

As part of the ARDOT TRC 0803 project, Elsayed et al. (2010) *(1)* analyzed the liquefaction potential of different bridge locations (Figure 5a). The research team used a hybrid, non-invasive technique for determining the shear wave velocity profiles at 16 bridge sites. Surface wave tests were performed to selected sites to determine shear wave velocities, which are later used for

liquefaction potential assessment. The performed liquefaction analysis using a simplified procedure developed by Seed et al. *(33)*, Boulanger et al. (2014) *(34)*, and Youd et al. (2001) *(35)*. A factor of safety (FS) against liquefaction was calculated using the method developed by Youd et al. (2001) *(35)*, and the liquefaction potential index (LPI) was determined by using the method developed by Iwasaki (1978 and 1982) *(36, 37)*. The current research is an extension of the work accomplished by Elsayed et al. (2010) *(10)*.

4. METHODOLOGY

Several tasks were identified and followed according to the plan. The first task is reviewing the literature, background data collection is in second, and the final task is the data analysis and modeling. However, the data analysis and modeling task comprise of several subtasks. A complete flowchart of the data collection and analysis procedures is shown in Figure 6.
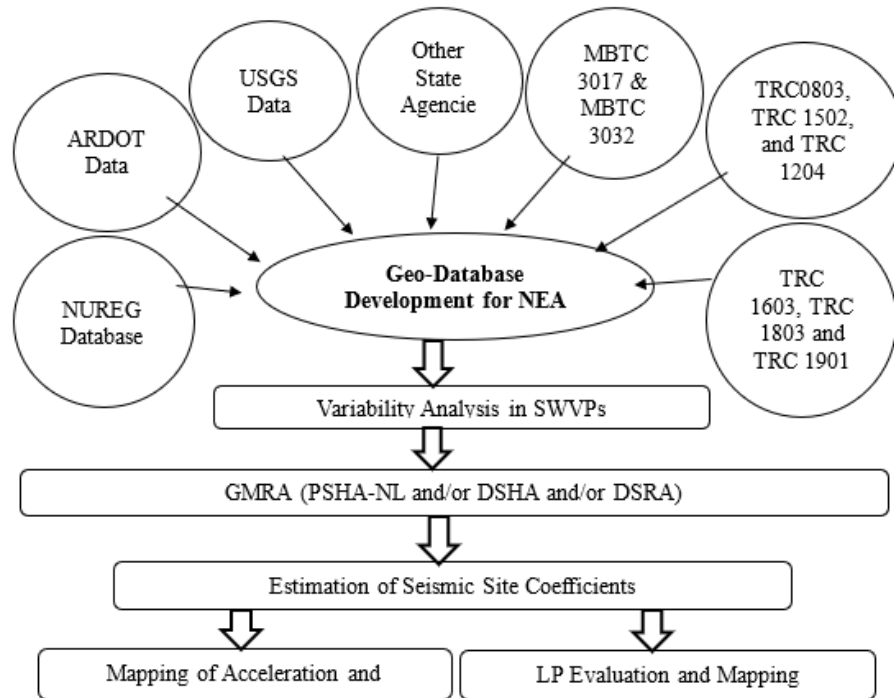
Figure 6. Data Analysis and Modeling Flow Chart.

4.1. Data Collection

The ARDOT has records of boreholes and shear wave velocity profiles of different points within the area of interest. The research team has collected the data from the bore logs and reported the analysis. The USGS is another source of data for ground motion analysis, and it has a compilation of V_{S30} within the U.S. These data have been used for the uncertainty analysis. The design reports of previous construction projects in the area are other good sources of data regarding previously used response spectrum data. The SPT, CPT, and SCPT data have been collected from the ARDOT and private organizations working in this area. Under TRC 1901 *(3)*, the SSGMRA was summarized for 51 sites in NEA; the PGA and A_s of each location were estimated. In the current research, the PGA and A_S values of bore log locations were considered as the same as those of the nearest sites (i.e., the nearest neighbor method was followed) reported in previous ARDOT projects (TRC 0803 *(1)*, TRC 1603 *(2)*, and TRC 1901 *(3)*) (Figures 7 and 8). The nearest neighbor method was deployed for the liquefaction analysis. Figure 9 shows the locations of different ARDOT sites with available geotechnical data (e.g., SPT and CPT) data besides the geophysical survey locations.
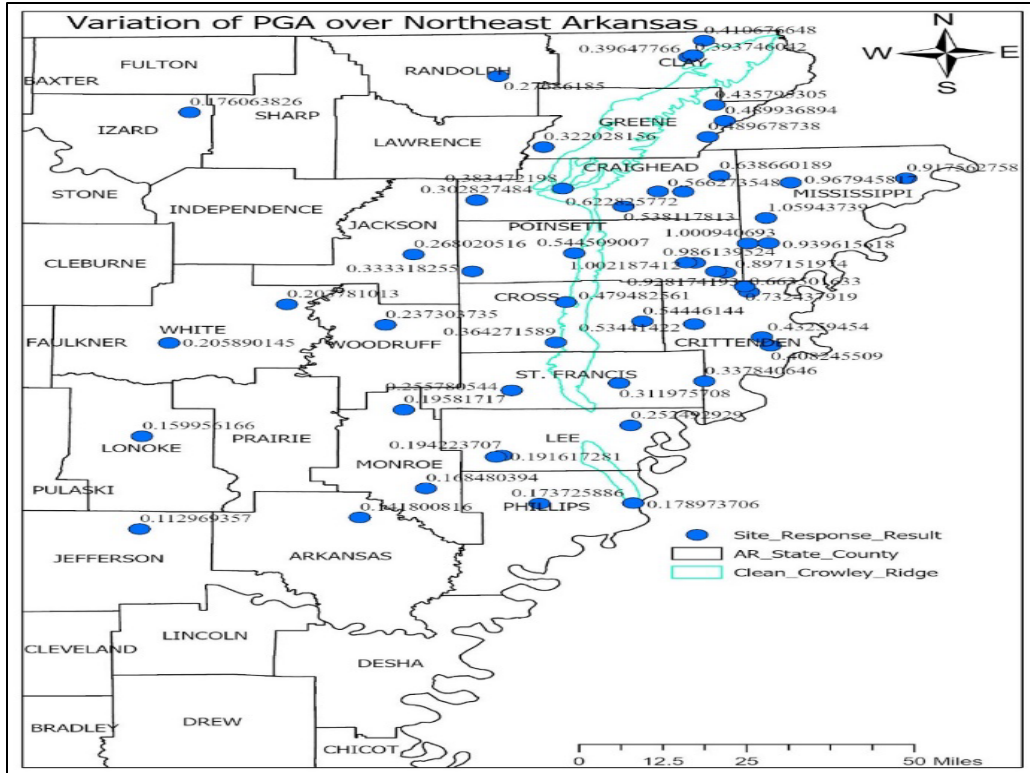
Figure 7. Variation of PGA in Northeast Arkansas.
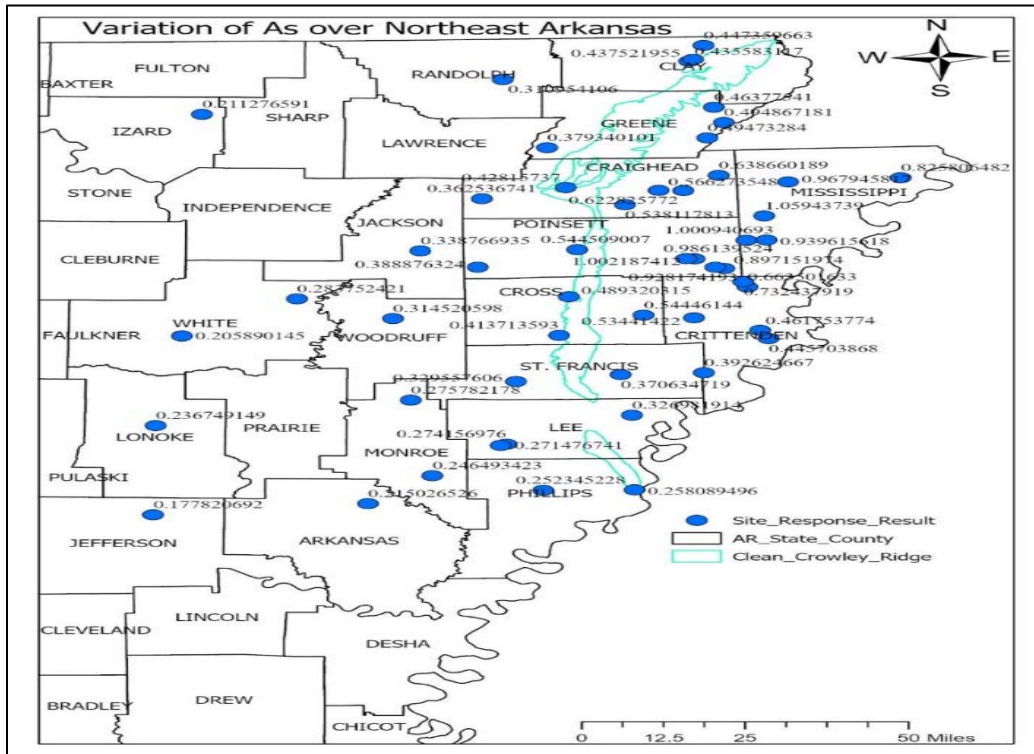
Figure 8. Variation of AS in Northeast Arkansas.
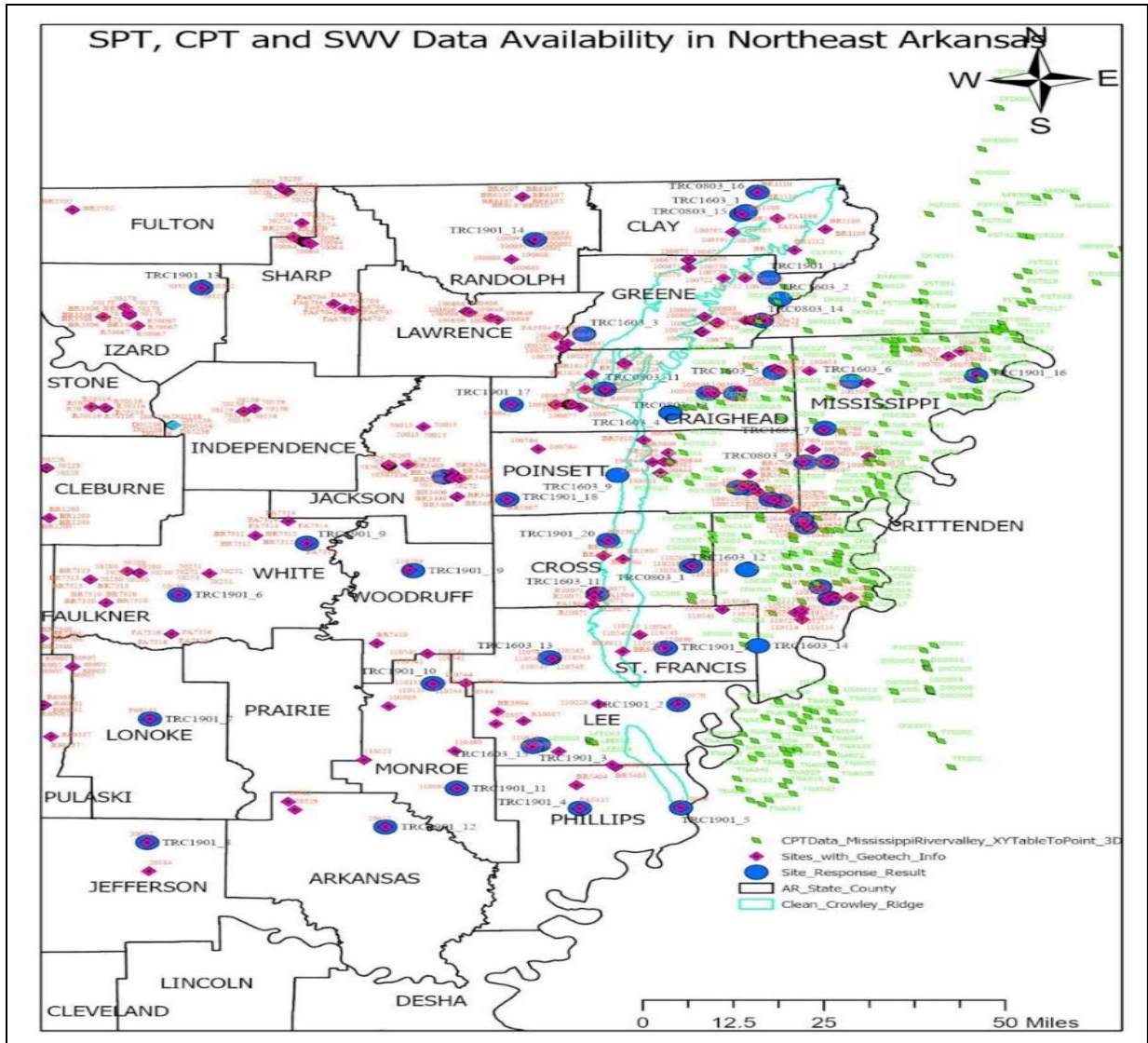
Figure 9. Locations (Availability) of Geotechnical Data in NEA.

4.2. Estimation of Site Seismic Coefficients

DEEPSOIL (Version 7.0), a unified one-dimensional equivalent linear and nonlinear site response analysis platform, has been used for the SSGMRA and simulation purposes. The input parameters for the software were extracted from the shear wave velocity profiles reported in TRC 1603 *(2)*. Staying within the scope of this study, no geotechnical test was conducted. Rather geotechnical test data such as V_S, CPT, SPT, Unit weight have been collected from the previously completed projects within Arkansas and ME. These experimental test data were not adequate to represent the overall geological condition of NEA. As a result, the proposed neural networking model was not developed to conduct the overall site classification of NEA. Rather, available geotechnical test data within Arkansas and ME were used for Site Classification by using DEEPSOIL, which accounts for the influence of large confining pressures on strain-dependent modulus degradation and damping of soil.

The Ground Motion Prediction Equation (GMPE) is required to calculate the peak ground acceleration (PGA) and other elastic response spectral acceleration of an earthquake. For this purpose, earthquake history data (accelerogram) is required. Besides, adequate numbers of ground motions could not be obtained from the Pacific Earthquake Engineering Research Center (PEER) database for this region. Since the large magnitude of ground motions (earthquake) have never been observed in the central United States and the unavailability of historical ground motion data from the PEER database, the Bayesian updating of GMPE was not possible to do in this study. As a workaround, the team collected time histories (the rocks outcrop motions) of several previously occurred major earthquakes around the world from the data repository of the Nuclear Regulatory Commission (NRC). As part of a previous project of the NRC, McGuire et al. 2001 *(38)* adjusted the earthquake acceleration time histories from various regions to circumscribe the frequency content expected from an earthquake occurring in the central United States. A similar approach has been taken in the current study. The input signals were processed using SeismoSignal (Version 2020). Later, SeismoMatch (Version 2020) was used for spectral matching of selected ground motions with the target ground motion. The target ground motion was obtained from the USGS web-based tool called Uniform hazard tool and this target motion is called as Uniform hazard Spectrum (UHS). Finally, 1-D (one-dimensional) linear and non-linear ground response analyses were conducted for five different sites surrounding the city of Jonesboro using these matched ground motions as input ground motions. The delineated design ground response spectra were developed for the five sites for their existing geological conditions with a design return period of 1000 years, as recommended by the AASHTO (i.e. Figure 18). To develop the design spectra, the method suggested by the AASHTO (2017) *(11)* has been followed.

The Equivalent Linear (EQL) and Non-Linear (NL) methods were followed to develop median ground surface spectral acceleration (Sa) for Harrisburg (Site 01), Manila (Site 02), Fontaine (Site 03), Bay (Site 04), and Marked Tree (Site 05). The locations of these five ARDOT bridge sites are shown in Figure 10 (blue circle). The median Vs profiles of these sites were collected from TRC 1603 *(2)*. The upper 30m median V_{S30} values for these sites are given in Table 1. According to the AASHTO Table C3.10.3.1-1 *(11)*, these sites can be considered as Site Class D since the shear wave velocity is greater than 600 ft/s (182.93 m/s) but less than 1200 ft/s (365.86m/s). The guidelines provided in Section 3.4.3.2 of the AASHTO Guide Specifications for LRFD Seismic Bridge Design (2nd Edition) (AASHTO, 2011 *(39)*)) were followed to perform the SSGMRA.
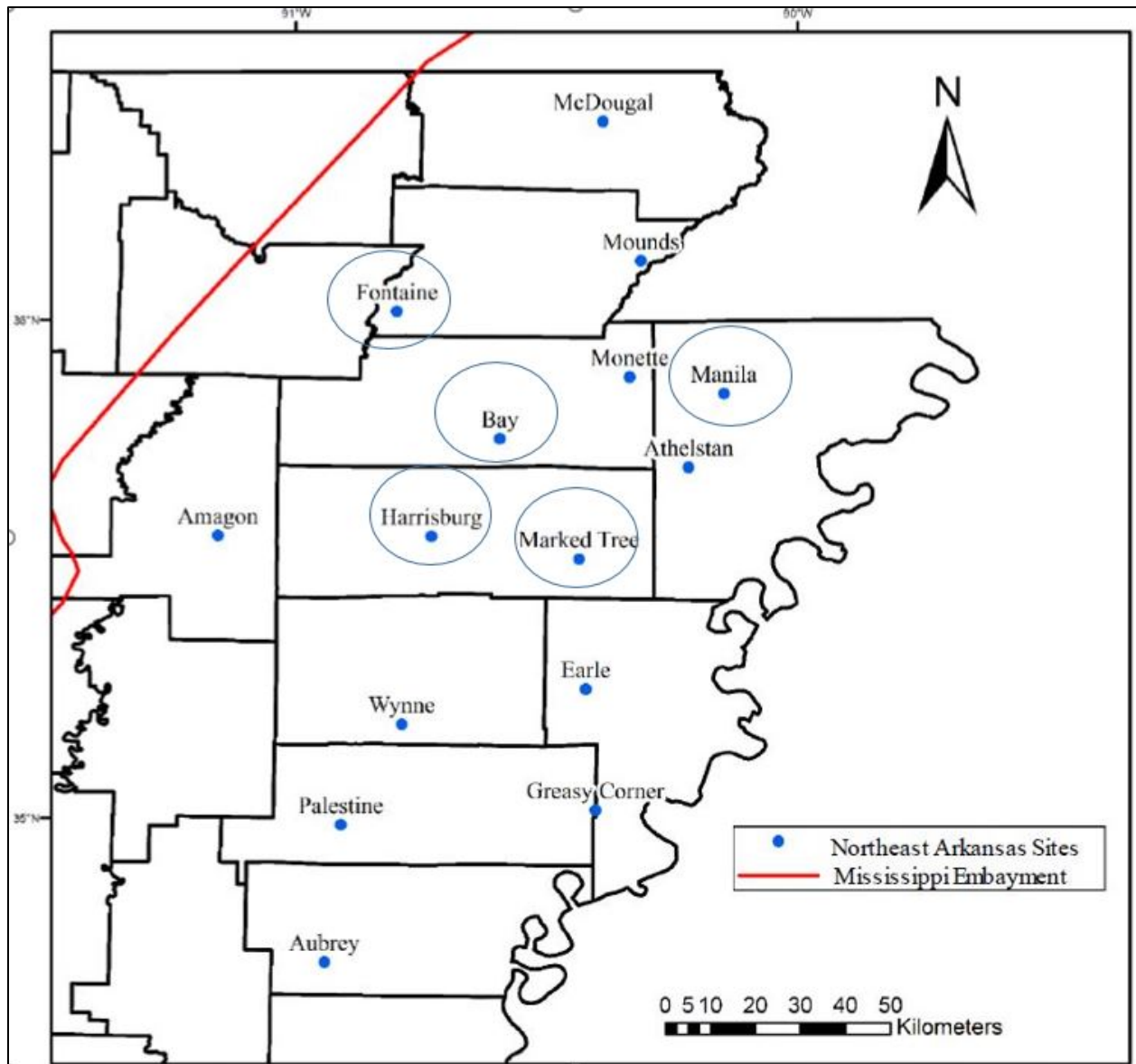
Figure 10. Dynamic site characterization testing locations in Northeast Arkansas (2).

Table 1. Upper 30m Median Shear Wave Velocity VS30 Values.

Site Name	Latitude	Longitude	V_{S30} (ft/s) [m/s]
Harrisburg	35.565781	-90.730197	816.72 [249]
Manila	35.852500	-90.147089	665.84 [203]
Fontaine	36.017175	-90.799475	770.80 [235]
Bay	35.761622	-90.594256	692.08 [211]
Marked Tree	35.520050	-90.435811	698.64 [213]

Uniform Hazard Spectrum (UHS) and Deaggregation

The USGS Unified Hazard tool was used to perform deaggregation for the conterminous U.S. 2014 (v.4.2.0). From deaggregation, the mean magnitude of an earthquake and mean source (earthquake) to site distance for these sites were determined. The summary of deaggregation is been given in Table 2. The uniform hazard spectrum (UHS) was chosen for Site Class A (2000 m/s). The return period was 1000 years, or the probability of exceedance was about 5% in 50 years. This UHS was chosen as the design target spectrum, and it provided the ground motion (g) for the spectral period ranging from 0.01 to 2 seconds. If conservative estimations of seismic site response are acceptable according to the National Earthquake Hazards Reduction Program (NEHRP) *(40)*, then it can be concluded that the selected UHS is an appropriate target spectrum. The UHS for each of the five sites is shown in Figure 11.

Table 2. Summary of Deaggregation Using USGS Unified Hazard Tool.

Site name	Latitude	Longitude	Mean Magnitude of Earthquake, M_W	Mean Source (Earthquake) to Site Distance, r (km)
Harrisburg	35.565781	-90.730197	7.50	45.13
Manila	35.852500	-90.147089	7.50	18.68
Fontaine	36.017175	-90.799475	7.44	84.01
Bay	35.761622	-90.594256	7.49	47.91
Marked Tree	35.520050	-90.435811	7.50	16.79
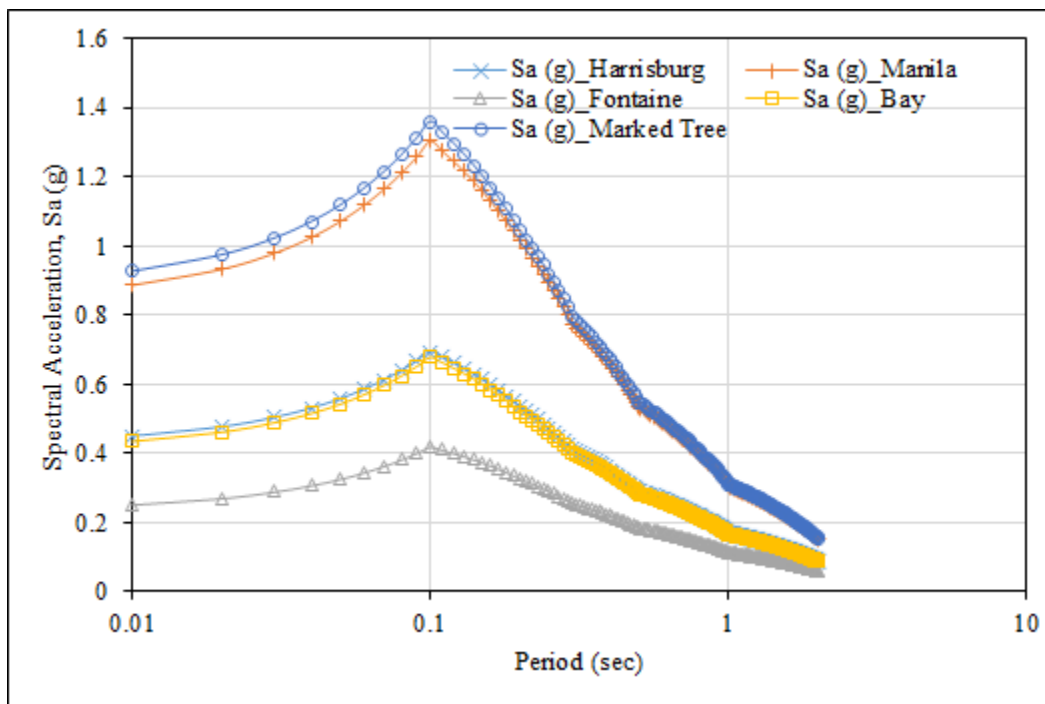
Figure 11. UHS target Spectrum for Selected Sites (Harrisburg, Manila, Fontaine, Bay, and Marked Tree).

Selection of Ground Motions

Twelve ground motions were selected from the US Nuclear Regulatory Commission Regulation (NUREG) database *(38)*. The summary of the selected ground motions is provided in Table 3. For selecting the ground motions, quantitative thumbs rules were followed *(22)*. These thumbs rules are:

- An earthquake magnitude varies from (M_W – 1) to (M_W + 1).
- The range of source to site distance is from 0.5*r to 2.0*r.
- The range of selecting PGA is from 0.5*PGA to 2*PGA (0.33*PGA to 3*PGA is acceptable, if necessary).
- Select as many different earthquake locations as possible.
- The duration of an earthquake is greater than 10 seconds.

Here, r= Mean source to site distance (from deaggregation), M_W= Mean magnitude of earthquakes (from deaggregation), and PGA= Peak Ground Acceleration (0 sec) of the selected UHS.

Table 3. Summary of the Selected Ground Motions.

Site Name	File Name	Earthquake	PGA, g	Magnitude, Mw	Source to Site Distance, km	Duration, s
Harrisburg	SOD285	San Fernando	0.228	6.6	58.1	9.1
	SHL000	Cape Mendocino	0.648	7.1	33.8	14.4
	SOR-UP	Northridge	0.263	6.7	54.1	11.5
	RAN000	Northridge	0.240	6.7	55.2	11.9
	IZN180	Kocaeli, Turkey	0.239	7.4	29.7	14.5
	MDR000	Duzce, Turkey	0.201	7.1	34.6	15.5
	ABY000	Landers	0.370	7.3	69.2	18.0
	TCU047-V	Chi-Chi, Taiwan	0.566	7.6	33.0	16.3
	TCU015-N	Chi-Chi, Taiwan	0.218	7.6	47.3	22.4
	FER-T1	Tabas, Iran	0.358	7.4	94.4	23.2
	ILA063-W	Chi-Chi, Taiwan	0.226	7.6	71.6	15.9
	TCU095-W	Chi-Chi, Taiwan	0.782	7.6	43.4	14.0
Manila	BRN090	Loma Prieta	1.236	6.9	10.3	8.6
	TCU128-N	Chi-Chi, Taiwan	0.305	7.6	9.7	29.9
	IZT180	Kocaeli, Turkey	0.401	7.4	7.7	10.1
	DAY-TR	Tabas, Iran	0.947	7.4	17.0	9.7
	SHL000	Cape Mendocino	0.648	7.1	33.8	14.4
	ARC000	Kocaeli, Turkey	0.298	7.4	17.0	10.4
	TCU046-W	Chi-Chi, Taiwan	0.336	7.6	14.3	18.8
	TCU047-N	Chi-Chi, Taiwan	1.168	7.6	33.0	10.8
	TCU095-W	Chi-Chi, Taiwan	0.782	7.6	43.4	14.0
	TCU136-N	Chi-Chi, Taiwan	0.302	7.3	8.9	22.9
	FSD172	San Fernando	0.344	6.6	27.5	7.3
	H-SUP135	Imperial Valley	0.533	6.5	26.0	5.7
Fontaine	BRN090	Loma Prieta	1.236	6.9	10.3	8.6

Site Name	File Name	Earthquake	PGA, g	Magnitude, Mw	Source to Site Distance, km	Duration, s
	TCU128-N	Chi-Chi, Taiwan	0.305	7.6	9.7	29.9
	IZT180	Kocaeli, Turkey	0.401	7.4	7.7	10.1
	DAY-TR	Tabas, Iran	0.947	7.4	17.0	9.7
	SHL000	Cape Mendocino	0.648	7.1	33.8	14.4
	ARC000	Kocaeli, Turkey	0.298	7.4	17.0	10.4
	TCU046-W	Chi-Chi, Taiwan	0.336	7.6	14.3	18.8
	TCU047-N	Chi-Chi, Taiwan	1.168	7.6	33.0	10.8
	TCU095-W	Chi-Chi, Taiwan	0.782	7.6	43.4	14.0
	TCU136-N	Chi-Chi, Taiwan	0.302	7.3	8.9	22.9
	FSD172	San Fernando	0.344	6.6	27.5	7.3
	BOL090	Duzce, Turkey	1.765	7.1	16.0	6.4
	SHL000	Cape Mendocino	0.648	7.1	33.8	14.4
	MDR000	Kocaeli, Turkey	0.201	7.4	34.6	15.5
	SOD285	San Fernando	0.228	6.6	58.1	9.1
	RAN000	Northridge	0.240	6.7	55.2	11.9
	SOR-UP	Northridge	0.263	6.7	54.1	11.5
Bay	29P000	Landers	0.207	7.3	42.2	20.4
	IZN180	Kocaeli, Turkey	0.239	7.4	29.7	14.5
	HWA056-N	Chi-Chi, Taiwan	0.203	7.6	48.7	12.3
	TCU015-N	Chi-Chi, Taiwan	0.218	7.6	47.3	22.4
	ABY090	Landers	0.434	7.3	69.2	19.6
	FER-T1	Tabas, Iran	0.358	7.4	94.4	23.2
	TCU047-W	Chi-Chi, Taiwan	0.700	7.6	33.0	12.9
	BRN090	Loma Prieta	1.236	6.9	10.3	8.6
	TCU128-N	Chi-Chi, Taiwan	0.305	7.6	9.7	29.9
	IZT180	Kocaeli, Turkey	0.401	7.4	7.7	10.1
	DAY-TR	Tabas, Iran	0.947	7.4	17.0	9.7
	SHL000	Cape Mendocino	0.648	7.1	33.8	14.4
Marked Tree	ARC000	Kocaeli, Turkey	0.298	7.4	17.0	10.4
	TCU046-W	Chi-Chi, Taiwan	0.336	7.6	14.3	18.8
	TCU047-N	Chi-Chi, Taiwan	1.168	7.6	33.0	10.8
	TCU095-W	Chi-Chi, Taiwan	0.782	7.6	43.4	14.0
	TCU136-N	Chi-Chi, Taiwan	0.302	7.3	8.9	22.9
	FSD172	San Fernando	0.344	6.6	27.5	7.3
	BOL090	Duzce, Turkey	1.765	7.1	16.0	6.4

Scaling of Selected Ground Motions

The initially selected time histories often differ from the design motions in terms of shaking peak amplitude and response spectral ordinates. Due to these reasons, modification of time histories is required to be used in the analysis. Two modification methods are commonly used in practice: (1) simple scaling approach, and (2) spectrum matching approach. In this study, the spectrum matching approach has been followed.

The twelve selected ground motions from the NUREG database *(38)* were scaled using the SeismoSignal 2020 tool, which is an earthquake software for signal processing of time histories (acceleration vs period, velocity vs period, and displacement vs period) data. The baseline correction and filtering correction (not applied for all time histories) were used to process the obtained time histories. These corrections were applied so that original time histories get much disturbed, stay (oscillating back and forth) around the zero-line, and end at the zero-line (i.e., no drift in general). It should be noted that the baseline correction has less impact on processing time histories. For the baseline correction, the linear type base correction was applied. For the filtering correction, the filter type was Butterworth, and the filter configuration was Bandpass.

Spectral matching of Scaled Ground Motions

The spectrum matching approach has been used since a response spectrum developed as a part of site response analysis is replacing the code-based response spectrum approach while performing the structural design. SeismoMatch 2020 was used to match the scaled ground motions in terms of response spectra. It is an earthquake software for adjusting the earthquake accelerogram to match a specific target response spectrum. The matching algorithm proposed by Atik and Abrahamson (2010) *(41)* was used to match the scaled ground motions with the UHS target spectrum. Ground motions were matched with the target spectrum at least from $0.5T_n \sim 2.0T_n$, as mentioned in Section 3.10.2 of the AASHTO Guide Specifications for LRFD Seismic Bridge Design *(11)*. The average tolerance limit of misfit was between 20% and 30%. A scaling factor of 1.0 was used for spectral matching. A 5% damping was considered while performing the spectral matching. These matched ground motions have been used as input ground motion in the site response analysis using DEEPSOIL 7.0. A summary of the matched ground motions with the target spectrum has been given in Figures 12 and 13.
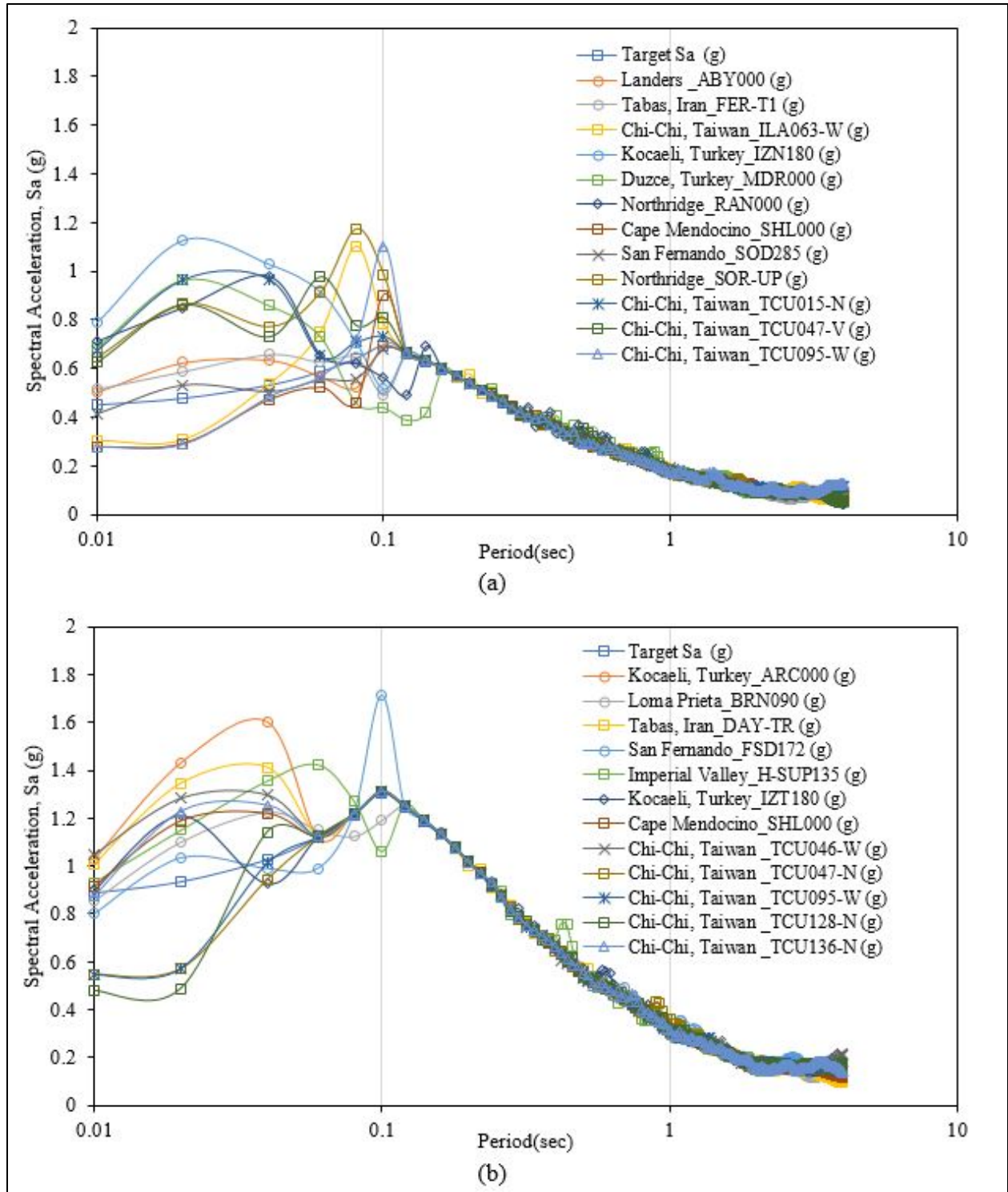
Figure 12. Summary of Matched Ground Motions with Target Spectrum for: (a) Harrisburg, and (b) Manila.
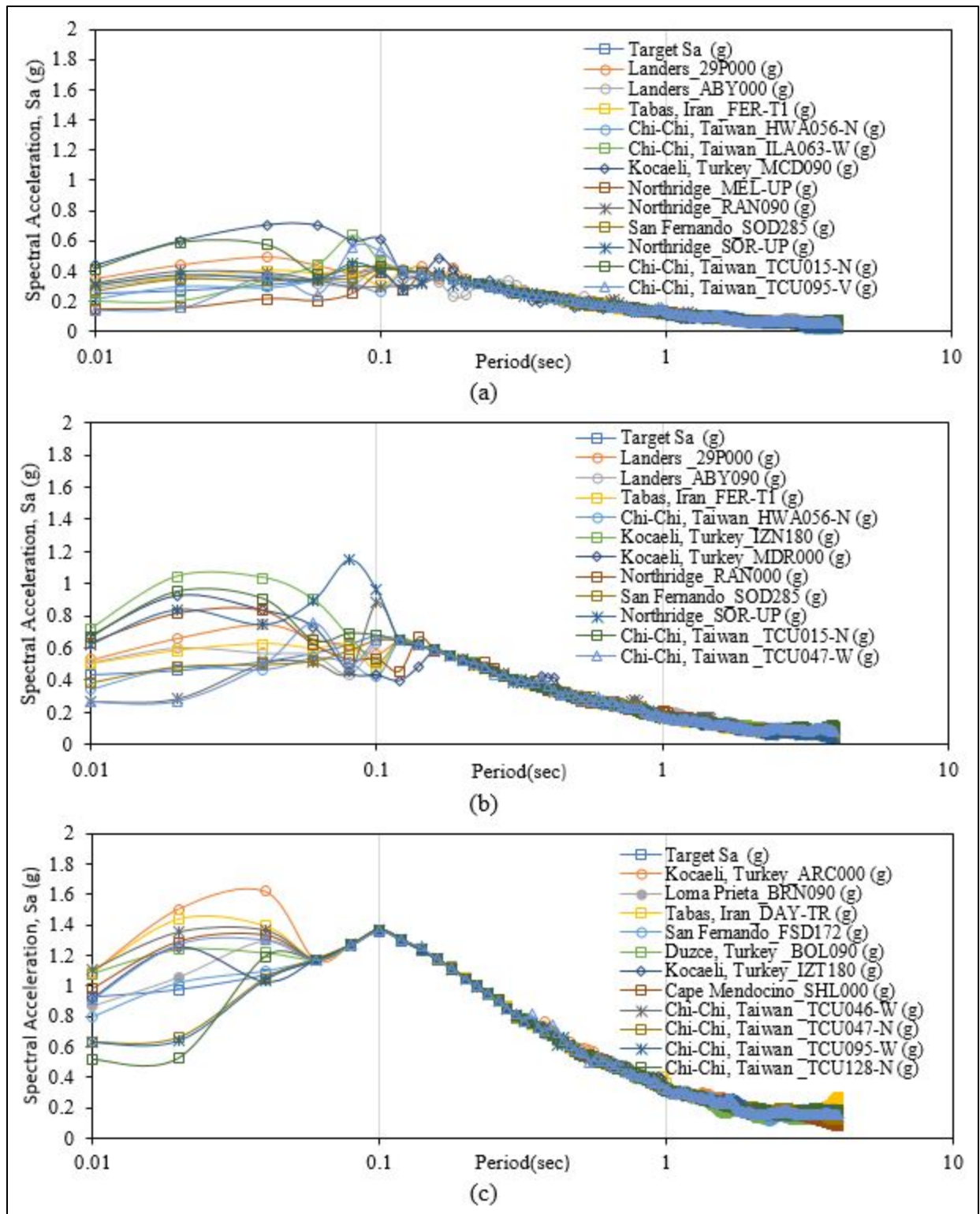
Figure 13. Summary of Matched Ground Motions with Target Spectrum for: (a) Fontaine, (b) Bay, and (c) Marked Tree.

Simulation of Wave Propagation:

DEEPSOIL 7.0 *(42)* was used to perform the seismic response analysis. DEEPSOIL 7.0, a one-dimensional site response analysis platform capable of performing both EQL and NL analyses. For performing both EQL and NL analyses, the GQ/H (General Quadratic/ Hyperbolic Model) soil model was used along with Non-Mashing Re/unloading formulation.

Darendeli (2001) *(43)* proposed reference curves for both modulus reduction and damping to develop soil's dynamic properties such as [G^*/G_{max}] and material damping [D]. In this study, the dynamic properties of soil were developed for each soil layer using empirical relationships in DEEPSOIL 7.0 *(43)*. These reference curves were fitted using GQ/H *(44)* soil model for each soil layer along with the MRDF-UIUC reduction factor fitting procedure *(45, 46)*.

The dynamic property of soil is generally influenced by mean effective confining pressure, soil plasticity index (PI), and over consolidation ratio (OCR). In this study, medium sand was considered along the soil column based upon shear wave velocity and geologic deposition of this project location *(2)*. The friction angle was considered 30°, and there was no cohesion of soil. The plasticity index (PI) was assumed to be zero, and the OCR was assumed to be 1.0 (one). Based upon the geologic deposition, the unit weight of soil for this site ranges from 1700 kg/m^3 to 2100 kg/m^3. The target shear strength of the nonlinear shear modulus reduction (G/G_{max}) is calculated using the Mohr-Coulomb equation (Eq. 1).

$$\tau_{target}=cVs+\sigma_v'*\tan(\phi)$$ [1]

Where,
σ_v' = Effective stress at the mid-depth of the interested soil layer,
ϕ = Friction angle of the soil layer, and
cV_s = Judgement-based shear strength developed at 0.1% shear strain for a linear elastic material with 80% of the maximum shear modulus derived from the shear wave velocity (Vs) of the soil layer under consideration, as shown in Eq. 2 *(42)*.

$$cVs=\rho*V_s^2*0.8*0.1\%$$ [2]

The coefficient of earth pressure at rest (K_0) was calculated as shown in Eq. 3.

$$K_0=[1-\sin(\phi)]*OCR^{\sin(\phi)}$$ [3]

For small strain viscous damping definition, the "frequency-independent damping" along with "do not update damping" matrices were used. For the frequency domain, the number iteration was considered as 15, and the effective shear strain ratio was considered as 0.65. For the time domain, a flexible step control was considered. "Linear in time domain" was selected in the time history interpolation method. In both analysis techniques (EQL and NL) in DEEPSOIL (Version 7.0), a series of surface response spectra was estimated against input ground motions.

Development of Delineated Design Spectrum

The final site-response spectra for the selected sites have been developed based upon a weighted average of the EQL (50%) and NL (50%) analyses results, as recommended by other researchers *(2, 47)*. At first, an average of the "seismic amplification ratio," the ratio of the surface repose spectrum, and the input rock response spectrum, for both EQL and NL analyses were obtained. These amplification ratios were obtained by dividing the surface response by input rock responses.

Then, these average amplification ratios were multiplied by the AASHTO Site Class A spectrum *(2)*.

The delineated design spectrum for all periods was determined as follows:

    a) AASHTO design spectrum for Site Class D was determined as per the guidelines listed in Sections 3.10.3.2 and 3.1.4 of the AASHTO LRFD Bridge Design Specifications.

    a. Multiply the AASHTO design spectrum for Site Class D by 2/3.

    b. Multiply amplification ratios (50%-50% weighted average of both EQL and NL analyses) by the AASHTO Site Class A spectrum.

    c. Plot these three spectra in a single diagram.

    d. The delineated design spectrum should be between 2/3 and 1.0 of the AASHTO design spectrum for Site Class D. That means for developing the delineated design spectrum higher limit is 1.0 of the AASHTO design spectra for Site Class D and the lower limit is 2/3 of the AASHTO design spectrum for Site Class D.

    e. The delineated spectrum obtained from the multiplication of the amplification ratio and the AASHTO Site Class A spectrum lies between 2/3 and 1.0 of the AASHTO design spectrum for Site Class D.

4.3. Cyclic Liquefaction Analysis

Boulanger and Idriss (2014) *(34)* updated their previous liquefaction analysis procedure published in 2009. These two methods are popular among practitioners in industries. In this research, the method developed by Boulanger and Idriss (2014) *(34)* has been used for cyclic liquefaction analysis. This method was followed for a total of 131 CPT sites in the current study. Step by step methodology for CPT Based Liquefaction Analysis using Boulanger and Idriss (2014) *(34)*. Basic inputs of this method is soil density, percent fine content in the soil, soil behavior type index, level of earthquake shaking, probability of liquefaction, depth of groundwater table (GWT). The cyclic stress ratio (CSR) and cyclic resistance ratio (CRR) are calculated based on the recorded CPT tip resistance, sleeve friction, pore pressure of water, percent fine content, and unit weight of soil data. Then the factor of safety is calculated from the ratio of CRR and CSR. While calculating the lateral displacements, at first, the design earthquake magnitude and peak ground acceleration for the site required were extracted from TRC 1901 *(3)*. The method suggested by Youd et al. (2001) *(35)* was followed to estimate lateral displacement indices. Then lateral displacements were estimated from the lateral displacement indices based on the sloping conditions of the soil. In this research, all the geolocations were considered as level ground with free face conditions.

In the case of estimating seismic vertical settlement, the average shear stress was calculated based on the CSR and peak ground acceleration values. Then the small shear strain modulus was calculated to estimate shear strain amplitude and volumetric strains of a design earthquake. Then, the seismic settlements were calculated using the volumetric strain for different groundwater levels. Finally, the liquefaction potential indices were calculated using the Geologismiki tool (CLiq v.3.0 - CPT liquefaction software), which is based on the method developed by Iwasaki et al. (1982) *(37)*. After evaluating cyclic liquefaction potentials, the findings were plotted using the ArcGIS Pro tool.

5. ANALYSIS AND FINDINGS

5.1. SEISMIC SITE COEFFICIENTS

Five bridge locations around the city of Jonesboro were selected for this study. Site 01 is in Harrisburg, Site 02 is in Manila, Site 03 is in Fontaine, Site 04 is in Bay, and Site 05 is in Marked Tree. The findings related to each site is discussed in the following sections.

Site 01: Harrisburg

The maximum shear strain is predicted around a depth of 400 m. The maximum shear strain for EQL analyses was found to be 0.143% and 0.138% for NL analyses. A summary of the maximum shear strain for EQL and NL analyses has been given in Figure 14.
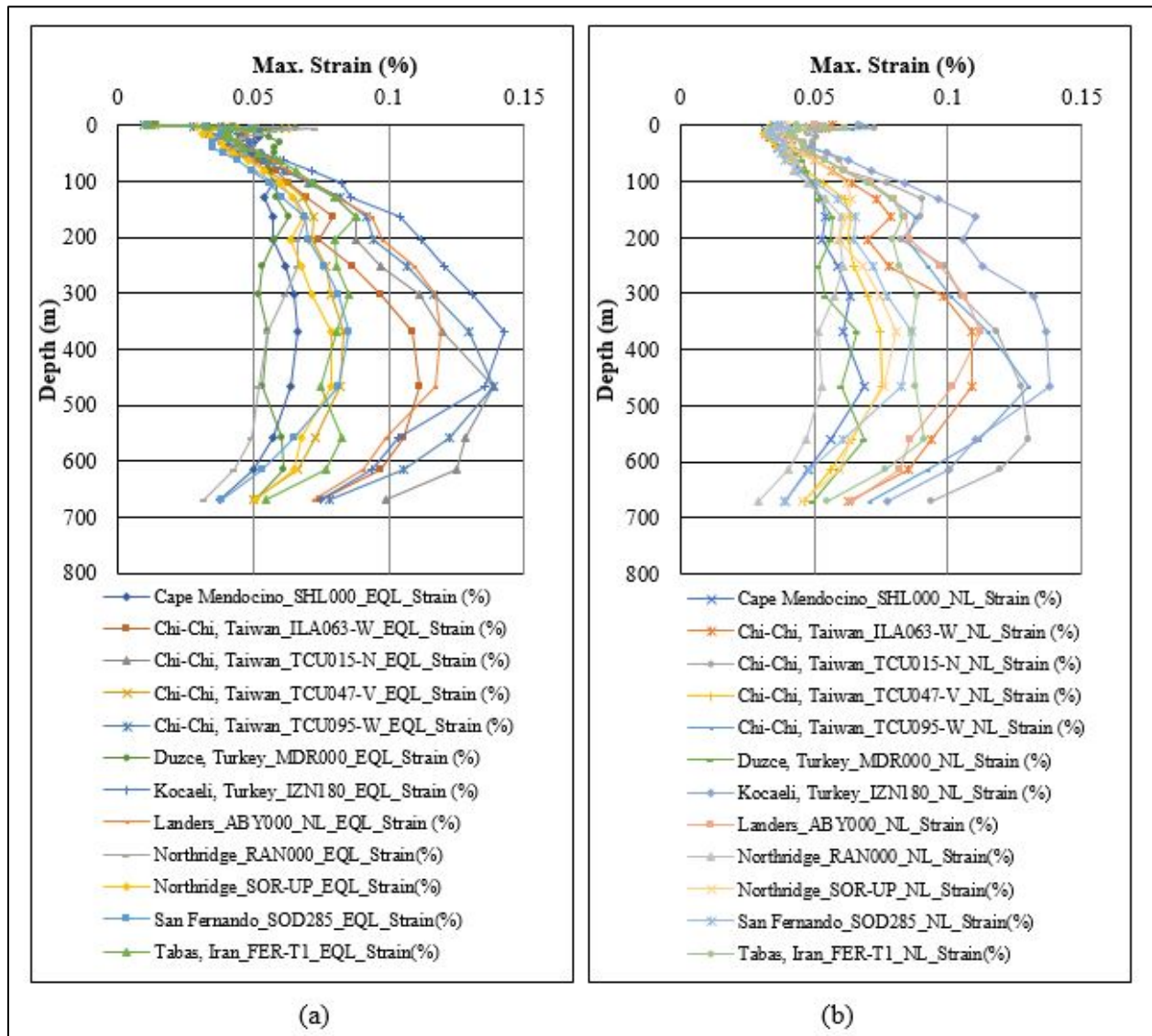
Figure 14. Summary of Maximum Shear Strain for Harrisburg: (a) EQL, and (b) NL Analyses.

Sample comparisons between the EQL and NL methods of Site 01 (Harrisburg) are shown in Figure 15. Summaries of seismic site response analyses for both EQL and NL methods have been given in Figures 16 and 17. The delineated design spectrum is shown in Figure 18.

(a)

(b)
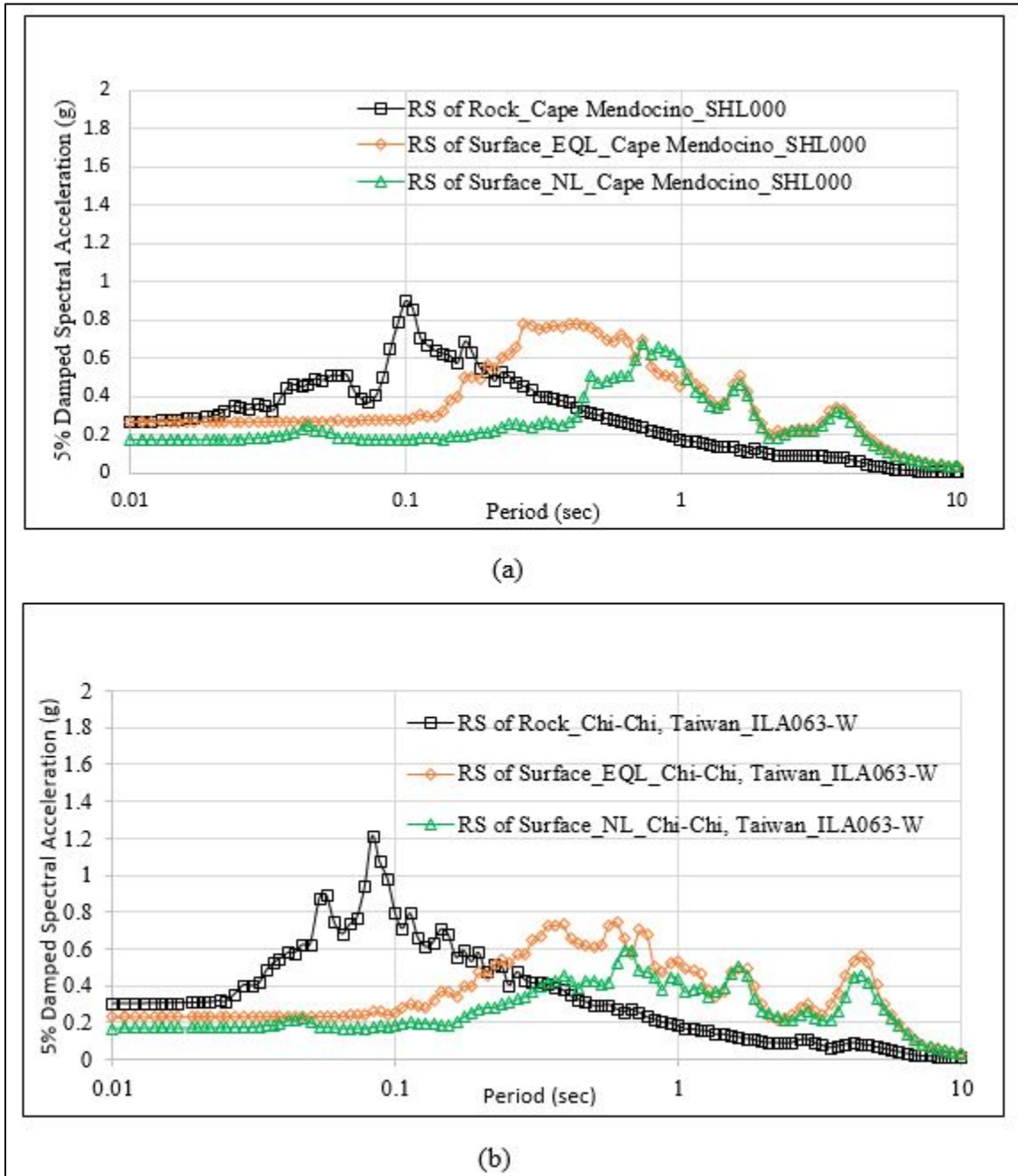
Figure 15. *Sample Comparisons between EQL and NL Methods for Harrisburg: (a) Cape Medocineo, USA, And (b) Chi, Taiwan.*
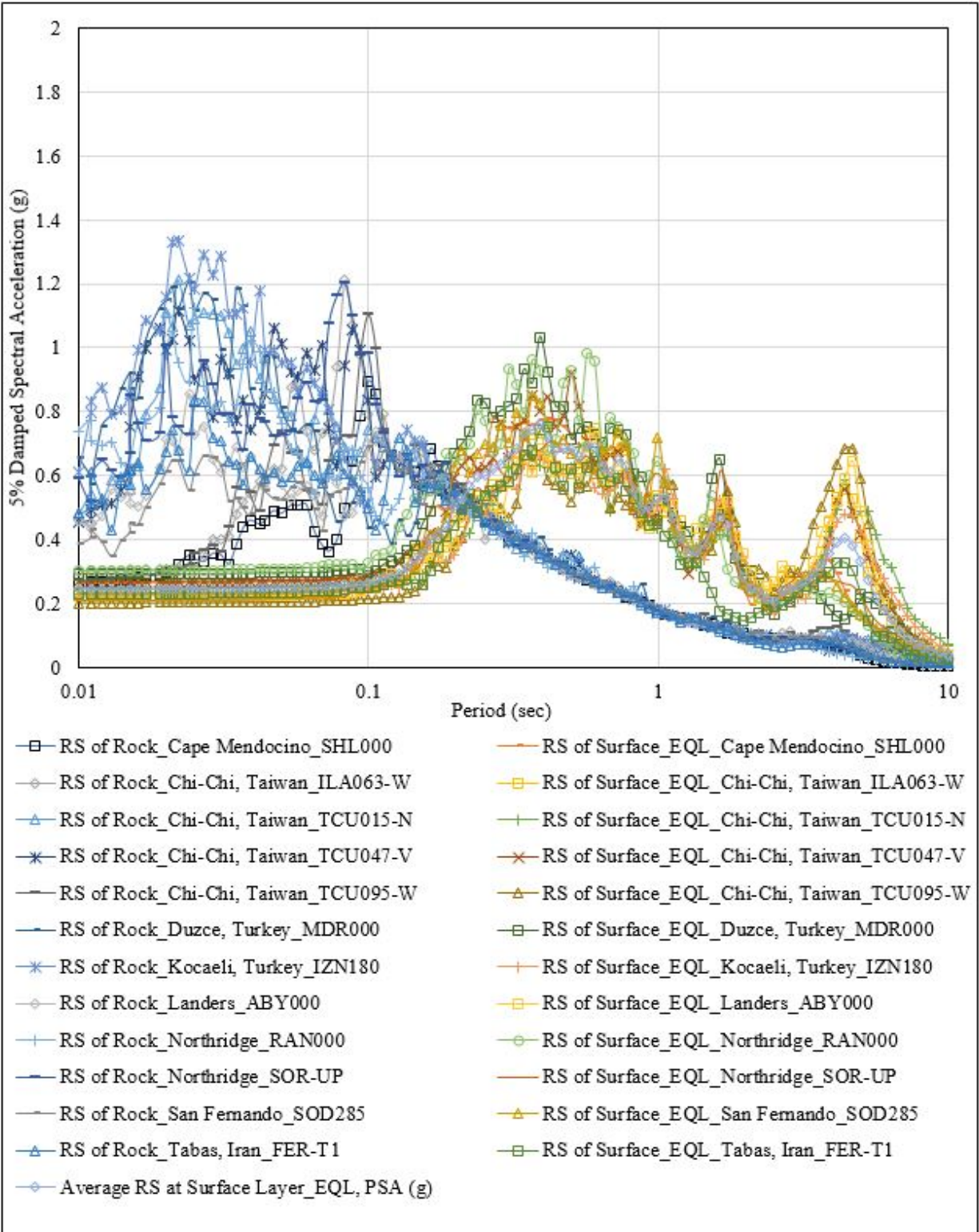
Figure 16. Summary of Response Spectrum for EQL Analyses of Harrisburg.
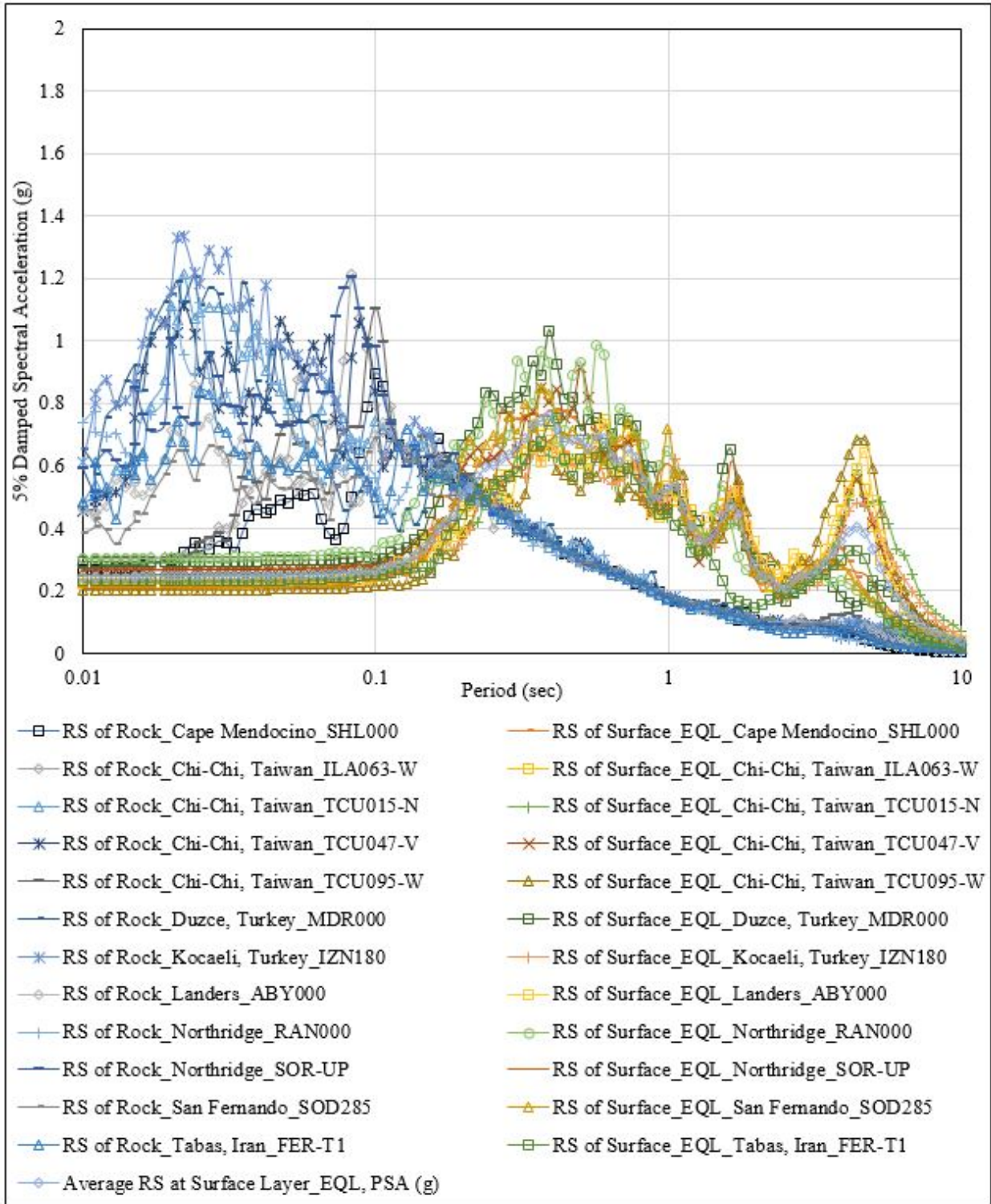
Figure 17. Summary of Response Spectrum for NL Analyses of Harrisburg.
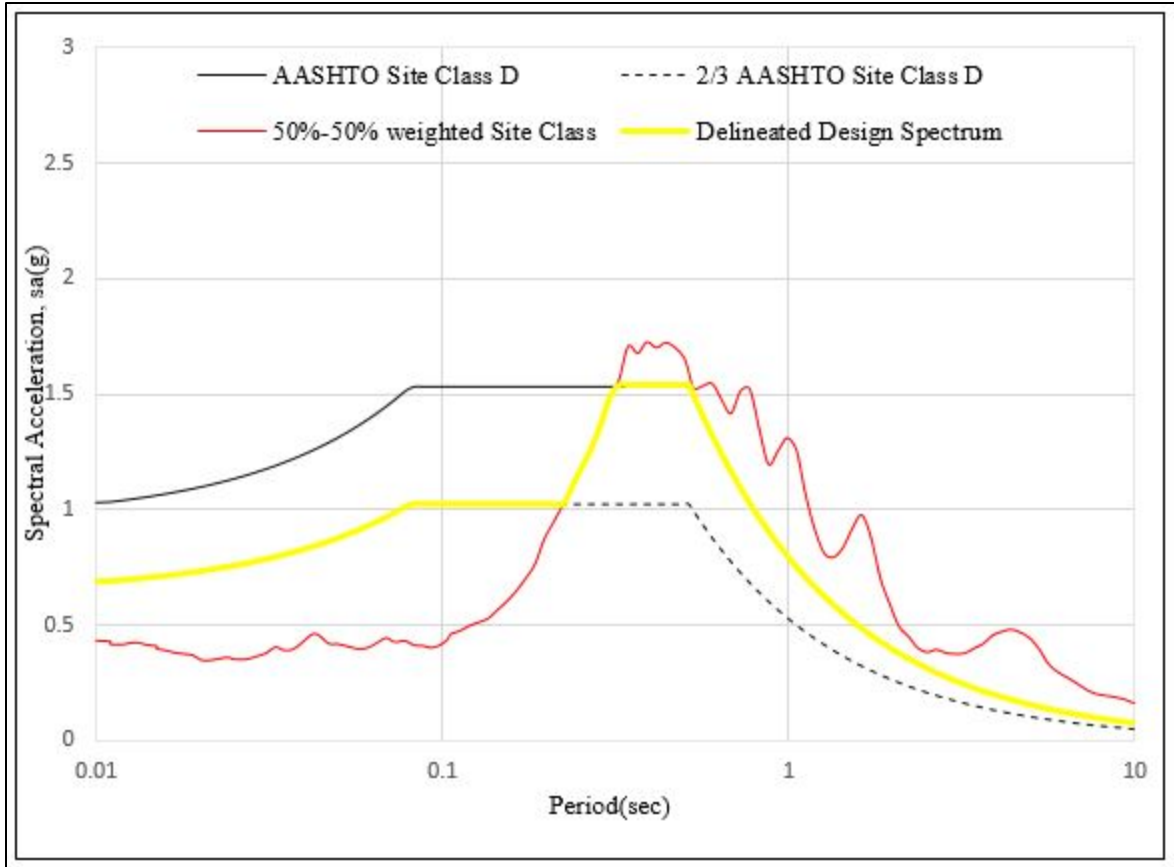
Figure 18. Delineated Design Spectrum for Harrisburg, AR.

From Figure 18, it is seen that there is a reduction of 33% from the PGA from 0.01 to approximately 0.224 seconds compared to the seismic accelerations predicted by the AASHTO general procedure. From 0.224 to approximately 0.345 seconds, the site-response spectrum remains between the 2/3 and 1.0 AASHTO general procedure response spectrum. From 0.345 to the rest of the periods, the spectrum is greater than that of AASHTO. For Harrisburg (Site 01), the maximum surface site response amplification was found to be 5~6 times higher than that of bedrock. This amplification was found mostly for long period (> 1 sec.).

Site 02: Manila

The maximum shear strain for EQL and NL analyses was found to be 0.442% and 0.472%, respectively. A summary of maximum shear strain for EQL and NL analyses is given in Figure 19. Sample comparisons between the EQL and NL methods for Site 02 (Manila) are shown in Figure 20. Summaries of seismic site response analyses for both EQL and NL methods have been given in Figures 21 and 22. The delineated design spectrum is shown in Figure 23.
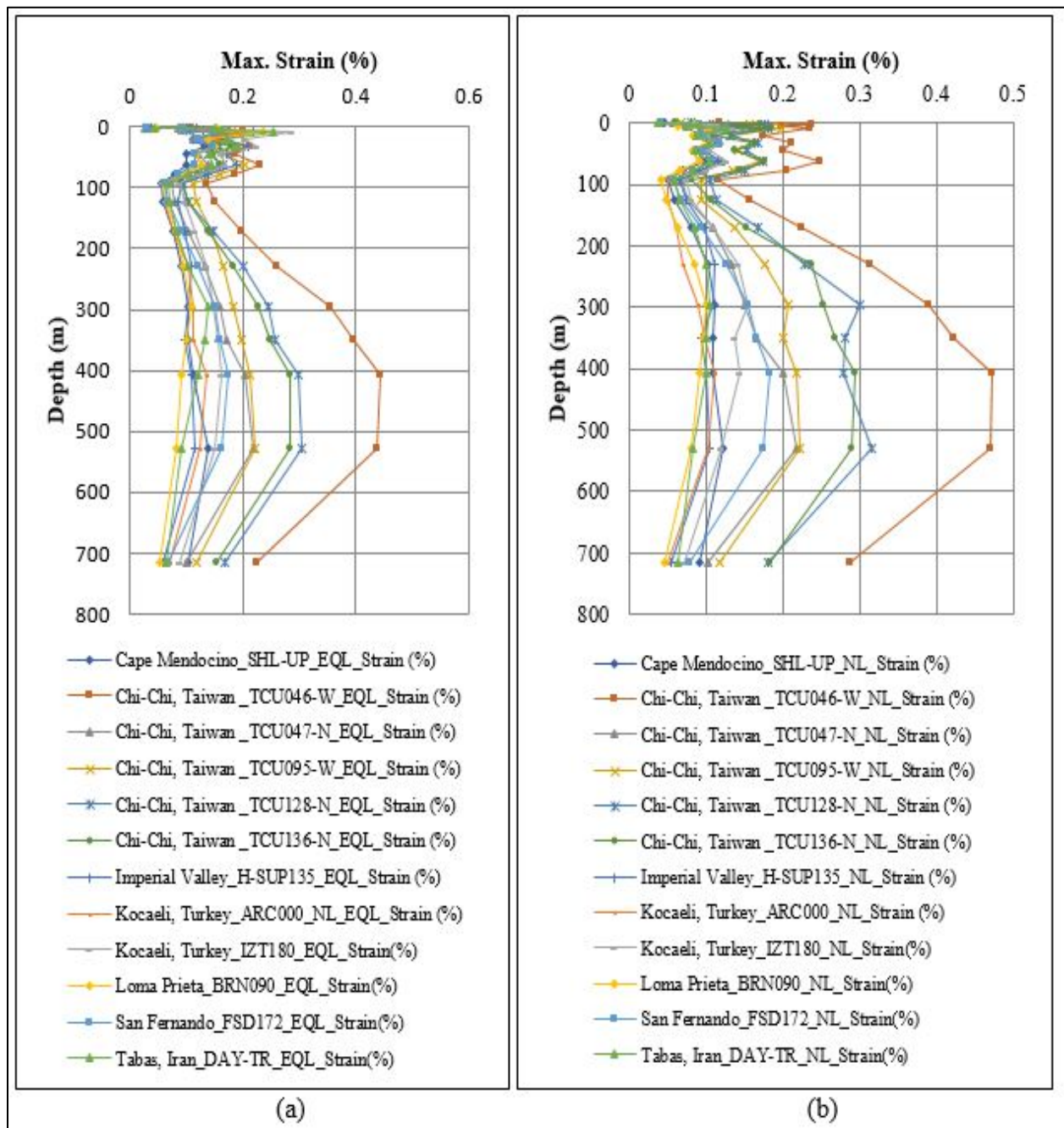
Figure 19. Summary of Maximum Shear Strain for Manila: (a) EQL, And (b) NL.
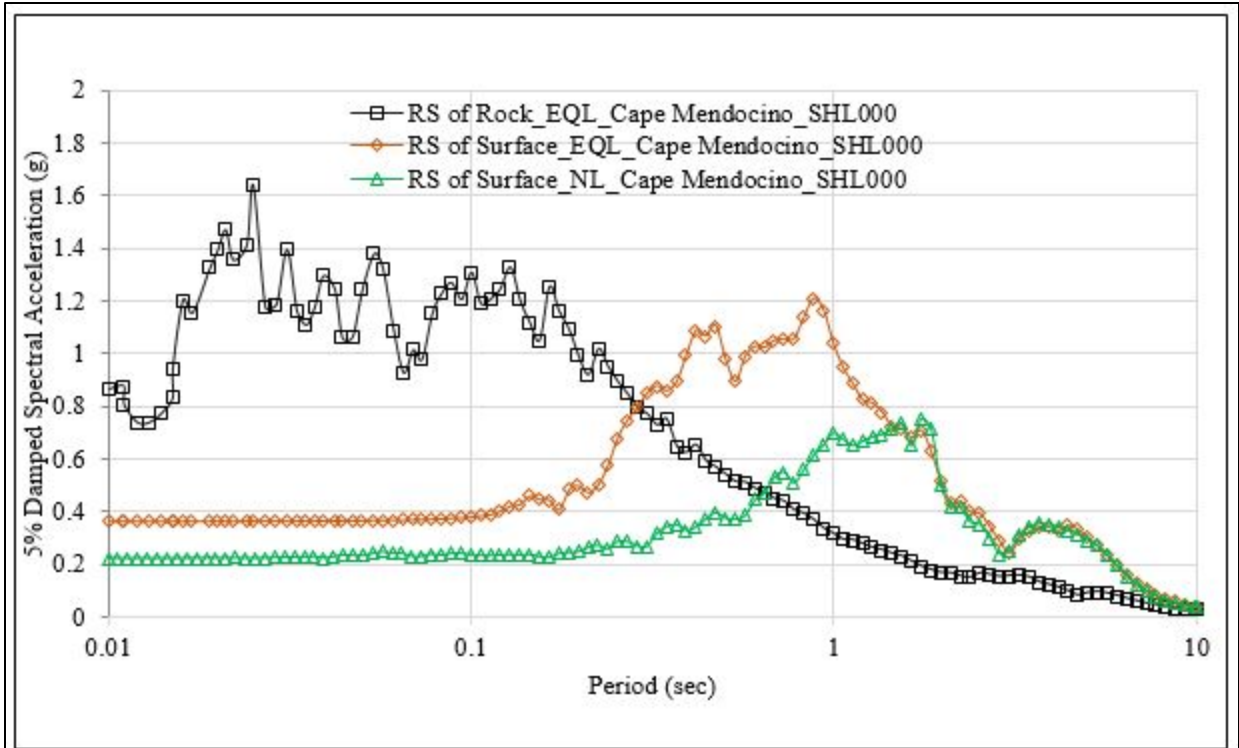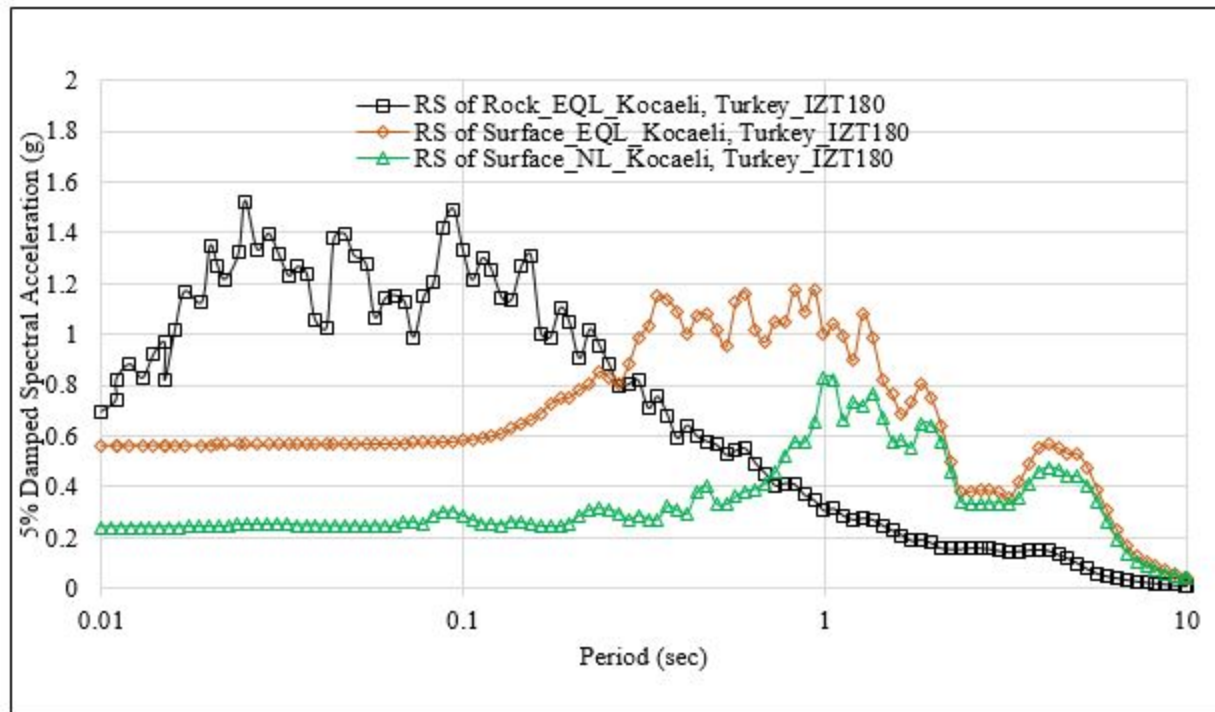
Figure 20. Comparisons between EQL and NL Analyses Results for Each Type of Input Rock Motion for Manila: (a) Cape Mendocino, USA, and (b) Kocaeli, Turkey.

Figure 21. Summary of Response Spectrum for EQL Analyses for Manila.

Figure 22. Summary of Response Spectrum for NL Analyses for Manila.

The chart legend includes the following series:

- RS of Rock_NL_Cape Mendocino_SHL000
- RS of Rock_NL_Chi-Chi, Taiwan _TCU046-W
- RS of Rock_NL_Chi-Chi, Taiwan _TCU047-N
- RS of Rock_NL_Chi-Chi, Taiwan _TCU095-W
- RS of Rock_NL_Chi-Chi, Taiwan _TCU128-N
- RS of Rock_NL_Chi-Chi, Taiwan _TCU136-N
- RS of Rock_NL_Imperial Valley_H-SUP135
- RS of Rock_NL_Kocaeli, Turkey_ARC000
- RS of Rock_NL_Kocaeli, Turkey_IZT180
- RS of Rock_NL_Loma Prieta_BRN090
- RS of Rock_NL_San Fernando_FSD172
- RS of Rock_NL_Tabas, Iran_DAY-TR
- Average RS at Surface Layer_NL, PSA (g)
- RS of Surface_NL_Cape Mendocino_SHL000
- RS of Surface_NL_Chi-Chi, Taiwan _TCU046-W
- RS of Surface_NL_Chi-Chi, Taiwan _TCU047-N
- RS of Surface_NL_Chi-Chi, Taiwan _TCU095-W
- RS of Surface_NL_Chi-Chi, Taiwan _TCU128-N
- RS of Surface_NL_Chi-Chi, Taiwan _TCU136-N
- RS of Surface_NL_Imperial Valley_H-SUP135
- RS of Surface_NL_Kocaeli, Turkey_ARC000
- RS of Surface_NL_Kocaeli, Turkey_IZT180
- RS of Surface_NL_Loma Prieta_BRN090
- RS of Surface_NL_San Fernando_FSD172
- RS of Surface_NL_Tabas, Iran_DAY-TR

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Figure 23. Delineated Design Spectrum for Manila, AR.

From Figure 23, it has been found that there is a reduction of 33% from the PGA from 0.01 to approximately 0.345 seconds compared to the seismic accelerations determined by the AASHTO method. From 0.345 to approximately 0.775 seconds, the site-response spectrum remains between the 2/3 and the regular 1 (one) AASHTO general procedure response spectrum. From 0.775 to the rest of the periods, the spectrum is greater than that of the AASHTO. For Manila (Site 02), the maximum surface site response amplification was found to be 4~5 times higher than that of bedrock. This amplification was found mostly for long period (> 1 sec.).

Site 03: Fontaine

For Site 03 (Fontaine), the maximum shear strain was found to be 0.100% for EQL analyses and 0.078 % for NL analyses. A summary of maximum shear strain for EQL and NL analyses is shown in Figure 24. A sample comparison between the EQL and NL methods is shown in Figure 25.

Figure 24. Summary of Maximum Shear Strain for Fontaine: (a) EQL, And (b) NL Analyses.
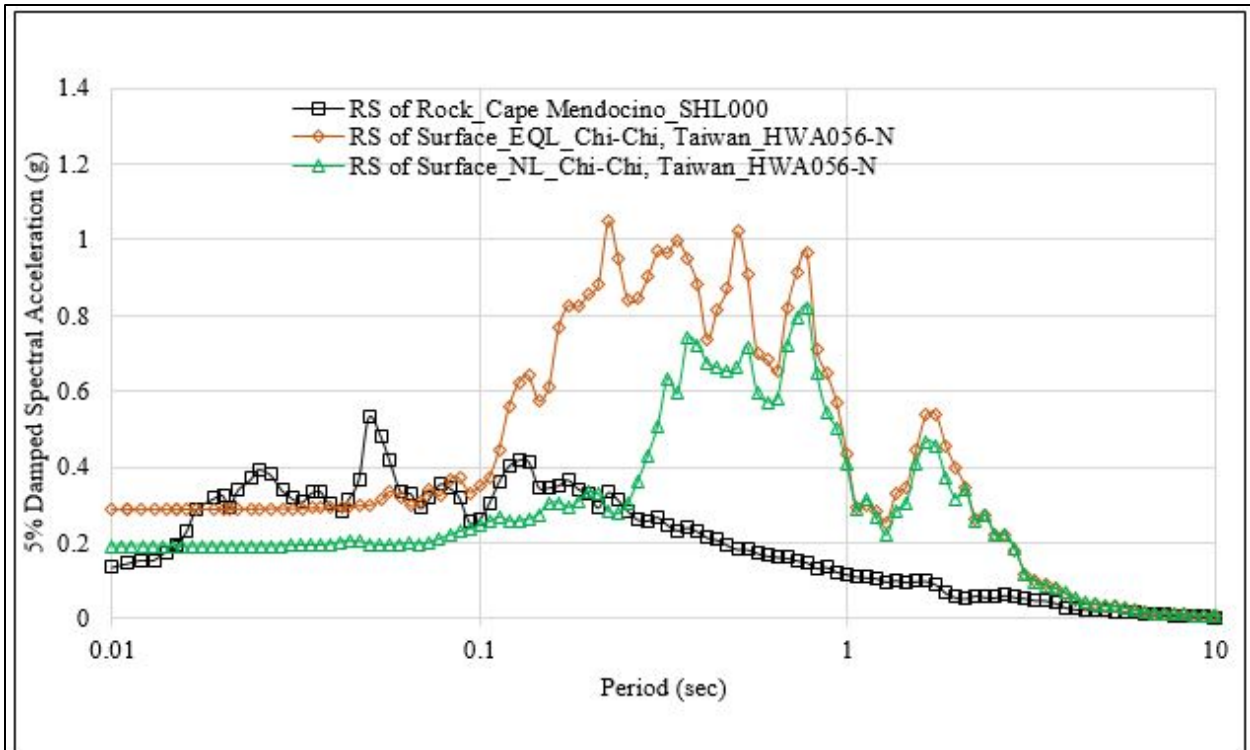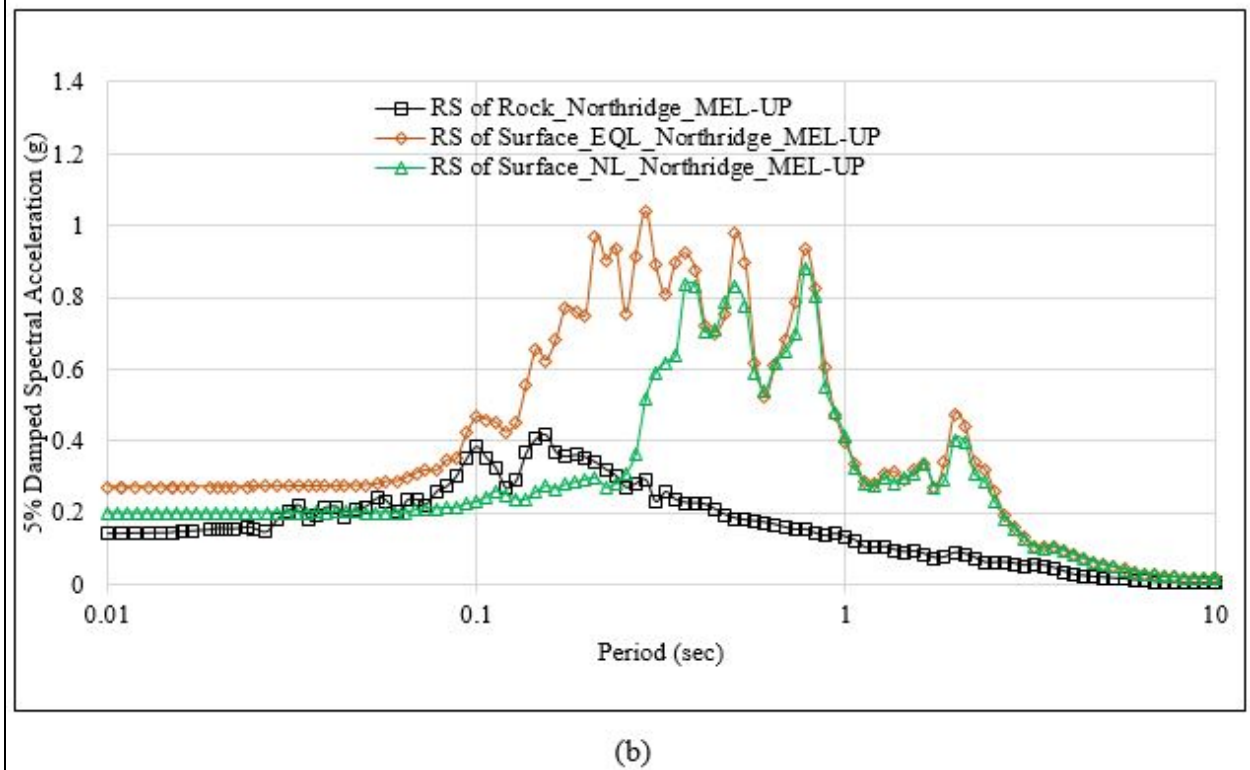
Figure 25. Sample Comparisons between EQL and NL Analyses Results for: (a) Chi-Chi, Taiwan, and (b) Northridge, USA.

Summary data of seismic site response analyses for Site 03 (Fontaine) both EQL and NL methods are given in Figures 26 and 27. Delineated Design Spectrum has been shown in Figure 28.


Figure 26. Summary of Response Spectrum for EQL Analyses for Fontaine.

Figure 27. Summary of Response Spectrum for NL Analyses for Fontaine.
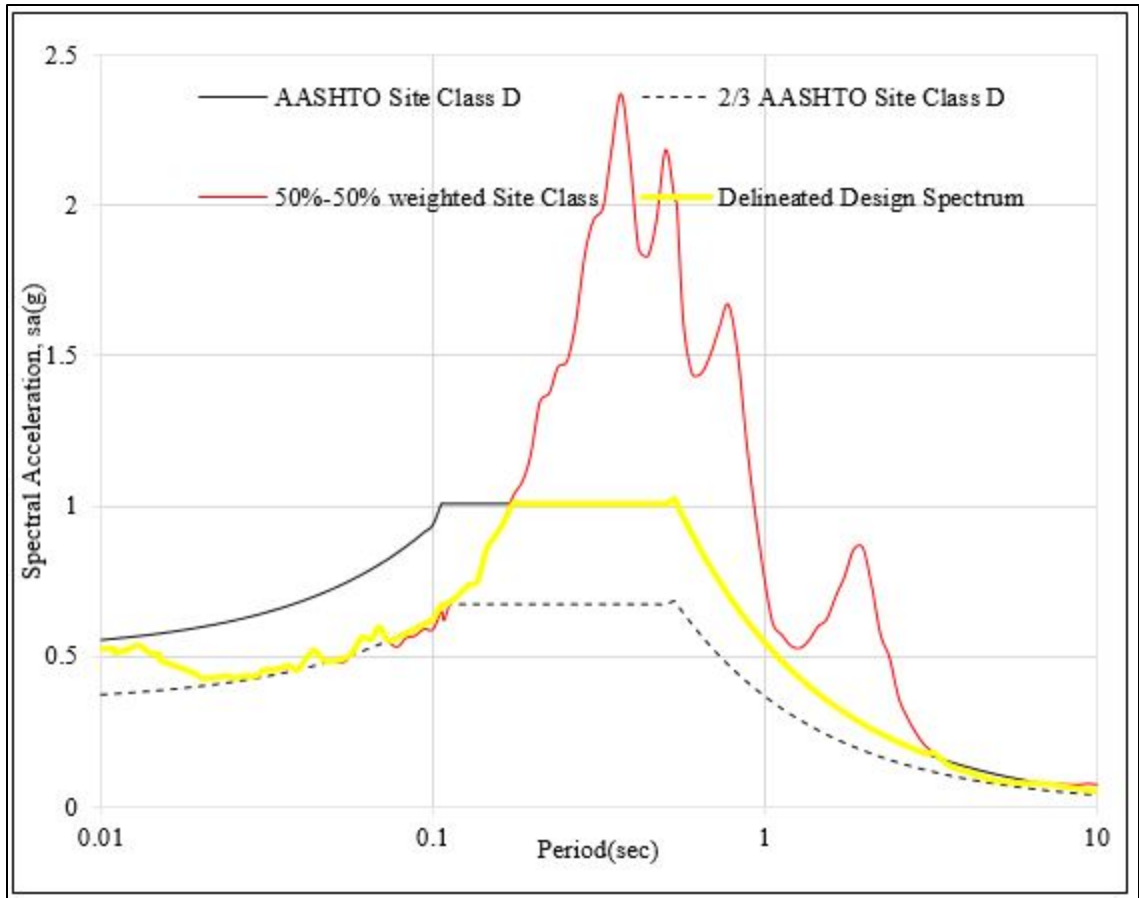
Figure 28. Delineated Design Spectrum for Fontaine, AR.

From Figure 28, it is seen that, from 0.01 to 0.113 seconds, there is a reduction of approximately 33% from the PGA compared to that determined by the AASHTO general procedure. From 0.113 to approximately 0.186 seconds, the site-response spectrum remains between the 2/3 and the regular (1) AASHTO general procedure response spectrum. From 0.186 to approximately 3.236 seconds, the spectrum is greater than that of AASHTO. From 3.326 seconds to the rest of the periods, the spectrum remains between the 2/3 and the regular (1) AASHTO general procedure response spectrum. For this site (Fontaine), the maximum surface site response amplification was found to be 7 times higher than that of bedrock. This amplification was found mostly for long periods (> 1 sec.).

Site 04: Bay

The maximum shear strain was found to be 0.165% for both EQL analyses and NL analyses. Summary of maximum shear strain for EQL (a) and NL (b) analyses has been given in Figure 29. Comparisons between EQL and NL method for each type of input rock motion have been shown in Figure 30.
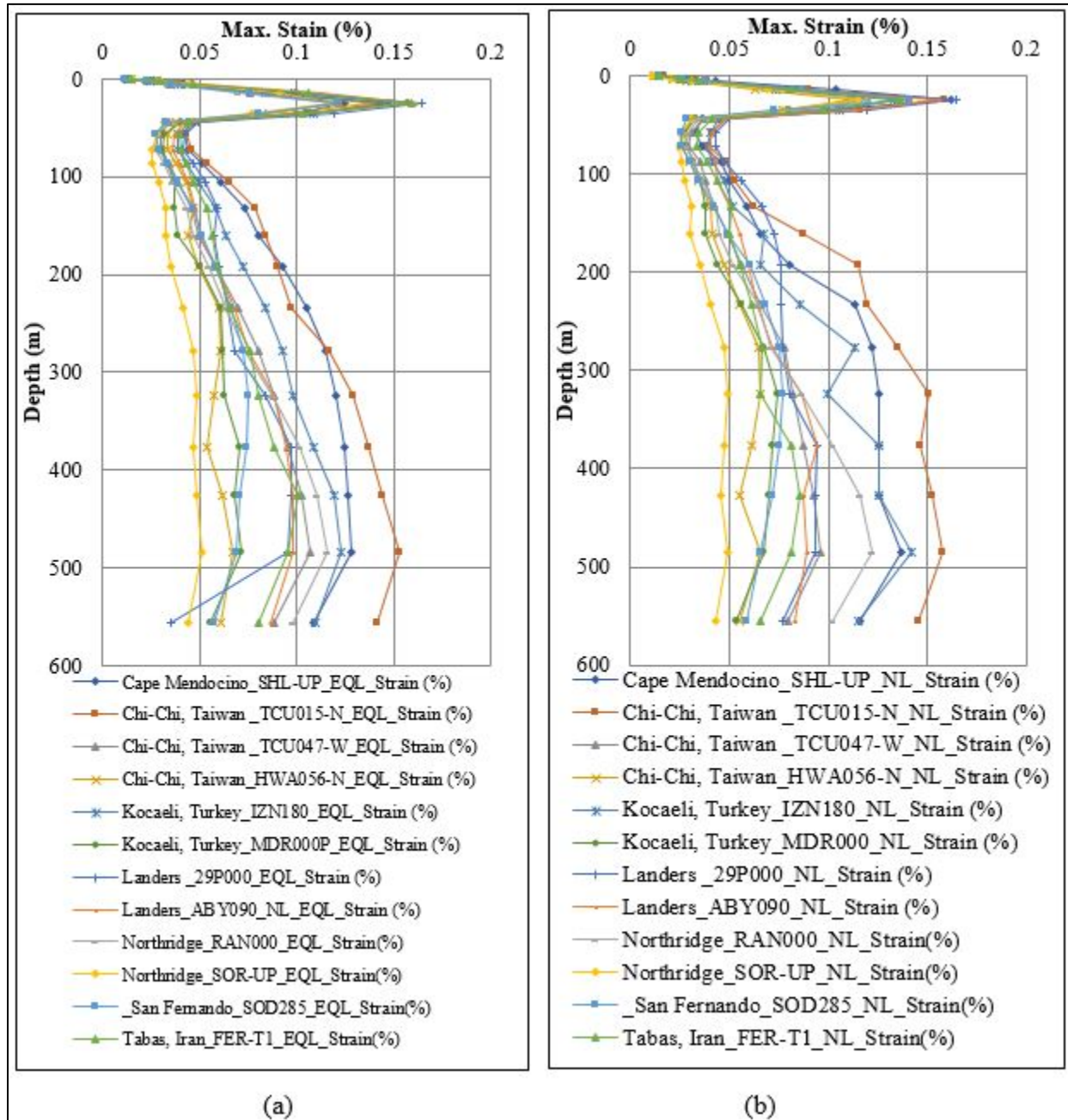
Figure 29. Summary of Maximum Shear Strain for Bay: (a) EQL, and (b) NL Analyses.
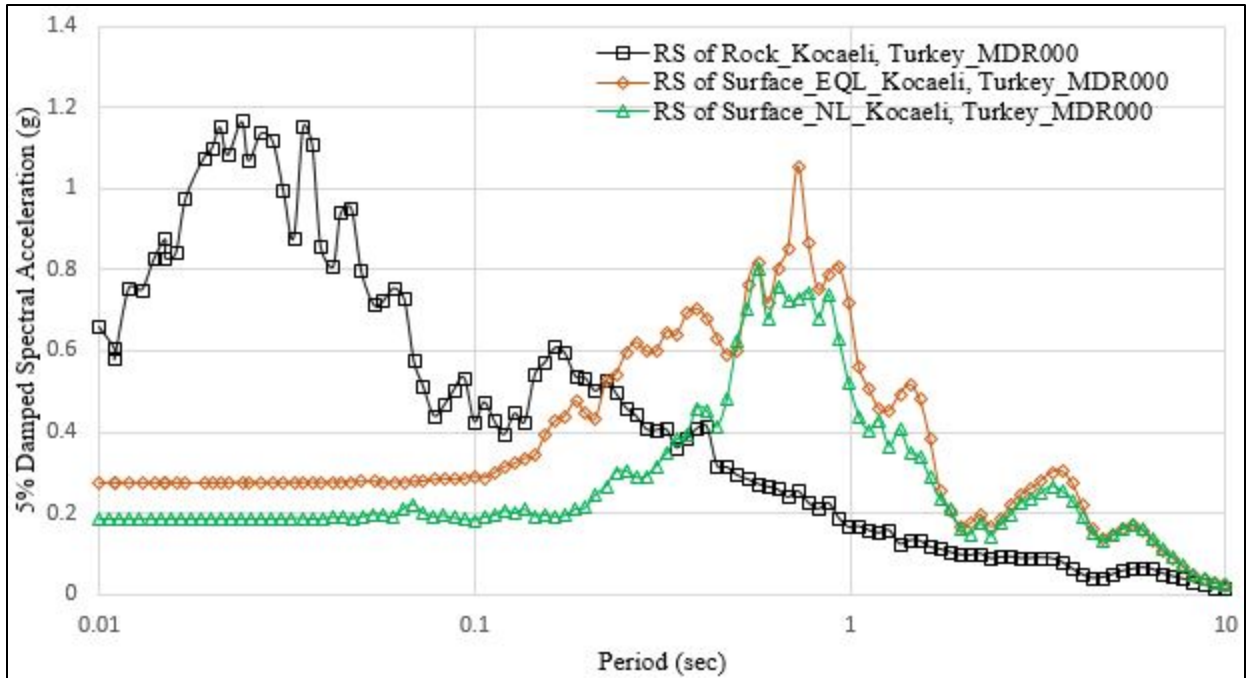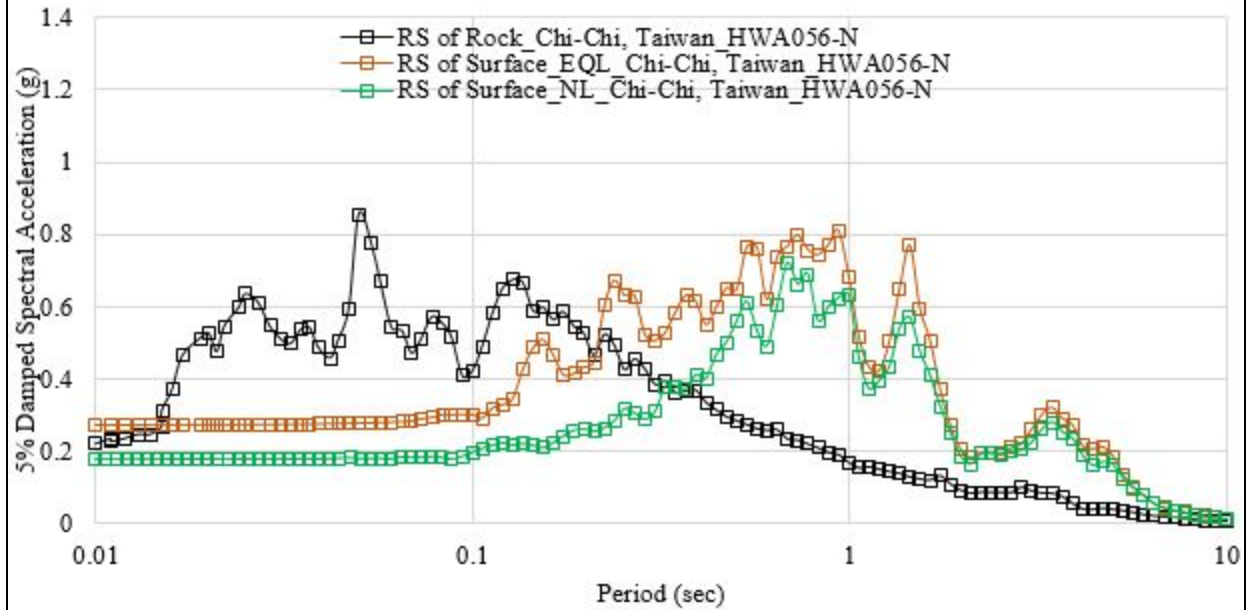
Figure 30. Samples Comparisons between EQL and NL Analyses Results for Input Rock Motion: (a) Kocaeli, Turkey, and (b) Chi-Chi, Taiwan.

Summary data of seismic site response analyses for both EQL and NL methods are presented in Figures 31 and 32. The delineated design spectrum is shown in Figure 33.
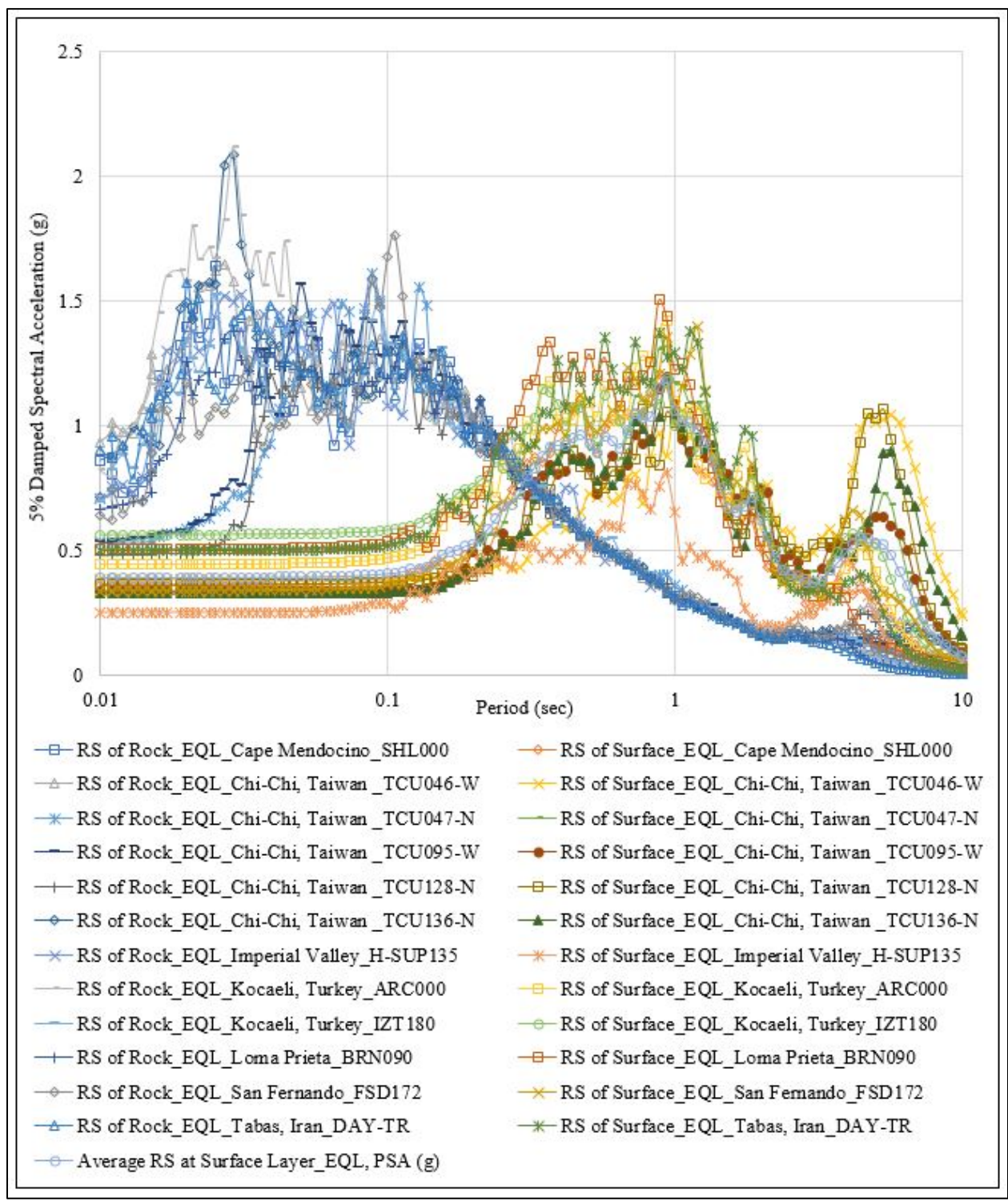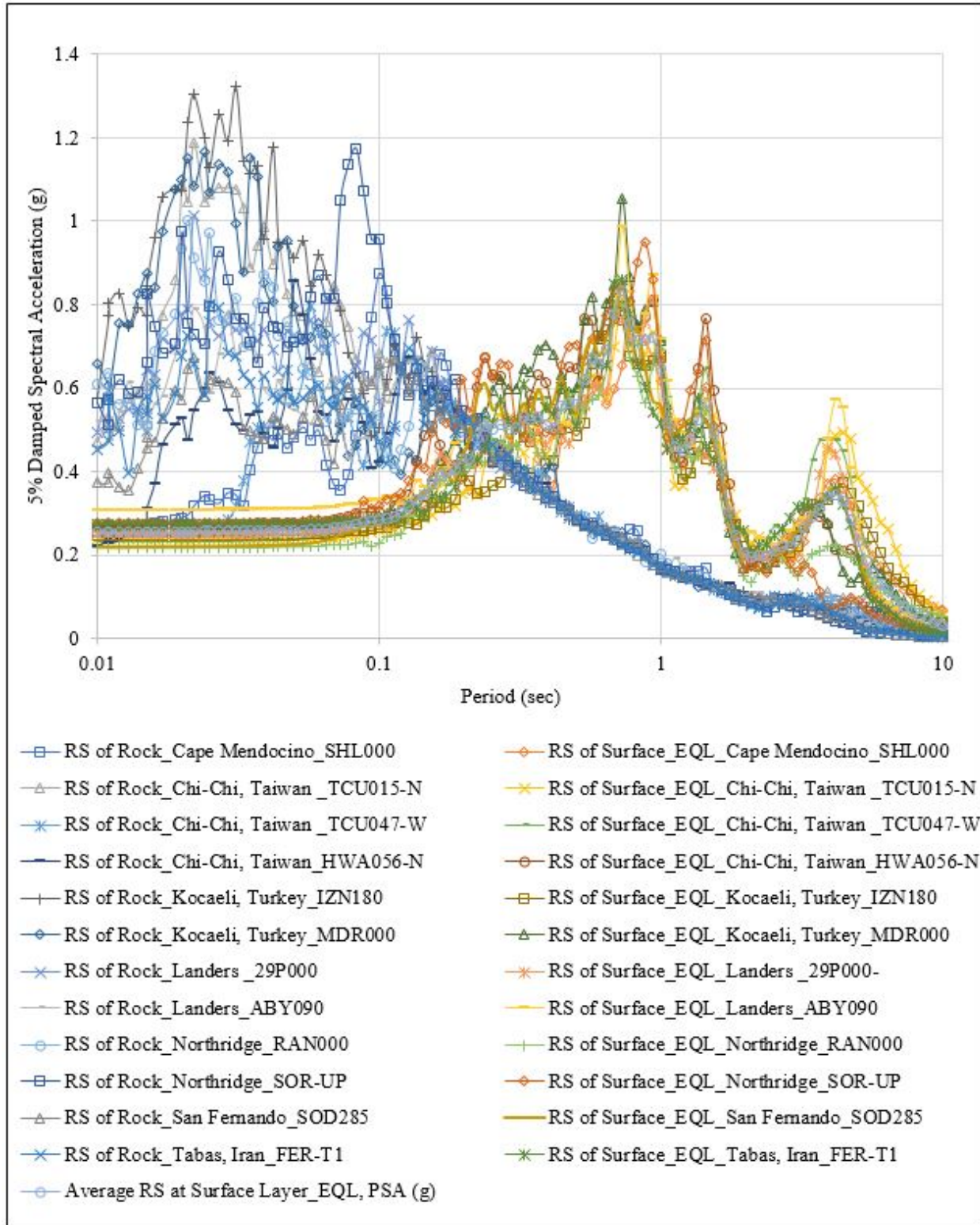

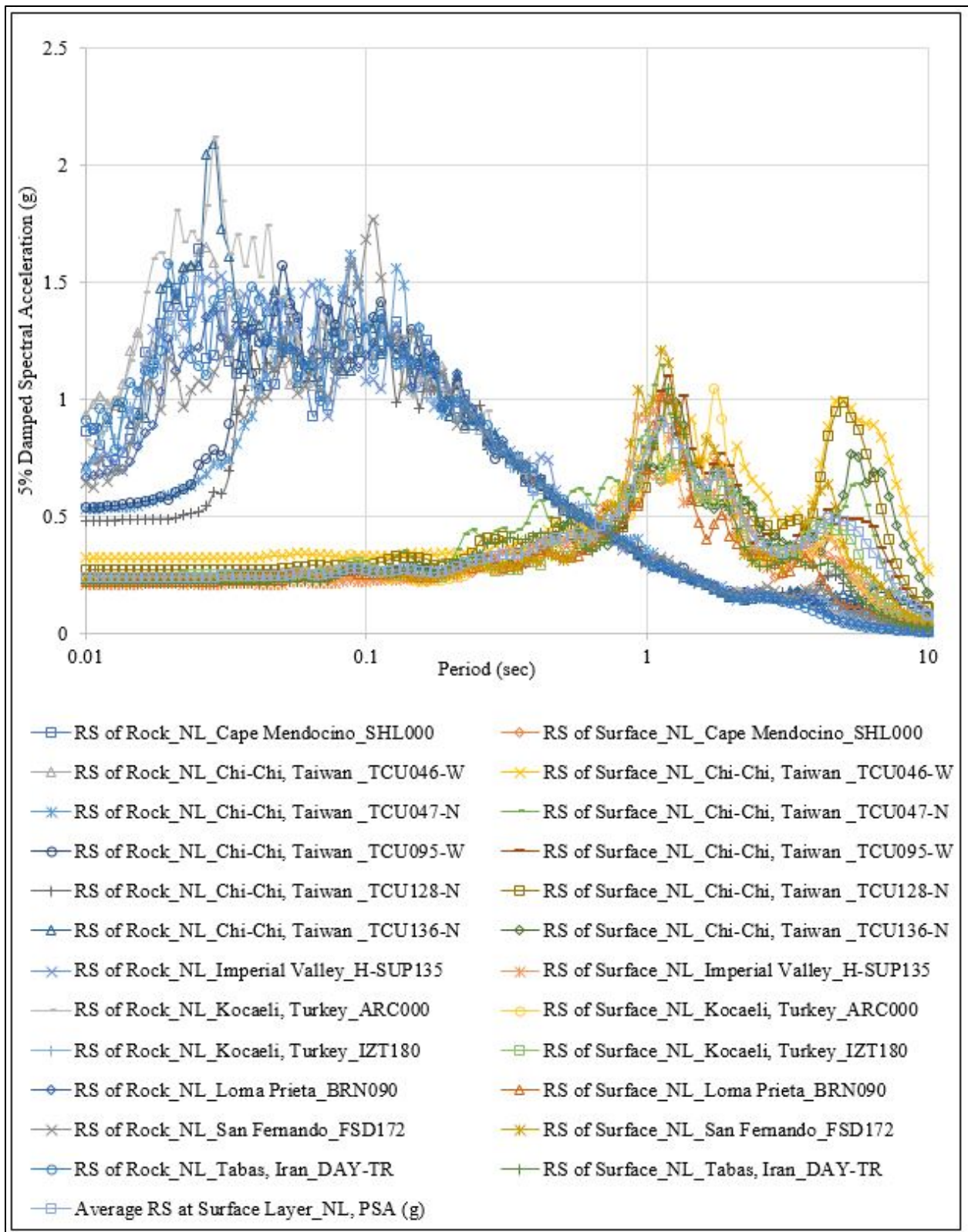Figure 31. Summary of Response Spectrum for EQL Analyses for Bay.
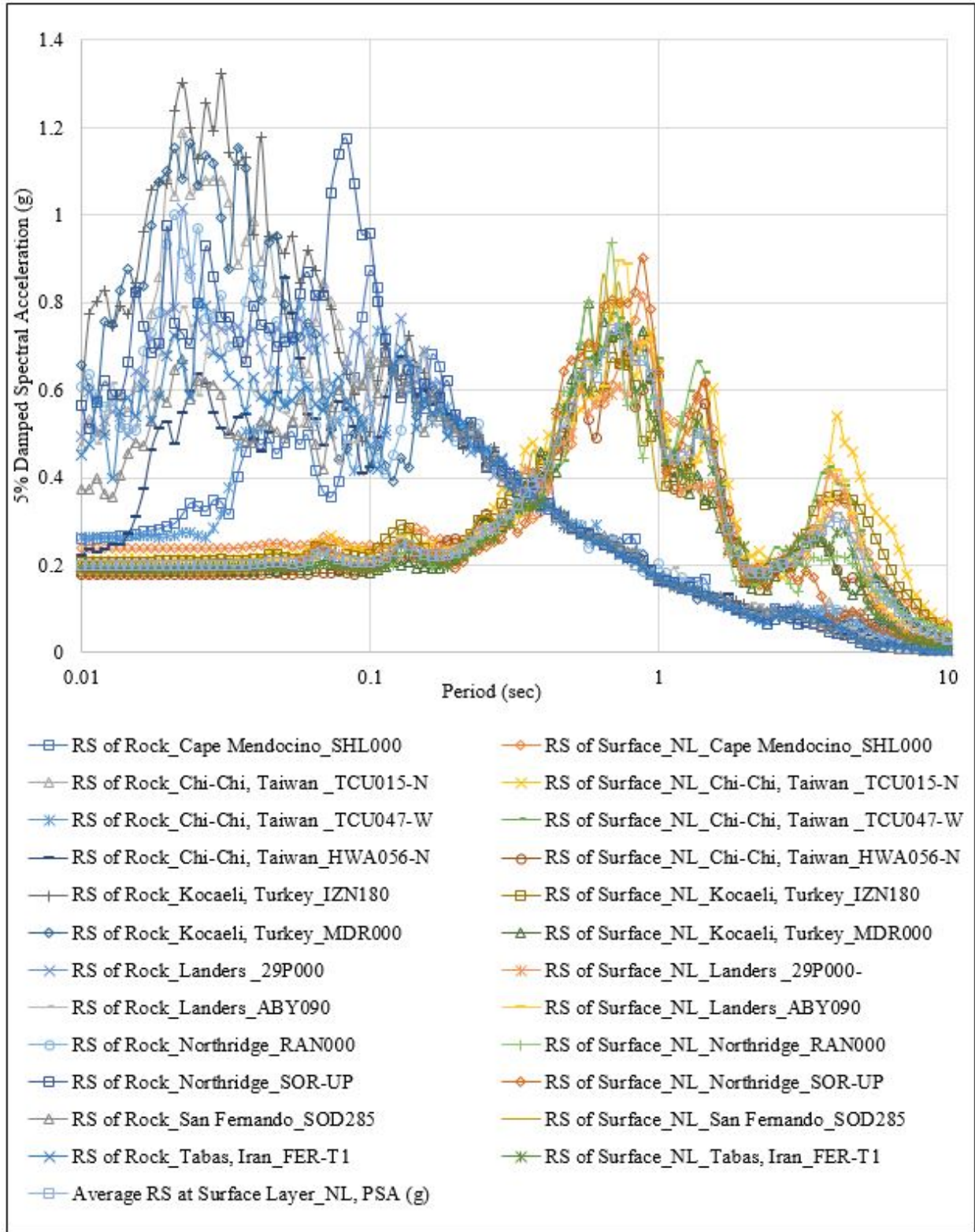
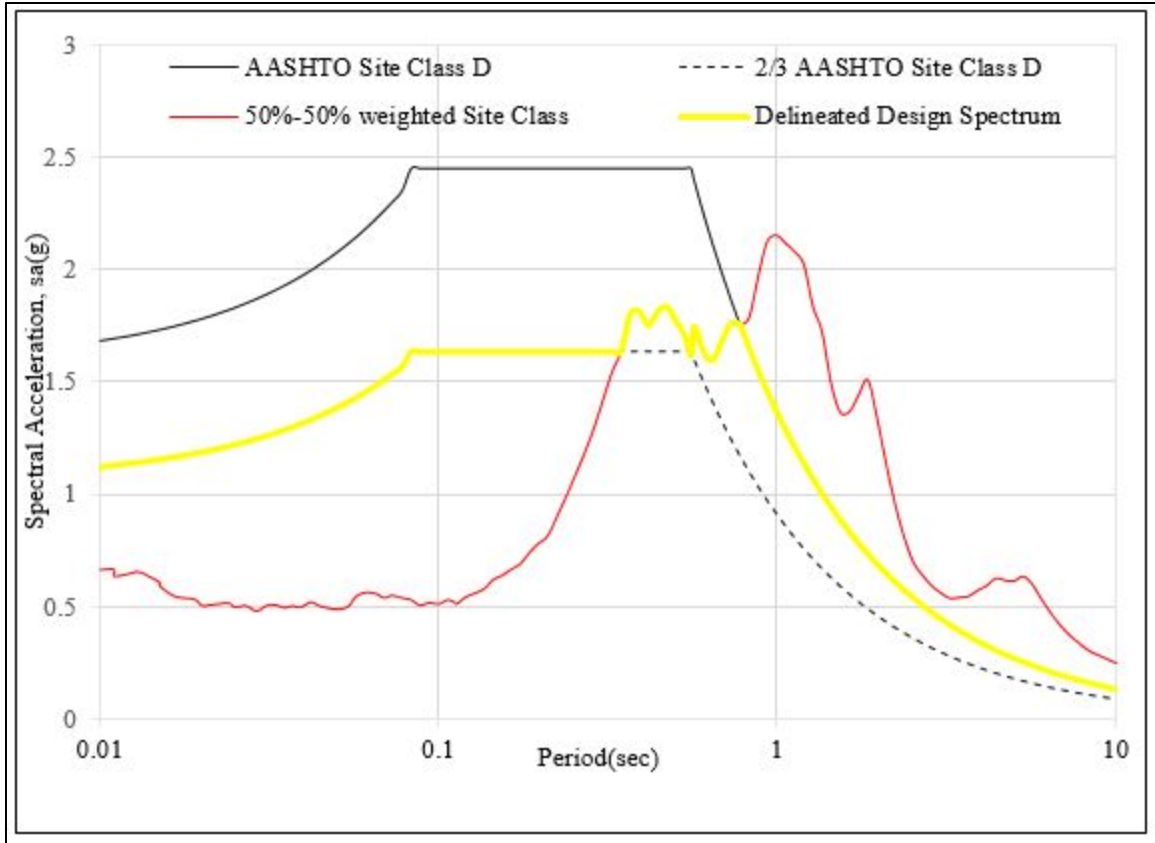**Figure 32. Summary of Response Spectrum for NL Analyses for Bay.**
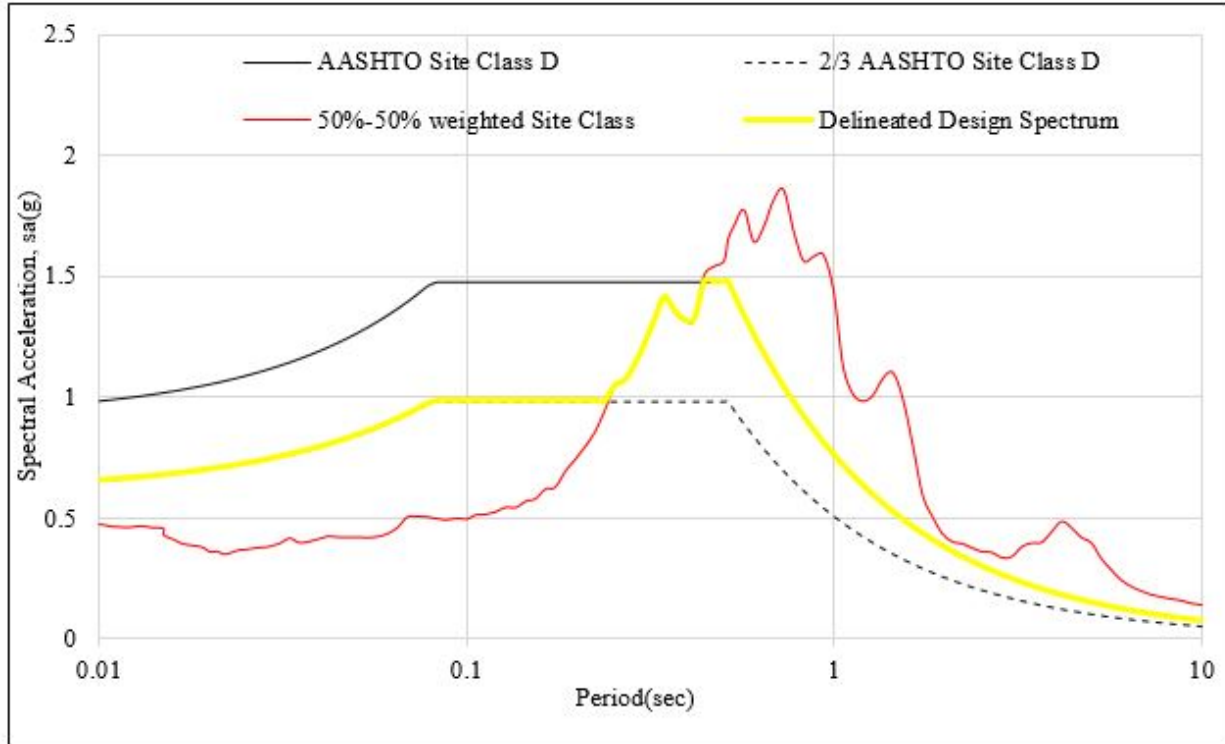
Figure 33. Delineated Design Spectrum for Bay, AR.

From Figure 33, it is seen that, from 0.01 to 0.238 seconds, there is a reduction of 33% from the PGA compared to that of the AASHTO general procedure. From 0.238 to approximately 0.443 seconds, the site-response spectrum remains between the 2/3 and the regular 1 (one) AASHTO general procedure response spectrum. From 0.443 to the rest of the periods, the spectrum is greater than that of AASHTO. For this site (Bay; Site 04), the maximum surface site response amplification was found to be 5 times higher than that of bedrock. This amplification was found mostly for long periods (> 1 sec.).

Site 05: Marked Tree

The maximum shear strain for EQL analyses was found to be 0.330% and 0.342 % for NL analyses. Summary data of maximum shear strain for EQL and NL analyses are given in Figure 34. Sample comparisons between EQL and NL methods for each type of input rock motion are shown in Figure 35.
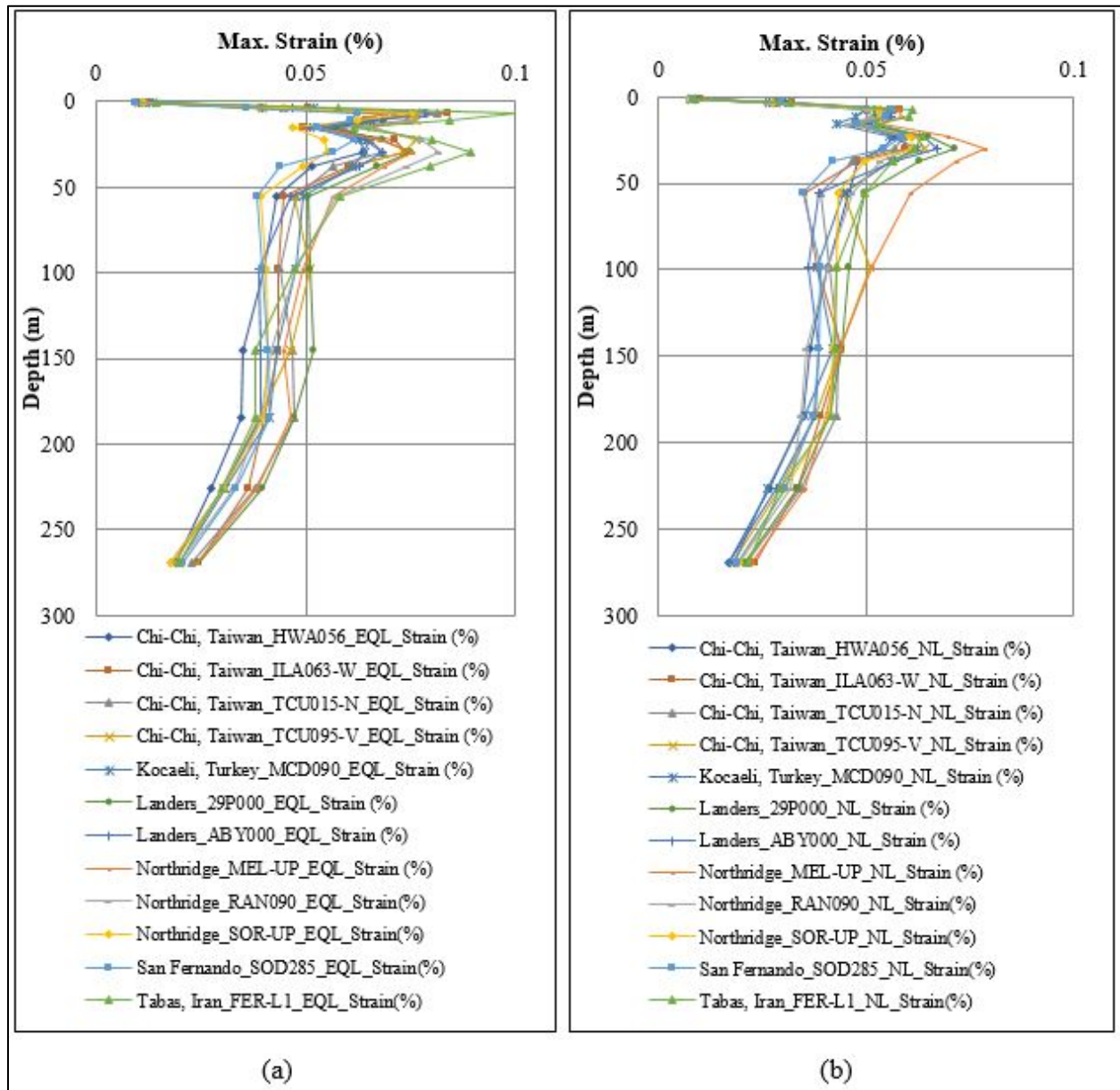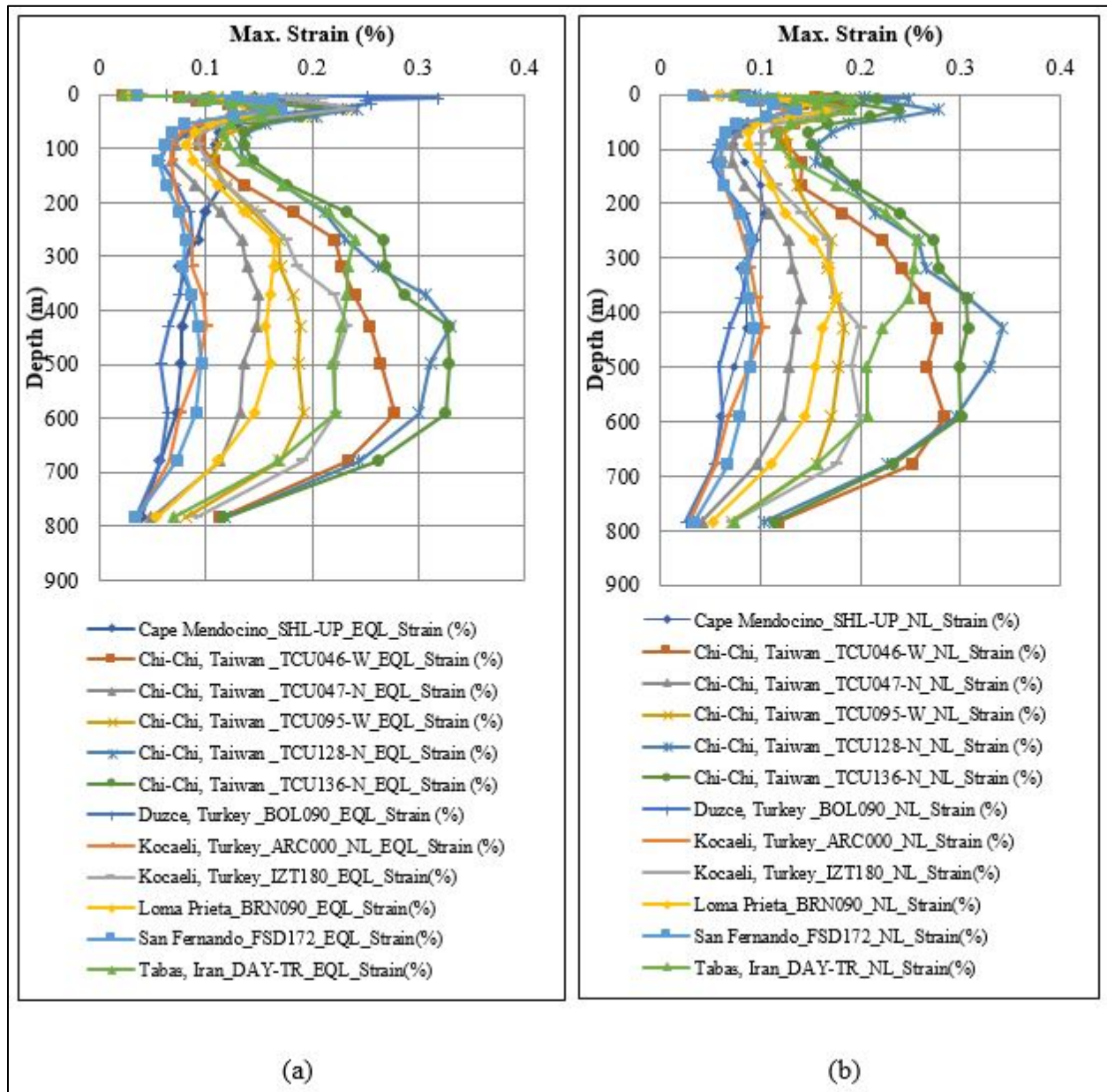
Figure 34. Summary of Maximum Shear Strain for Marked Tree: (a) EQL, and (b) NL Analyses.
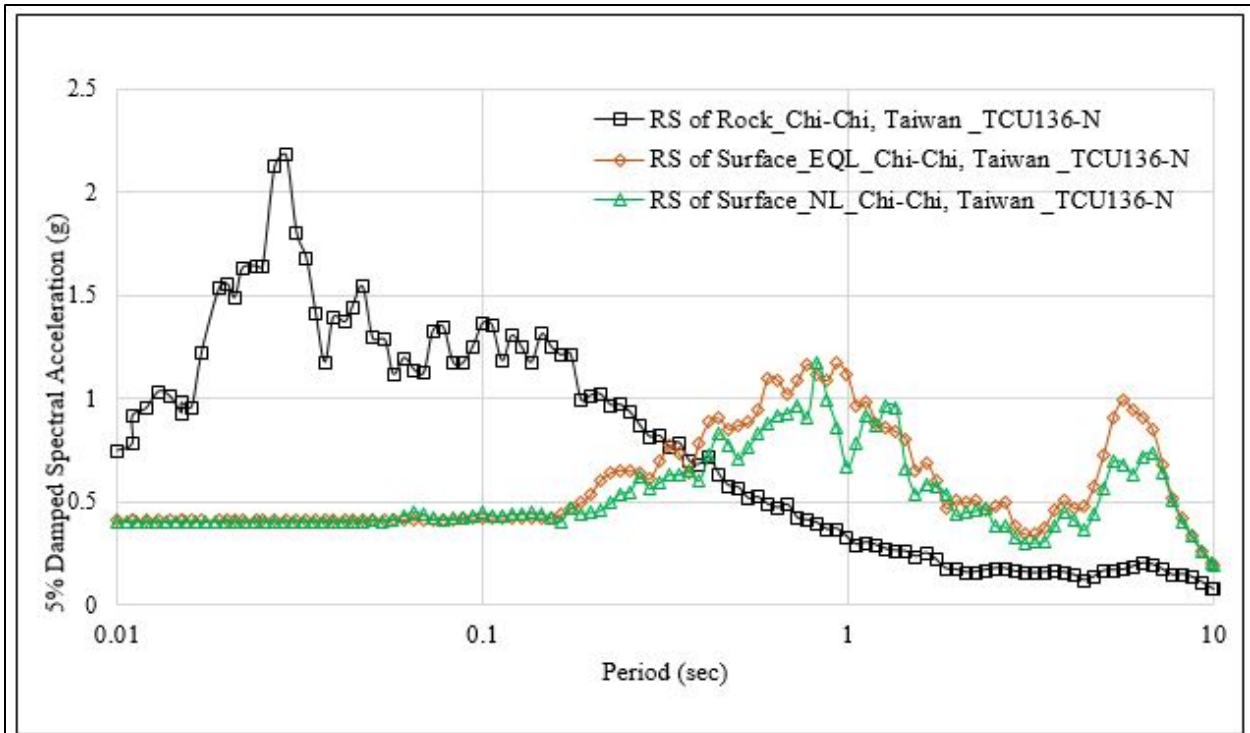
(a)
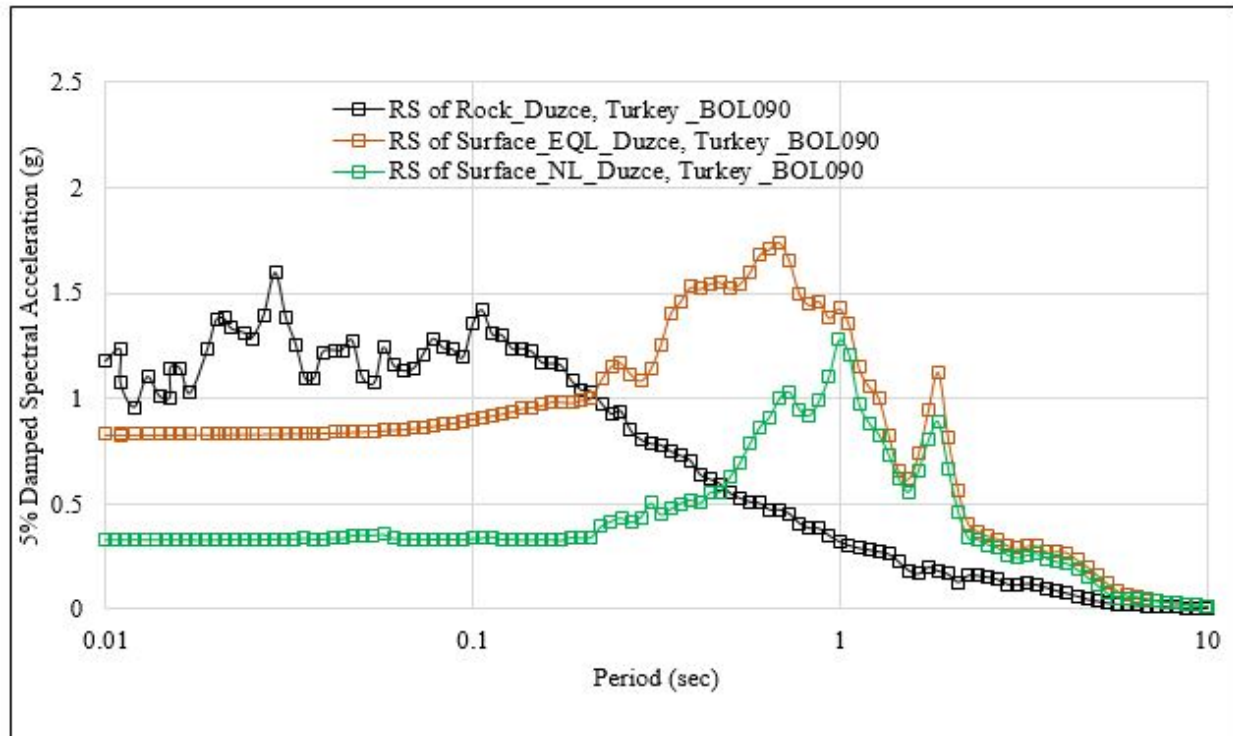
(b)

Figure 35. Sample Comparisons between EQL and NL Analyses Results for Marked Tree for Each Type of Input Rock Motion: (a) Chi-Chi, Taiwan, and (b) Duzce, Turkey.

Summary data of seismic site response analyses for both EQL and NL method are given in Figures 36 and 37. The delineated design spectrum is shown in Figure 38.
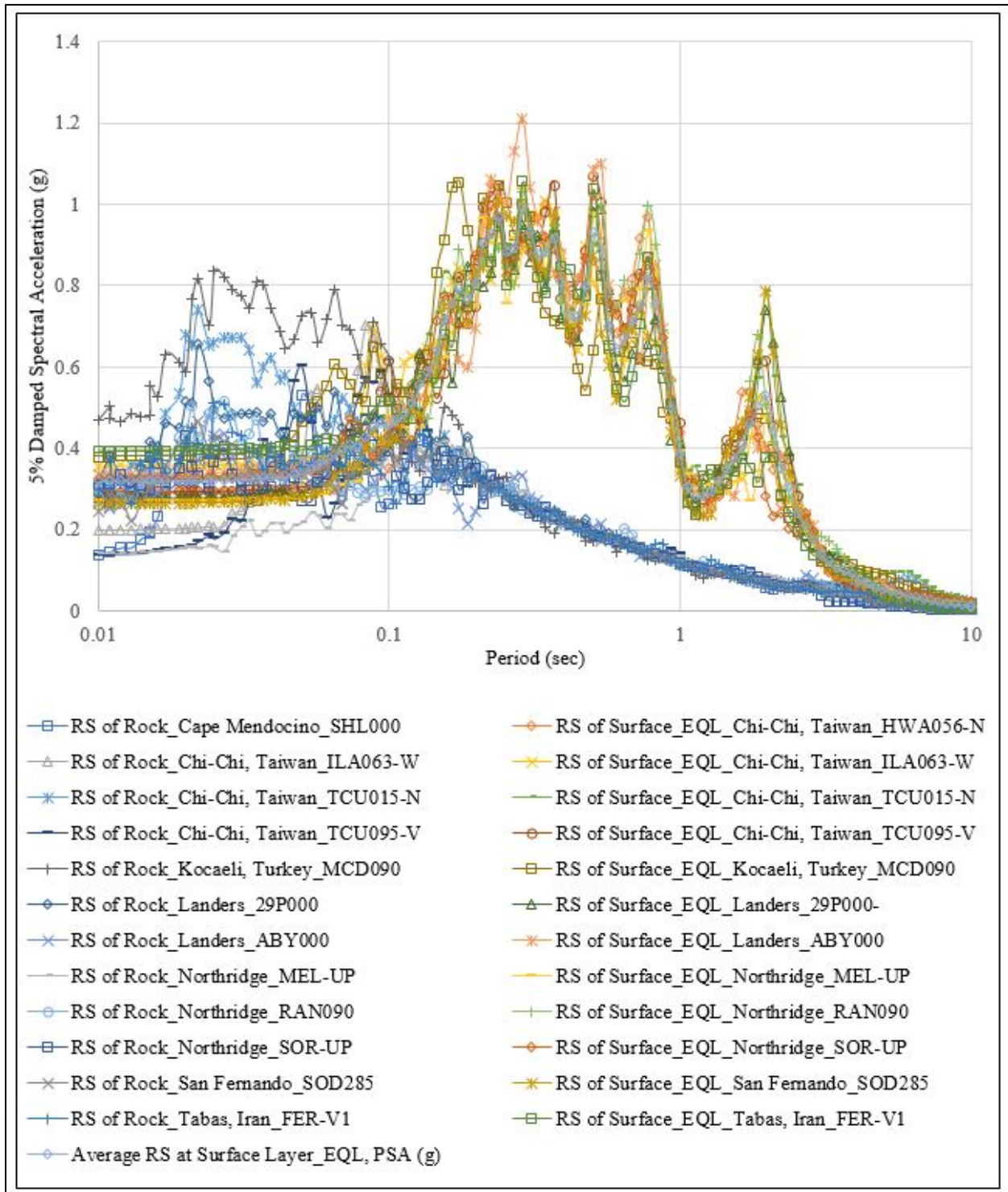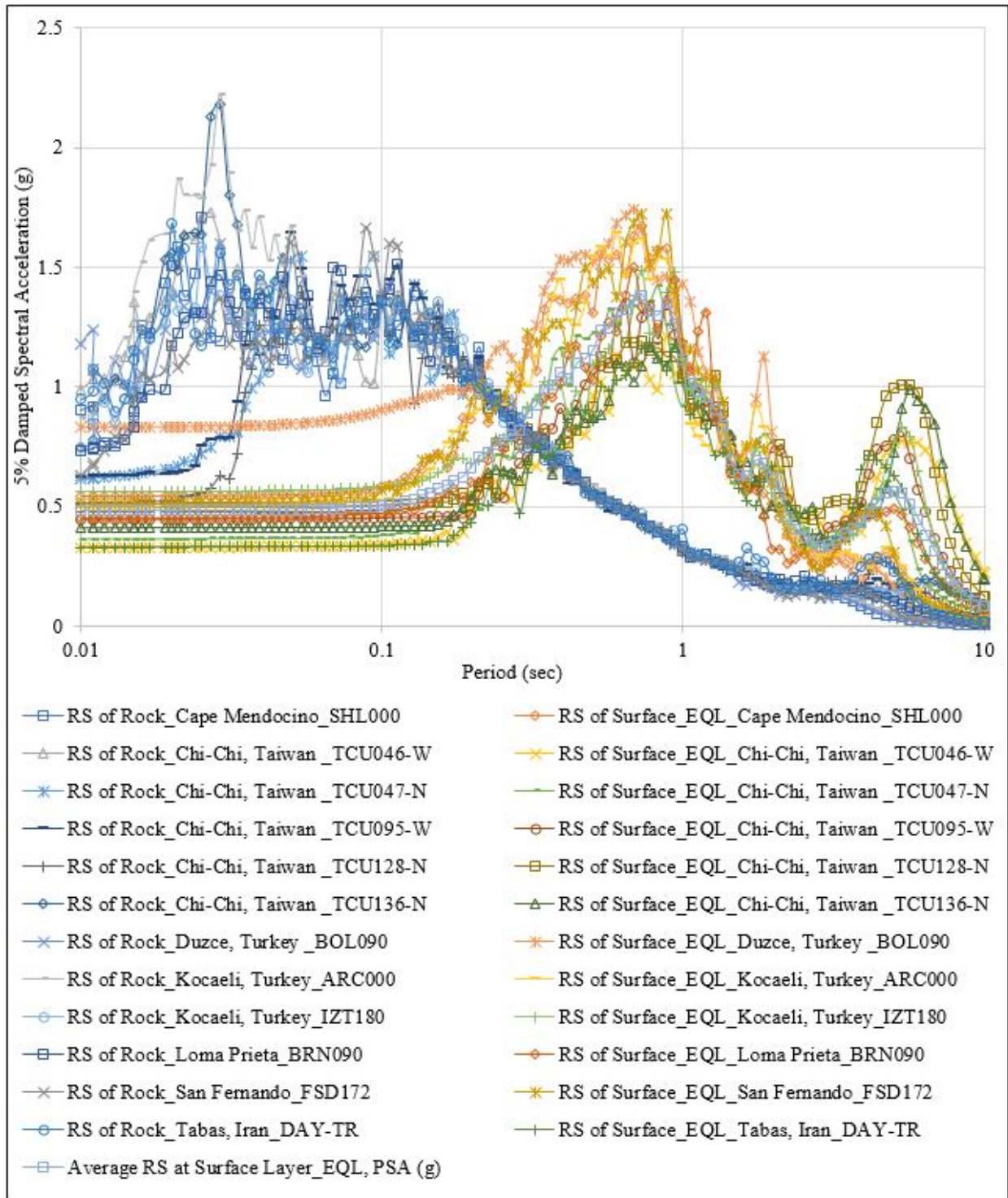

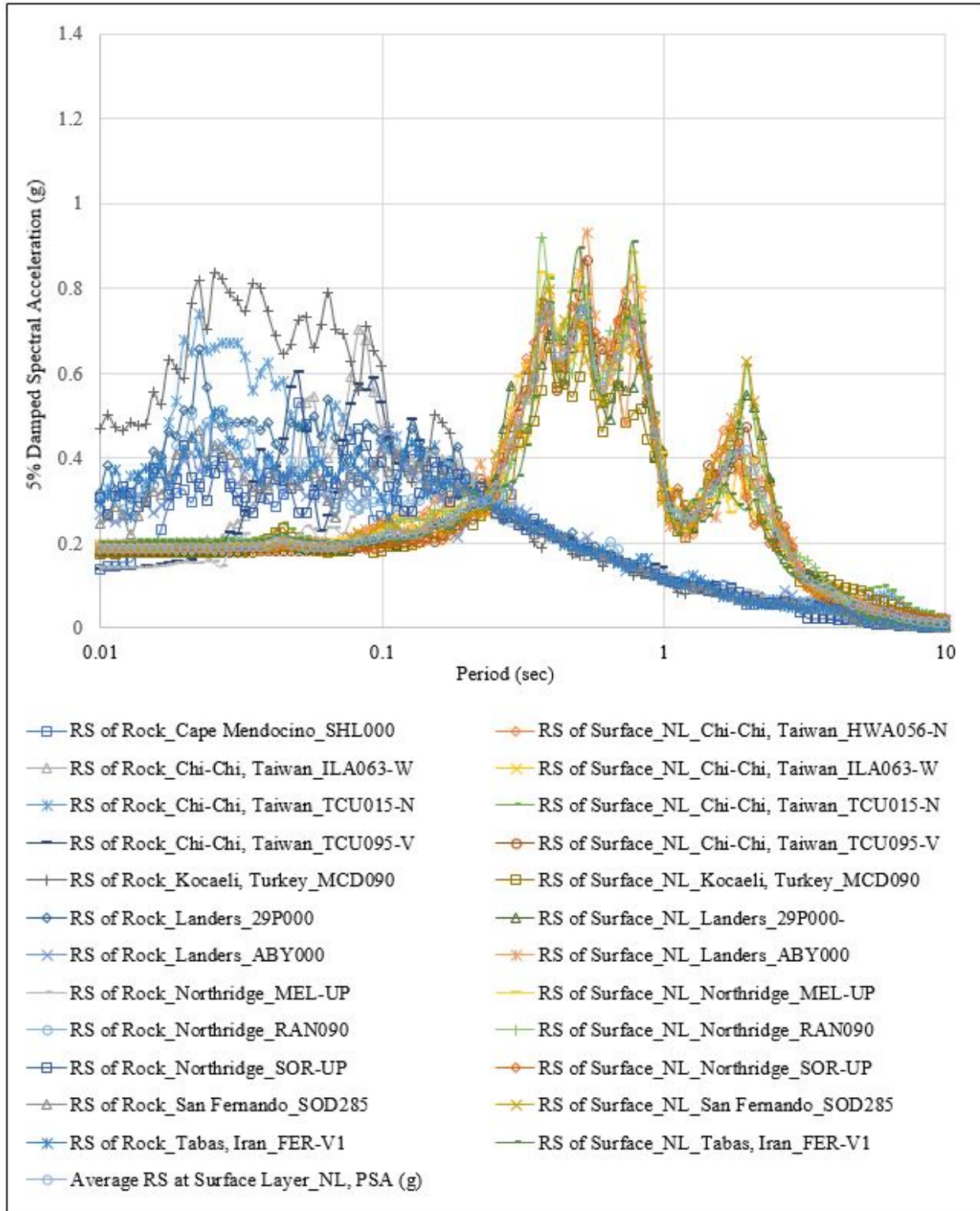*Figure 36. Summary of Response Spectrum for EQL Analyses for Marked Tree.*
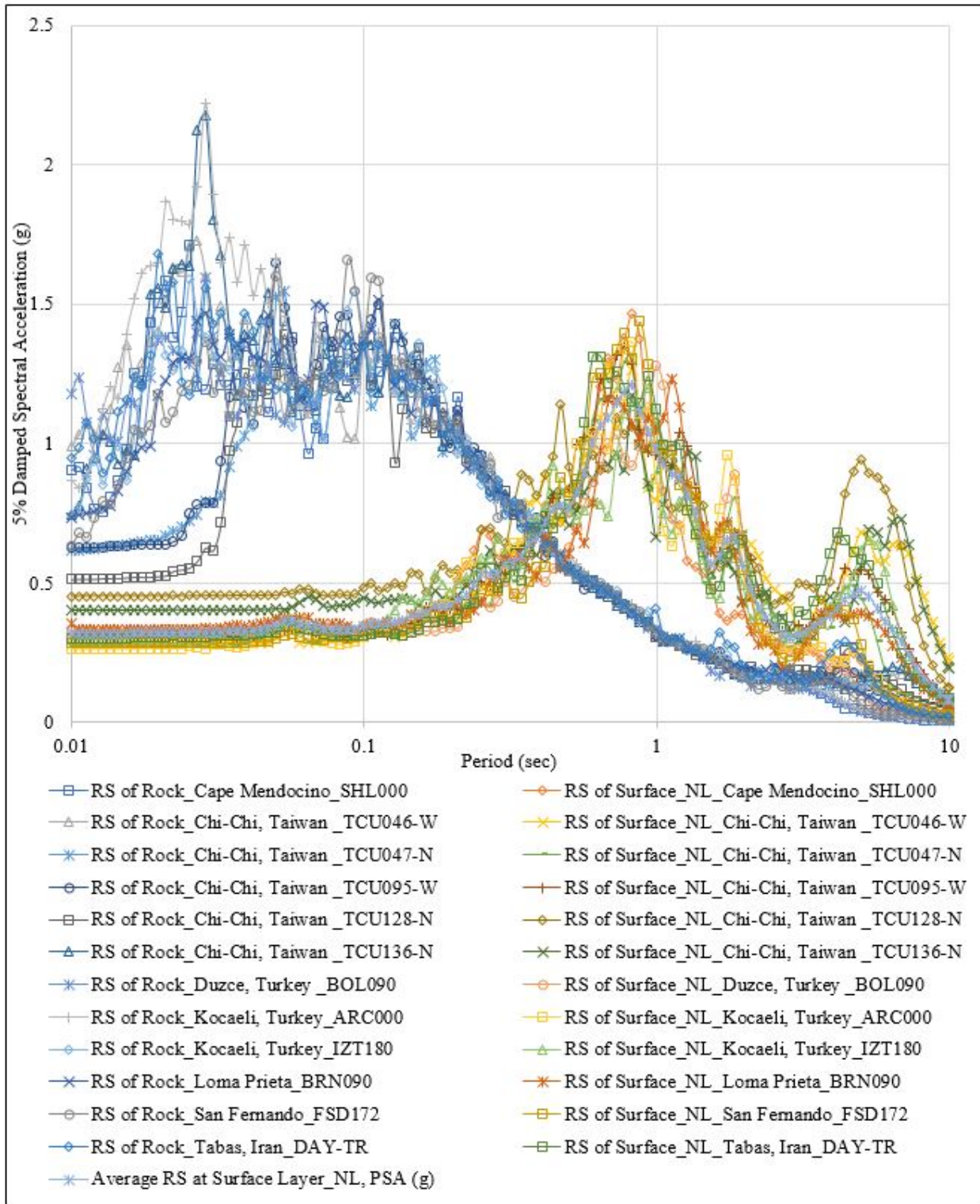
*Figure 37. Summary of Response Spectrum for NL Analyses for Marked Tree.*
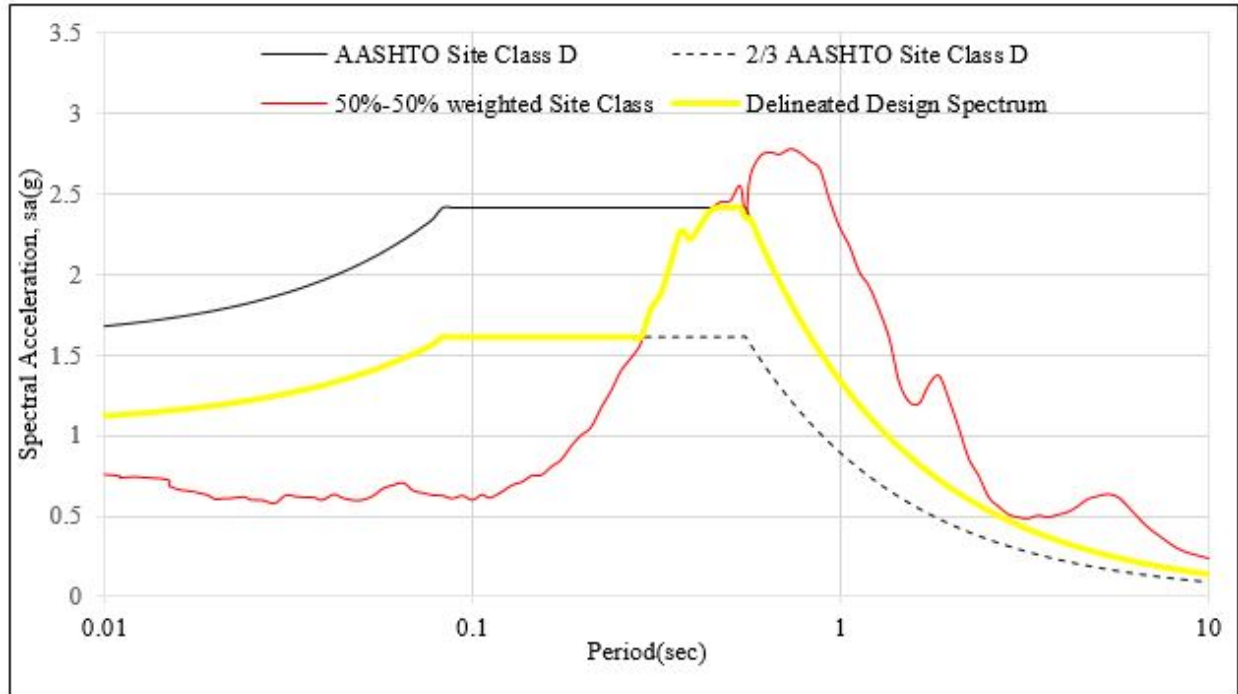
Figure 38. Delineated Design Spectrum for Marked Tree, AR.

From figure 38, it is seen that there is a reduction of 33% of the PGA from 0.01 to approximately 0.287 seconds) compared to the seismic accelerations determined by the AASHTO general procedure. From 0.238 to approximately 0.502 seconds, the site-response spectrum remains between the 2/3 and the regular 1 (one) AASHTO general procedure response spectrum. From 0.502 seconds to the rest of the periods, the spectrum is greater than that of AASHTO. For Marked Tree (site 05), maximum surface site response amplification was found to be 5 times higher than that of bedrock. This amplification was found mostly for long period (> 1 sec.).

From the aforementioned figures (Figures 15-17 20-22, 25-27, 30-32, and 35-37) the differences between the EQL and NL analyses can easily be distinguished. This difference is well pronounced during the propagation of strong ground motion. From these figures, it is seen that the EQL analysis shows higher responses (most of the cases) at periods less than 1 sec. compared to the NL analysis. The EQL also shows higher responses at periods higher than 1 sec. This is because a very flat response at high frequencies can be produced in EQL analysis because of high damping values at sites where high shear strains are expected (48). Besides, using constant stiffness and damping throughout the analysis (49), the EQL might show higher responses both at short (< 1 sec) and long (>1 sec) periods. On the other side, the soil behavior under large strains from strong ground motions at soft soil sites can be predicted in a better way in the NL analyses because of accounting changes in soil properties at each time step (46). However, the EQL analysis is still the most common method in practice, and it has been proved to be valuable for studies within the NMSZ. That is why the final site-response spectra for these sites have been developed based upon a weighted average (50%+50%) of both EQL and NL analyses results.

From these results, it can be said that the EQL overestimates the amplification factor both at short and long periods. Because of its simplicity in the calculation of the dynamic properties of soil, the EQL overestimates the amplification factor. On the other hand, in the NL analysis, the surface response spectrum is lower than that of input motion in a short period. The NL analysis can calculate the dynamic properties of soil for large strain using the GQ/H model in DEEPSOIL 7.0. It counts the impact of the largely deposited subsoil of a zone. Hence, the entire thickness of the soil column should be considered while performing the SSGMRA.

The surface site response gets more amplified when the target motions are weak (i.e. lower PGA, SS, S1 values). That means amplification occurs (short period < 1.0 sec) earlier when the target motions are weak. The maximum surface site response also gets higher when the target motions are weak. The probable reason behind these scenarios is because the earthquake energy is slowly released through the soil column. Whereas, for strong target motions, the earthquake energy is quickly released through the soil column than that of weak target motions. This contributes to the earlier amplification and higher surface site response when the target motions are weak. Also, from the results obtained using both EQL and NL methods it can be concluded that site response is getting deamplified for short period waves and then amplified for long period waves due to the deep deposition of ME. Because of deamplification, there is a reduction of almost 1/3 in the design acceleration response spectrum for the short period range (<1.0 seconds).

5.2. LIQUEFACTION POTENTIAL ANALYSIS FOR NEA

Green and Clay County

Green County is located on the northeast corner, and at the north of Clay County, of the state of Arkansas. Four CPT sounding data are available for this county.

GEE001

GEE001 is located near the intersection of Green 870 Rd. and Greene 823 Rd. in Green County. The CPT data interpretation shows almost two-meters thick silty sand and sandy silt layers with a sensitive fine-grained soil layer at the top of the 20.5 m soil column. All the other soils up to 20.5 m depth of the columns are sand and silty sand, except about a one-meter thick silty sand and sandy silt layer at a depth of 6.6 m and a very thin silty sand and sandy silt layer at a depth of 9.0 m. The cone resistance increases with the increase of depth of the soil column. The average I_c of the soil column is 1.74. The percent fine content (FC) of the topmost 5 m of soils varies from 1.09% to 36.73%. The FC in other layers below a depth of 5 m varies from 1.14% to 22.19%, with an average value of 7%. The unit weight of soils varies from 13.73 kN/m^3 to 21.43 kN/m^3. The average unit weight of the soil column is 19.11 kN/m^3, and the COV of unit weight of the soils is

0.05 kN/m^3. A detailed depth-wise summary of the basic CPT data interpretation results is shown in Figure 39.

Figure 39. Basic Interpretation of CPT Data of GEE001.

According to the cyclic liquefaction analysis, the CSR is greater than CRR for approximately five different layers within 20.5 m. Depth ranges of the soil layers with a probability of failure greater than 1.0 are 0 to 4.2 m, 5.5 to 10 m, 12.0 to 14.5 m, 15.5 to 17.5 m, and 18.0 to 20.5 m. Overall LPI for the site is around 32.638, which indicates a very high risk of liquefaction. LSN of location is 55.63, which insinuates severe damage to the upper layer of the soils. For the given criteria, the ground surface of the site is prone to a vertical settlement of 30.602 cm and a lateral displacement of 212.43 cm after earthquakes. The overall probability of liquefaction of the site is 98.24%. Parametric analysis of vertical settlements against input PGA shows that PGA values greater than 0.65g do not have any future impact on the overall vertical settlements. The maximum possible value of the vertical settlement is around 32 cm. A significant portion of the settlements can occur for a ground motion with a PGA value of less than 0.35g. However, the LPI for the site keeps increasing with the increment of PGAs of ground motion, with a faster pace at the beginning part, and a slower pace at the last moments. For ground motion with PGA greater than 0.27g is critical for this location and can cause "very high risk" of overall liquefaction. According to sensitivity analysis, PGA values above 0.65g do not affect the value of LSN. However, for ground motion with PGA values above 0.3g can cause severe damage for this location, and major risk of expression of liquefaction can start at a PGA value of 0.25g. The depth-wise distributions of liquefiable layers of the GE001 site along with liquefaction potential, probability of liquefaction, vertical settlements, and lateral displacements are shown in Figure 40. For this sounding site, the depth of GWT reduces the overall vertical settlement at an approximate rate of 0.0175 cm/cm.

Similar to settlements, the depth of GWT can reduce the LSN and LPI values. But, the rate of change in the increment of LSN for a decreasing depth of the GWT is very high when the GWT exists within 2 m depth from the ground surface.

Figure 40. Overall Liquefaction Analysis Results of GEE001.

Figure 41, shows the CPT based SBT classification chart and liquefaction triggering curve of based on the liquefaction analysis for the CPT sounding site, GEE001. The centroid of the soils metrics in CPT based SBT classification chart in Figure 41, is 0.74% and 143.81. The normalized friction ratio has a range of 0.03 to 2.11%. The COV of the data points is 0.38. Similarly, for the normalized CPT penetration resistance, the range is 7.21 to 300.93. The COV of the resistance is 0.38. For the data points based on the upper 5 m of the soil column, the centroid of the chart has become 0.59% and 137.353, which indicates the shifting of datasets towards the third quadrant of a Cartesian coordinate system, and signifies liquefaction reduction of OCR, age, and cementation and shifting towards higher sensitivity of soils in region 6. The average values of the normalized friction ratio and CPT penetration resistance for the lower 15 m of the soil are 0.79% and 145, which impart a major shift towards the first quadrant of the classification chart. So, the soil layers below five meters have a major influence on the overall probability of liquefaction of the location. In the liquefaction triggering curve in Figure 41, the coordinates of the centroid of the data points are 156.444 and 0.61, which indicate major parts of the soils are prone to liquefaction. The average values of the equivalent clean-sand value of the normalized cone penetration resistance and modified cyclic stress ratio for the upper 5 m are 141.64 and 0.76, which impart to a higher risk of liquefaction. On the other hand, the coordinates of the centroid of the data points generated based on the soil layers below 5 m are 161.188 and 0.56; these points are also mostly in the liquefaction

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zone side of the liquefaction triggering curve. In summary, the total soil column is highly expected to trigger liquefaction.
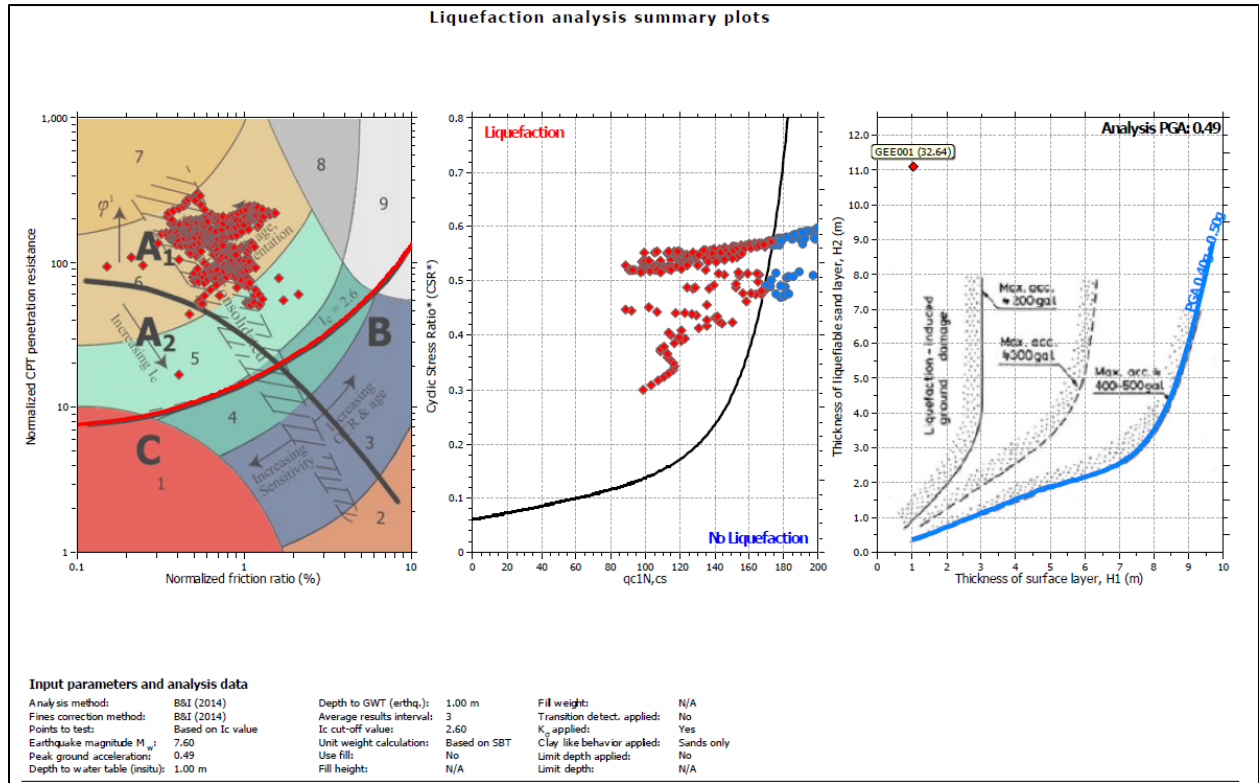
Figure 41. Summary Plots of the Liquefaction Analysis of Sounding GEE001.

GEE002

This CPT sounding is located at the northeast of the intersection of N. AR 139 Hwy and Green 839 Rd. The SBT analysis shows two thin silty sand and sandy silt layers, two clay, and silty clay layers, and one sand and silty sand layer within the top two meters of the soil column. All the other soils in the 20 m depth columns are sand and silty sand, except two very thin silty sand and sandy silt layers at depths of 3.7 m and 16.7 m. The total depth of the sounding is 16.8 m. The cone tip resistance is comparatively less up to a depth of 4.5 m, and it is almost consistent at a higher depth. The average I_c of the soil column is 1.74. The soil layers below 2 m are sand and silty sand. The percent fine content (FC) of the upper 5 m of soils varies from 2.24% to 37.95%, with an average value of 9.5%, and the COV is 0.80. The FC in other layers below a depth of 5 meters varies from 2.14% to 17.48%, with an average value of 6.37% and a COV of 0.41. The unit weight of soils varies from 15.06 kN/m^3 to 21.58 kN/m^3. The average unit weight of the soil column is 19.3 kN/m^3,

and the COV of unit weight of the soils is 0.05 kN/m^3. A detailed depth-wise summary of the basic CPT data interpretation results is shown in Figure 42.
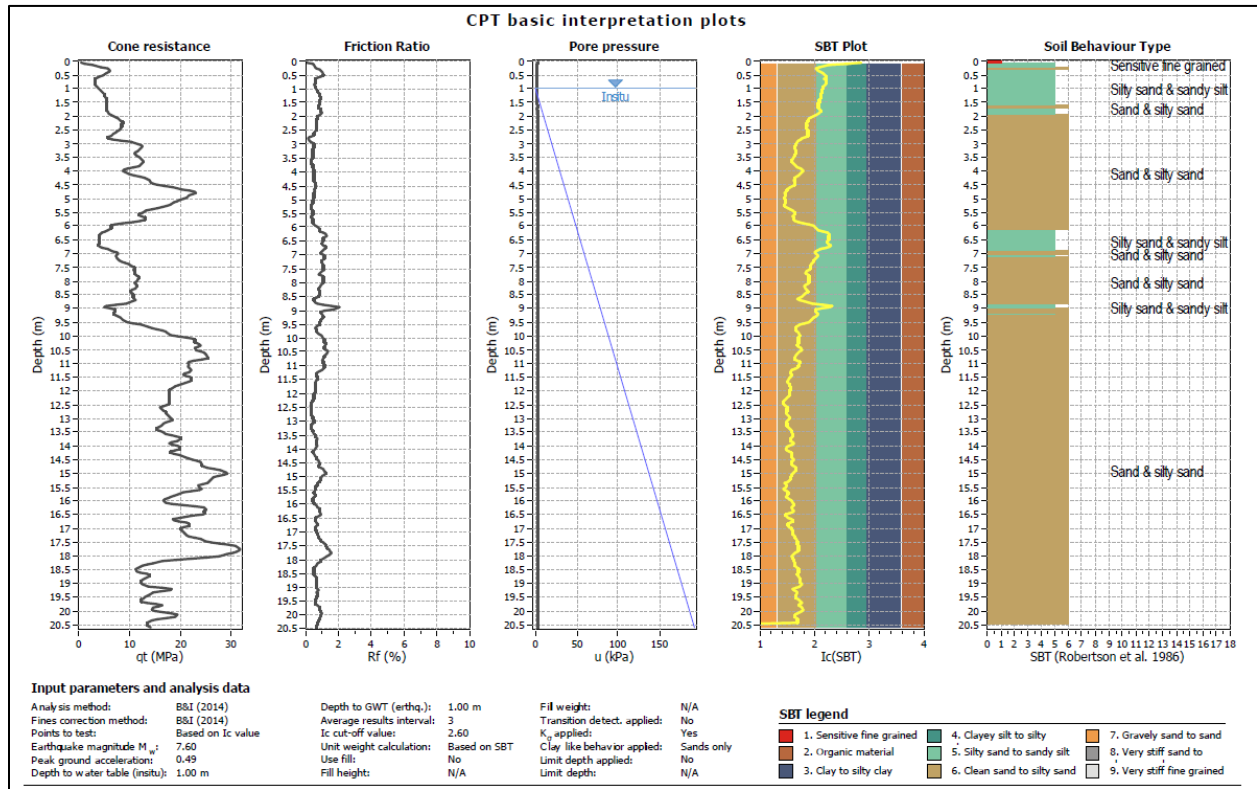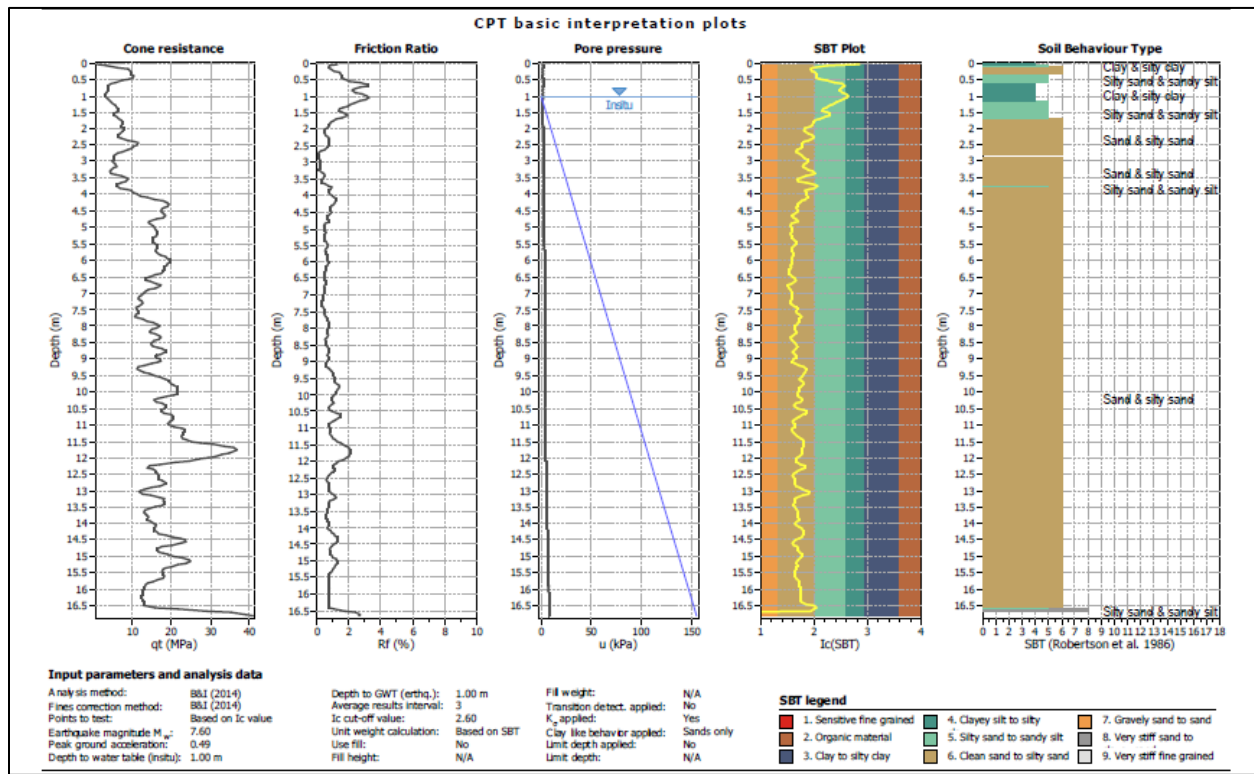

Figure 42. Basic Interpretation of CPT Data of GEE002.

According to the cyclic liquefaction analysis, the CSR is greater than CRR for four soil layers. For this location, the depth ranges of the soil layers with an FS less than 1 are from 0 to 4.0 m, from 6.5 to 9.5 m, and from 12 to 16.5 m. The overall LPI for the site is around 25.33, which indicates a very high risk of liquefaction. The LSN of this location is 46.808, which indicates a major expression of liquefaction in the upper layer of the soils. The ground surface of the site is prone to a vertical settlement of 21.94 cm and a lateral displacement of 147.87 cm after earthquakes. The overall probability of liquefaction of the site is 91.92%. A parametric analysis of the vertical settlement against input PGA shows that a PGA value greater than 0.95g will not have any impact on the overall vertical settlements. A significant portion of the settlements can occur for a ground motion with a PGA value of less than 0.45g. However, the LPI stays proportional to the PGAs. A ground motion with a PGA value greater than 0.32g is critical for this location, and it can cause "very high risk" of overall liquefaction. A PGA value above 0.65g does not affect the LSN anymore. However, a major risk of surface damage can show a major expression of liquefaction with the ground motion having a PGA greater than 0.35g. The depth-wise distribution liquefiable layers and different scenarios of liquefaction potential, probability of liquefaction, vertical settlements, and lateral displacements are shown in Figure 43. For this sounding site, the depth of GWT reduces overall vertical settlement at an approximate rate of 0.015 cm/cm. Similar to settlements, the depth of GWT can reduce the LSN and LPI values. But, the rate of change in the increment of LSN for a decreasing depth of the GWT is very high when the GW exists within 4 m depth from the ground surface.
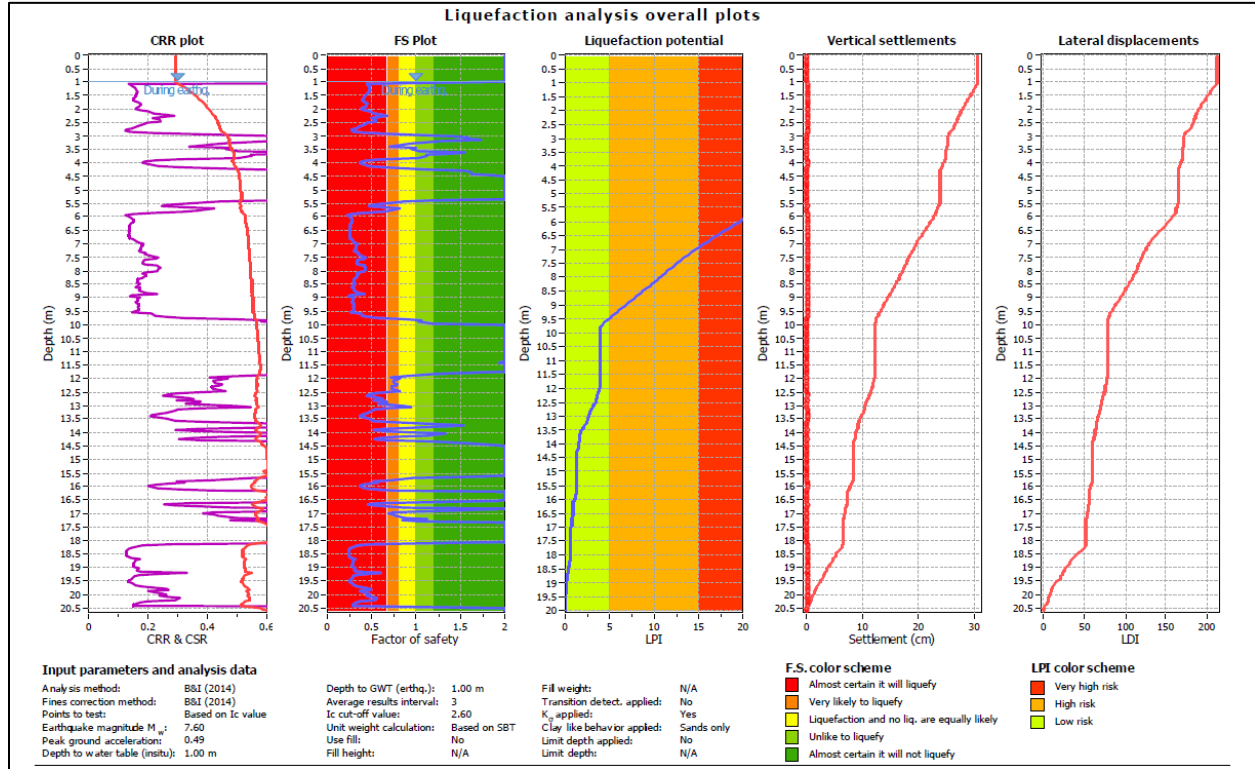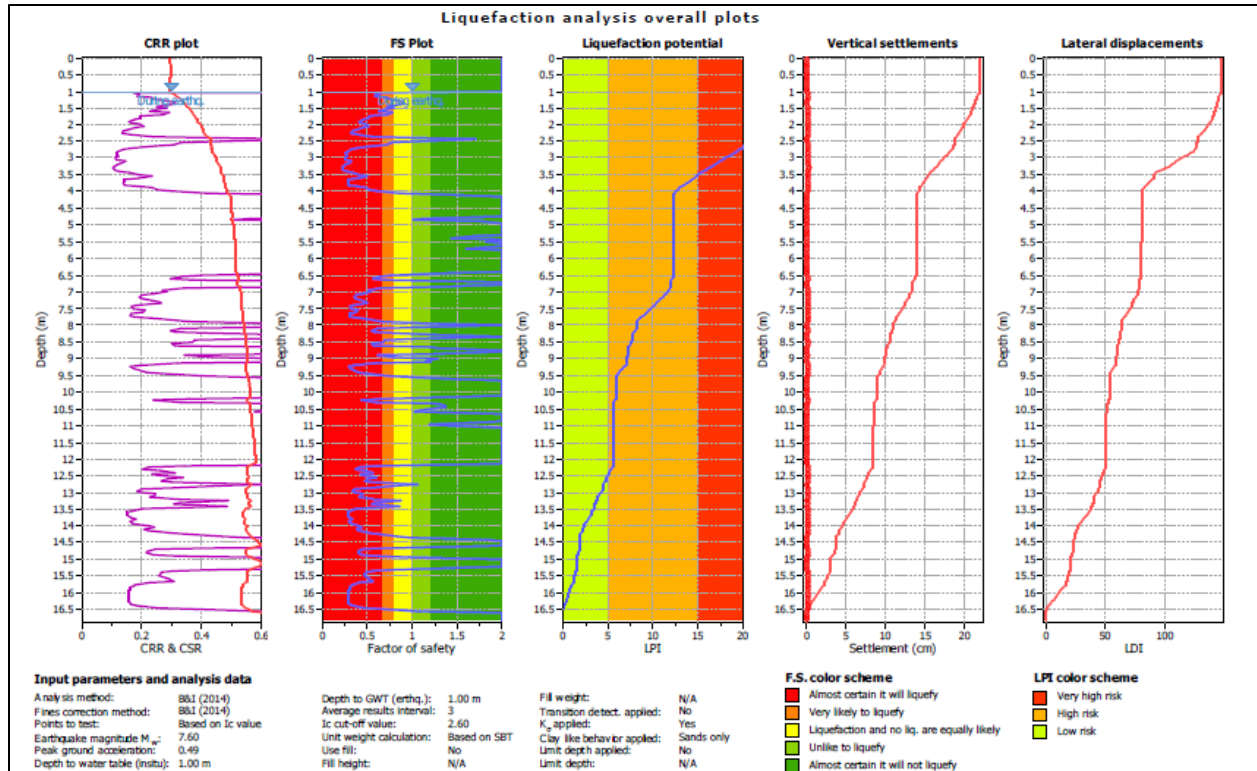
Figure 43. Overall Liquefaction Analysis Results of GEE002.

Figure 44 shows the summary of the liquefaction analysis for the CPT sounding site GEE002. The coordinates of the centroid of the soil's metrics in the CPT classification chart in Figure 52, is 0.93% and 155.365. The normalized friction ratio has a range of 0.07 to 3.27%. The COV of the normalized friction ratio points is 0.61. Similarly, the CPT penetration resistance varies from 14.76 to 328.2. The COV of the resistance is 0.35. For the upper 5 m of the soil column, the coordinates of the centroid of the chart become 1.06 and 131.71, which indicates a shift of datasets towards the fourth quadrant of the chart. The average values of the normalized friction ratio and CPT penetration resistance for the lower 11.5 m of the soil are 0.87 % and 165.52, which contributes to a major shift of the centroid towards Region 6 of the classification chart. So, the soil layers below five meters have a major influence on the overall probability of liquefaction of this location. In the liquefaction triggering curve (Figure 44), the coordinates of the centroid of the data points are 165.75 and 0.61. The average values of data points for the upper 5 m are 141.76 and 0.75, which indicates all the data points are triggers liquefaction. On the other hand, the coordinates of the centroid of the data points of the soil layers below 5 m are 173.06 and 0.55; these points are located at both sides of the curves. Further analysis of the equivalent clean-sand value of the normalized cone penetration resistance and modified cyclic stress ratio for the upper 10 m and the bottom 6.5 m of the soil column show the coordinates of the centroids of the datasets are: 157.082 and 0.64, and 173.54 and 0.56.
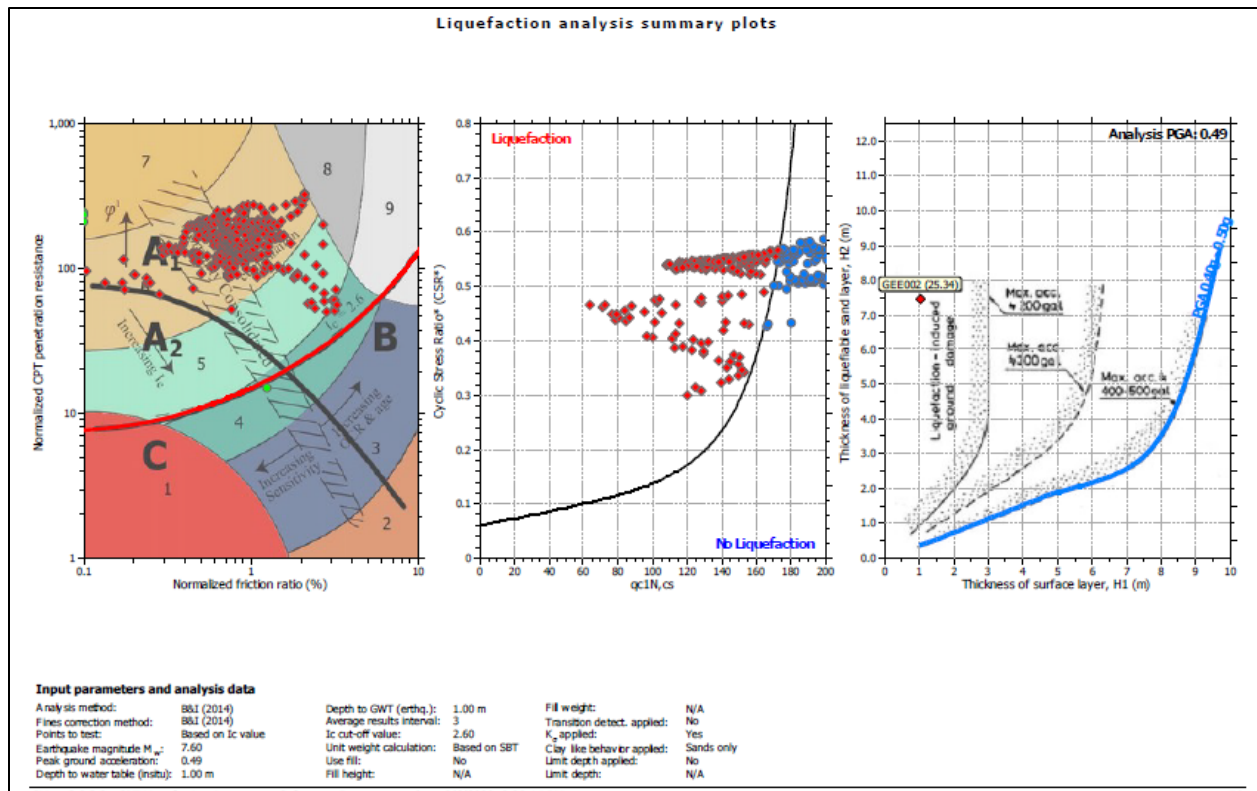
Figure 44. Summary Plots of the Liquefaction Analysis of Sounding GEE002.

GEE003: This site is located inside Green County but further northeast of GEE004. The location of the sounding is near the intersection of Green County Rd. 936 and Hwy. US 412 E. The CPT data interpretation shows three different layers (clay, silty sand, and sandy silt, and clay and silty clay) that exist within the topmost 2.5 m of the soil column. All the other soils in the 20m depth column are sand and silty sand, which is dilative. The cone resistance increases with the increase of depth. The average I_c of the total soil column is 1.28. For the upper 5 m, the Ic value is 1.85 with a range from 1.35 to 2.76 and a standard deviation of 0.34. The soil layers below 2.2 m are sand and silty sand. The percent FC of the topmost 5 m of soils varies from 1 to 44%. The FC in other layers below a depth of 5 m varies from 1.1 to 100%, with an average value of 6.37%. The unit weight of soils varies from 13.73 kN/m^3 to 21.53 kN/m^3. The average unit weight of the soil column is 19.47 kN/m^3 with a COV of 0.05 kN/m^3. A detailed depth-wise summary of the basic CPT data interpretation results is shown in Figure 45.

CPT basic interpretation plots

| Cone resistance | Friction Ratio | Pore pressure | SBT Plot | Soil Behaviour Type |

Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	1.00 m	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	Kₒ applied:	Yes
Earthquake magnitude M_w:	7.60	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.49	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	1.00 m	Fill height:	N/A	Limit depth:	N/A

SBT legend

1. Sensitive fine grained	4. Clayey silt to silty	7. Gravely sand to sand
2. Organic material	5. Silty sand to sandy silt	8. Very stiff sand to
3. Clay to silty clay	6. Clean sand to silty sand	9. Very stiff fine grained

Figure 45. Basic Interpretation of CPT Data of GEE003.

Four major clusters of liquefiable soil layers are noticed at depths from 1 to 4 m, from 12 to 15 m, from 16 to 16.5 m, and from 19.5 to 20 m. According to the cyclic liquefaction analysis, the CSR is greater than the CRR for all the four soil layers. The FS values of these four layers are less than 1.0. The overall LPI for the site is about 19.74, which indicates a "very high risk" of liquefaction. The LSN of this location is 40.174, which indicates potential severe damage to the upper layer of the soils due to liquefaction. The ground surface of the site is prone to a vertical settlement of 20.47 cm and a lateral displacement of 131.77 cm after an earthquake. The overall probability of liquefaction of the site is 77.039%. A parametric analysis of vertical settlements against input PGA shows that a PGA value greater than 0.8g does not have any major impact on the overall vertical settlements. A significant portion of the settlements can occur for a ground motion with a PGA value of less than 0.35g. However, the LPI increases at a slow rate with an increase of the PGA. The ground motion with a PGA greater than 0.4g is critical for this location, and it can cause a "very high risk" of liquefaction. In the case of LSN, a PGA value greater than 0.6g does not affect the LSN anymore. However, a ground motion with a PGA value greater than 0.45g can only show a major expression of liquefaction. Figure 46 shows the depth-wise distribution liquefiable layers and different scenarios of liquefaction potential, probability of liquefaction, vertical settlements, and lateral displacements. For GEE003, the GWT does not affect overall vertical settlement significantly below a depth of 4 m. However, the GWT above 4 m can cause an increment of the settlement for this location. On the other hand, a reduction of the GWT can significantly impact the LSN and LPI values. In the case of groundwater moving up towards the surface from a depth of 4m can increase the LSN by ten folds.
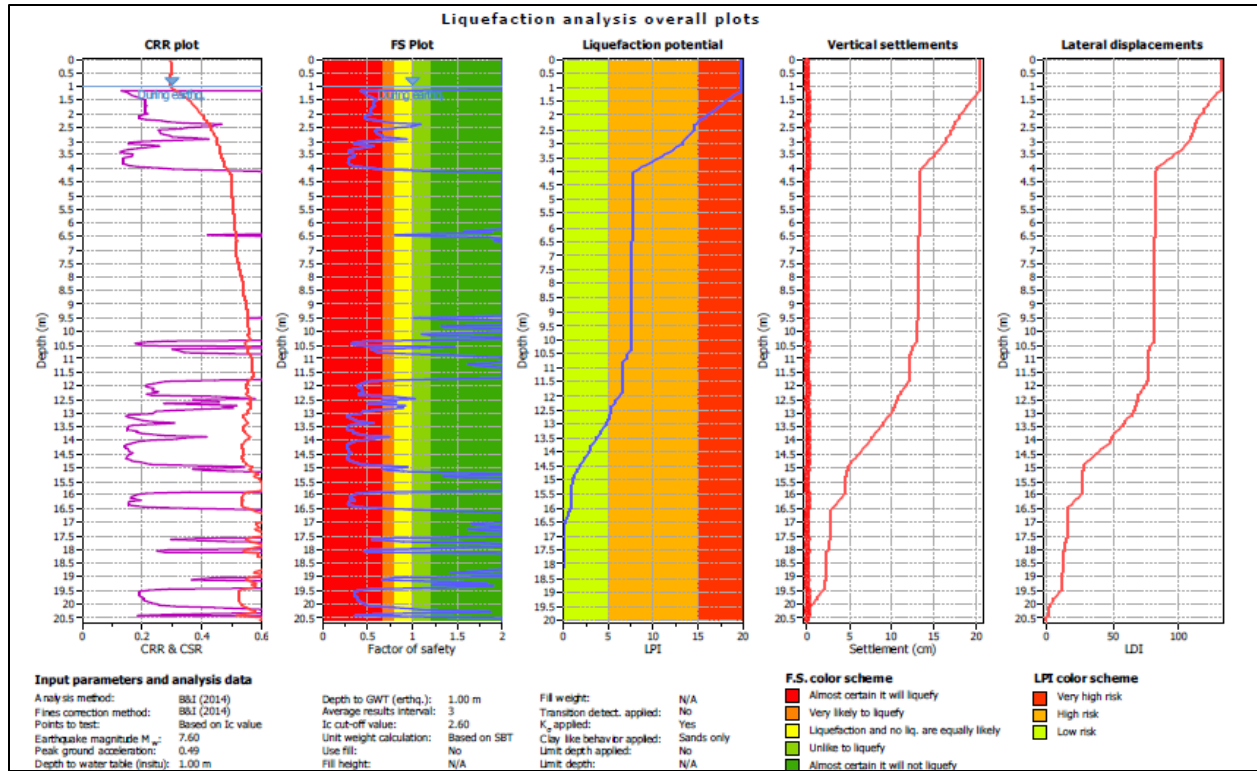
Figure 46. Overall Liquefaction Analysis Results of GEE003.

Figure 47 shows the CPT based SBT classification chart and liquefaction triggering curve based on the liquefaction analysis for the CPT sounding site GEE003. The coordinates of the centroid of the soils metrics in CPT based SBT classification chart is 0.86% and 167.542. The NFR ranges from 0.07 to 5.13% with a COV of 0.61%. On the other hand, the range of the normalized CPT penetration resistance is 27.5 to 318.24 with a COV 0.37. For the data points based on the upper 5 meters of the soil column, the coordinates of the centroid of the chart have become 1.2% and 141.11, which indicate a shift of datasets towards the fourth quadrant of the Cartesian coordinate systems. This signifies a liquefaction increment of OCR, age, and cementation and transition from a stiffer condition to a loose condition. The average values of the normalized friction ratio and CPT penetration resistance for the lower 15m of the soil are 0.75 % and 176, which impart a major shift towards the second quadrant of the classification chart. So, the soil layers below 5 m have a major influence on the overall probability of liquefaction of the location. In the liquefaction triggering curve (Figure 47), the coordinates of the centroid of the data points are 175.443 and 0.599. The average values of the equivalent clean-sand value of the normalized cone penetration resistance and modified cyclic stress ratio for the upper 5 m are 147.79 and 0.75, which imparts to a higher risk of liquefaction. On the other hand, the coordinates of the centroid of the data point the soil layers below 5 m are 184.3 and 0.55, these points are mostly at "no liquefaction zone" of the liquefaction triggering curve.

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Figure 47. Summary Plots of the Liquefaction Analysis of Sounding GEE003.

GEE004: GEE004 is a sounding location near the intersection of Hwy AR 135 S. and Green County Rd. 960. The Soil Behavior Type (SBT) analysis of the CPT sounding shows thin silty sand and sandy silt layer exists on top of subsequent clay and silty clay, silty sand, and sandy silt layers within the top-most 2.5 m. Soils below 2.5 m are sand and silty sand. The CPT resistance increases with the increase of depth, and the friction ratio is comparatively higher for the upper 2.5 m of the soil. The average I_c of the soil column is 1.74. The percent fine content (FC) of the topmost 5 m of soils varies from 3.32% to 44.82%, with an average FC of 15.86% and a COV of 0.77. The FC values below a depth of 5 m vary from 2% to 100%, with an average of 6% and a standard deviation of 9.5%. The unit weight of soils varies from 13.73 kN/m^3 to 21.58 kN/m^3. The average unit weight of the soil column is 19.47 kN/m^3 and a standard deviation of 1.13 kN/m^3. A detailed depth-wise summary of the CPT test results is shown in Figure 48.
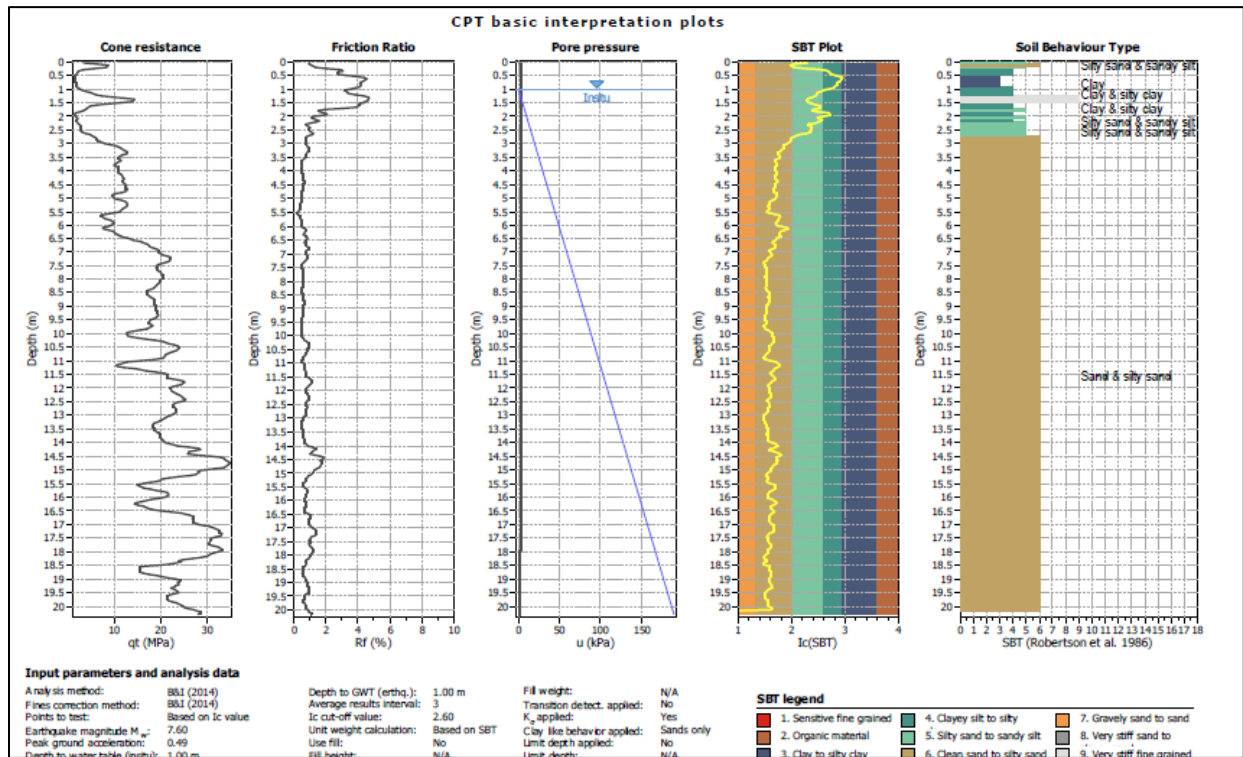
Figure 48. Basic Interpretation of CPT Data of GEE004.

For the GEE004 sounding location, the cyclic liquefaction analysis shows the CSR is greater than CRR in seven to ten different soil layers. The depth-wise locations of the soil layers with an FS of less than 1.0 are from 1.5 to 3.5 m, from 3.5 to 6.5 m, from 9.8 to 10.1 m, from 11 to 11.5 m, from 13.2 to 13.7 m, from xx to 15.5 m, from 16.0 to 16.5 m, from 18.5 to 19.0 m, and from 19.5 to 20.0 m. The overall LPI for the site is about 22.146, which indicates a very high risk of liquefaction. The LSN value at this location is 41.783, which indicates a major expression of liquefaction at the upper layer of the soils. The ground surface of this site is prone to a vertical settlement of 18.53 cm and a lateral displacement of 127.43cm after an earthquake. The overall probability of liquefaction of the site is 85.01%. A parametric analysis of vertical settlements against an input PGA shows that a PGA value greater than 0.8g does not have any notable impact on the overall vertical settlement. A significant portion of the settlement can occur for a ground motion with a PGA value of less than 0.40g. However, the LPI keeps increasing with the increment of PGAs, with a faster pace until 0.9g and a slower pace at a higher PGA value. Based on the rate of change of LPIs with the increase of the ground motion's PGA, a PGA greater than 0.35g is critical for this location and can result in a "very high risk" of liquefaction. In the case of LSN, which primarily indicates the probability of damage to the surfaces, a PGA above 0.6g does not impact the values of LSN significantly. However, a major expression of liquefaction can occur at ground surfaces in response to experiencing a ground motion with a PGA value greater than 0.45g. The depth wise distribution of liquefiable layers and different scenarios of liquefaction potential, probability of liquefaction, vertical settlements, and lateral displacements are shown in Figure 49. The GWT has a significant impact on different parameters related to liquefaction assessment. For this site, a rising GWT increases the vertical settlement significantly when the depth of the GWT

is less than 6.5 m. However, the groundwater level below 6.5 m is not expected to impact the settlement significantly for this location. On the other hand, an increase in the GWT can significantly impact the LSN, unlike the LPI. In the case of groundwater moving up to the surface within a depth of 1 m can increase the LSN by two times.
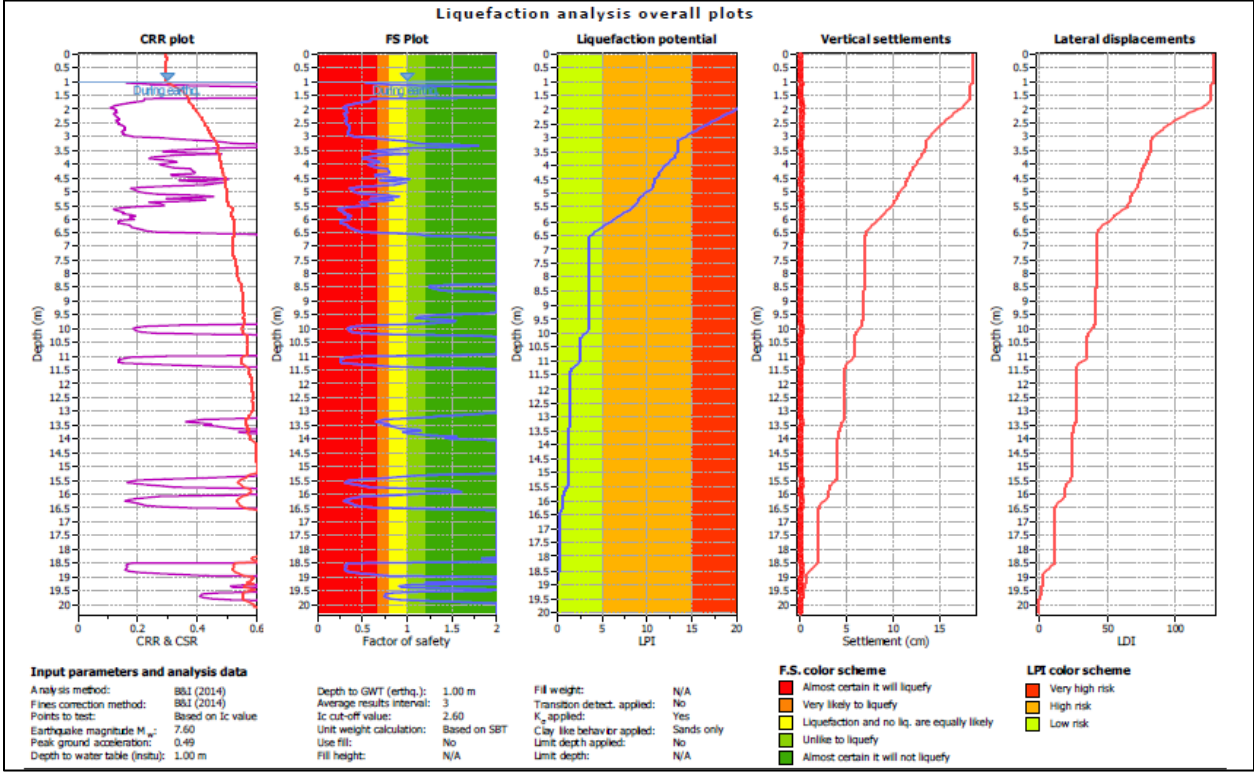
Figure 49. Overall Liquefaction Analysis Results of GEE004.

Figure 49 shows a summary of the liquefaction analysis for the CPT sounding site GEE004. The normalized friction ratio (NFR) has a range of 0.21 to 4.7% with a COV of 0.47%. On the other hand, the normalized CPT penetration resistance ranges from 25 to 272. With a COV of 0.34. The coordinates (NFR and normalized CPT resistance) of the centroid of the soils metrics in the CPT classification chart (CCC) is 1.01% and 163.9 when data points only for the upper 5 m of the soil column is considered, the coordinates of the centroid of the chart become 1.73% and 112.09. These values indicate a shifting of datasets towards the S-W direction of the plot that means all the scattered points at the right side of the classification charts are the data of the upper 5 m. The average values of the NFR and CPT penetration resistance for the lower 15 m of the soil are 0.76% and 181, which indicate a major portion of data points near the normally consolidated soil zone of the upper left part of the centroid of the classification chart. So, the soil layers below 5 m have a major influence on the overall probability of liquefaction for this location.

In the liquefaction triggering curve (Figure 50), the coordinates (q_c and CSR) of the centroid of the data points are 176.56 and 0.60. The average values of data points for the upper 5 m are 130.61 and 0.75, which indicate all the points are susceptible to liquefaction. On the other hand, the coordinates of the centroid of the data points of the soil layers below 5 m are 191.6 and 0.56, these points are mostly in no liquefaction zone of the liquefaction triggering graph.
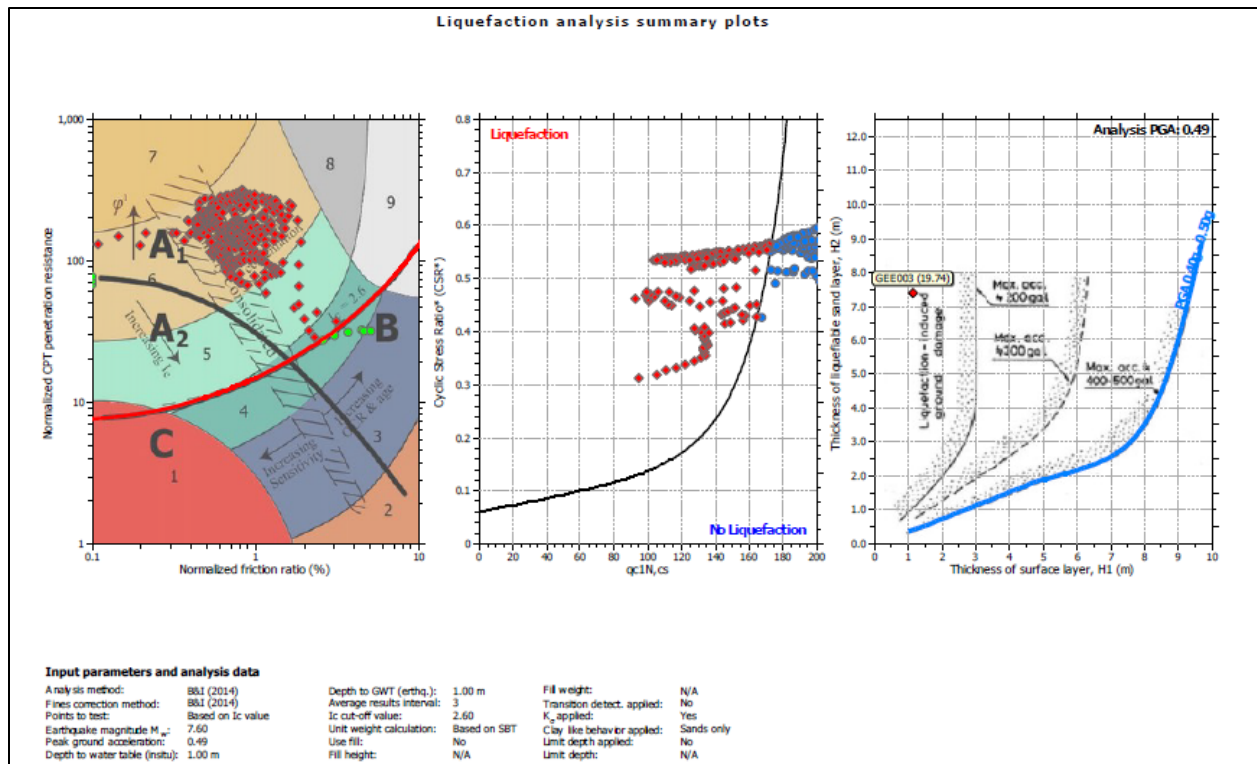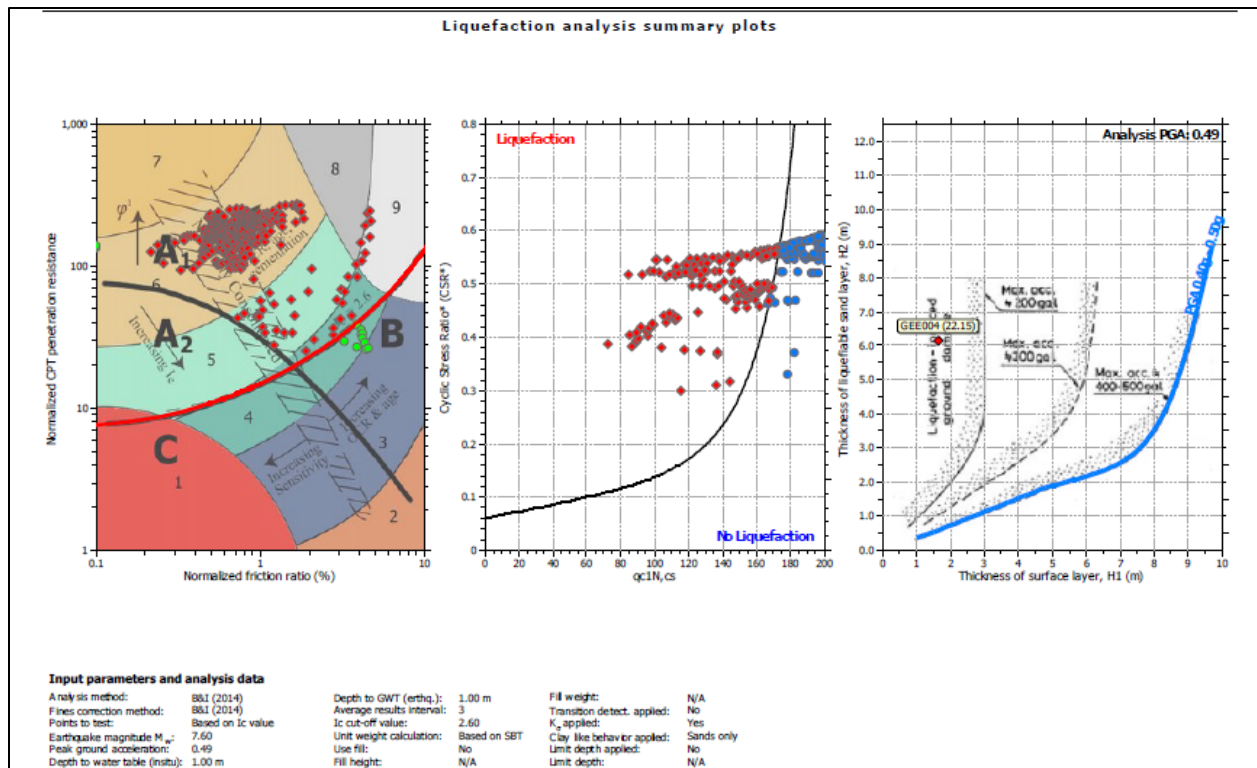
Figure 50. Summary Plots of the Liquefaction Analysis of Sounding GEE004.

CLY001

Clay County is located in the northeast corner of the state of Arkansas. Only one CPT sounding data is available for a location near the intersection of Clay 508 Rd and Clay 515 Rd. CPT data interpretation shows five thin silty sand and sandy silt layers that exist within the top-most 3 m of the soil column. All the other soils in the 20 m depth columns are sand and silty sand. The cone resistance increases with the increase of depth. The average I_c of the soil column is 1.28. The soil layers below 3 m are sand and silty sand. The percent fine content (FC) of the topmost 5 meters of soils varies from 0.7 to 22%. The FC in other layers below the depth of 5 m approximately varies from 0.2 to 100%, with an average of 5%. The unit weight of soils varies from 13.73 kN/m^3 to 21.58 kN/m^3. The average unit weight of the soil column is 19 kN/m^3, and the standard deviation

of the unit weight of the soils is 1.72 kN/m³. A detailed depth-wise summary of the basic CPT data interpretation results is shown in Figure 51.
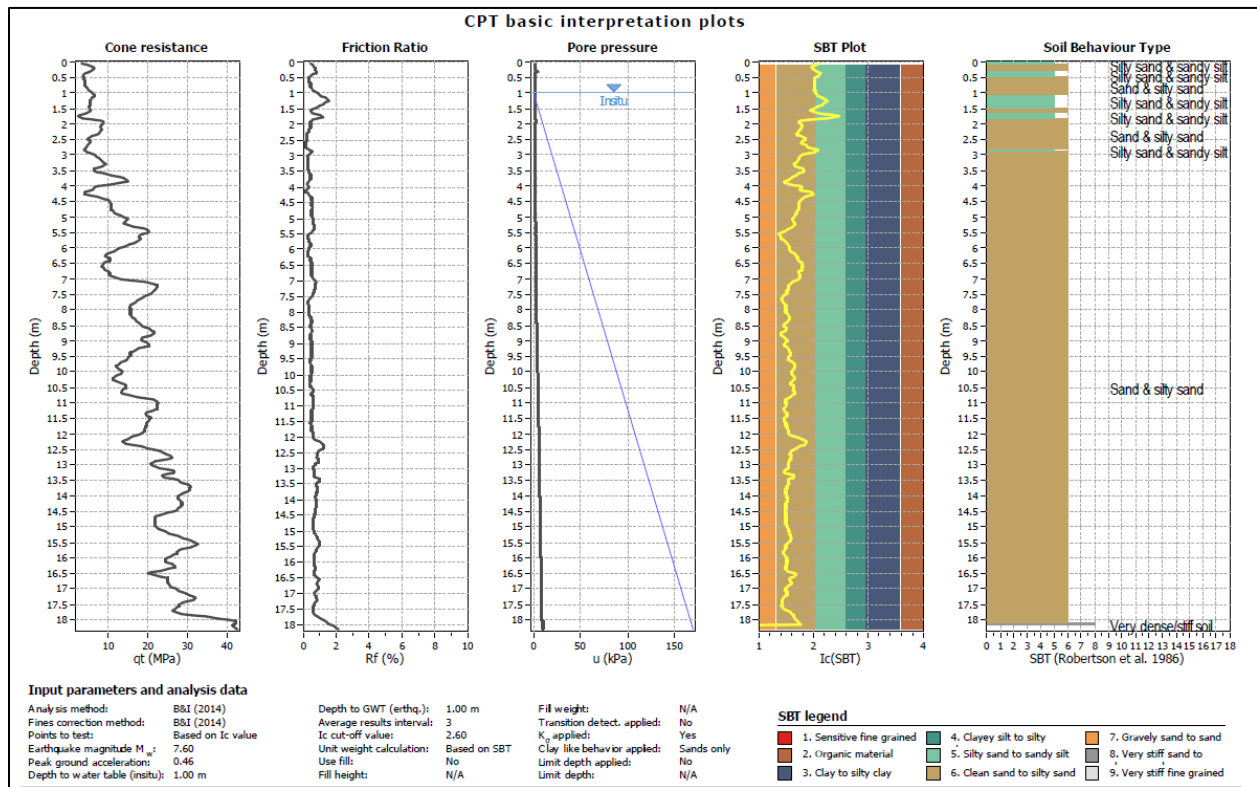
Figure 51. SBT Analysis of CLY001.

According to the cyclic liquefaction analysis, the CSR is greater than CRR for four soil layers. Depth ranges of the soil layers with FS less than 01 are 1 to 3.5 m, 4 to 5 m, 6 to 7 m, and 9.5 to 11 m. Overall LPI for the site is around 25.31, which indicates a very high risk of liquefaction. LSN of location is 50.352, which indicates severe damage to the upper layer of the soils. For the given criteria, the ground surface of the site is prone to a vertical settlement of 17.83 cm and a lateral displacement of 135.59 cm after earthquakes. The overall probability of liquefaction of the site is 91.87%. Parametric analysis of vertical settlements against the input PGA values shows that the PGA values greater than 0.6g do not have any future impact on the overall vertical settlements. A significant portion of the settlements can occur for a ground motion with the PGA values less than 0.30g. However, the LPI keeps increasing with the increment of PGAs, with a faster pace at the beginning part, and a slower pace at the last moments. For ground motion with the PGA values greater than 0.3g is critical for this location and can cause "very high risk" of overall liquefaction. In the case of LSN, which primarily indicates the probability of damage to the surfaces, the PGA values above 0.6g does not affect the LSN anymore. However, a major risk of surface damage can show a major expression of liquefaction with the ground motion having PGA values greater than 0.27g. The depth-wise distribution liquefiable layers and different scenarios of liquefaction potential, probability of liquefaction, vertical settlements, and lateral displacements are shown in Figure 52. GWT has a significant impact on the different parameters related to liquefaction assessment. For CLY001, GWT does not affect overall vertical settlement significantly. However, the groundwater level below 5 m can significantly reduce the settlement for this location. On the

other hand, raising GWT can significantly impact the LSN and LPI values, In the case of groundwater moving up to the surface from a depth of 1 m, the LSN increased by four-fold.
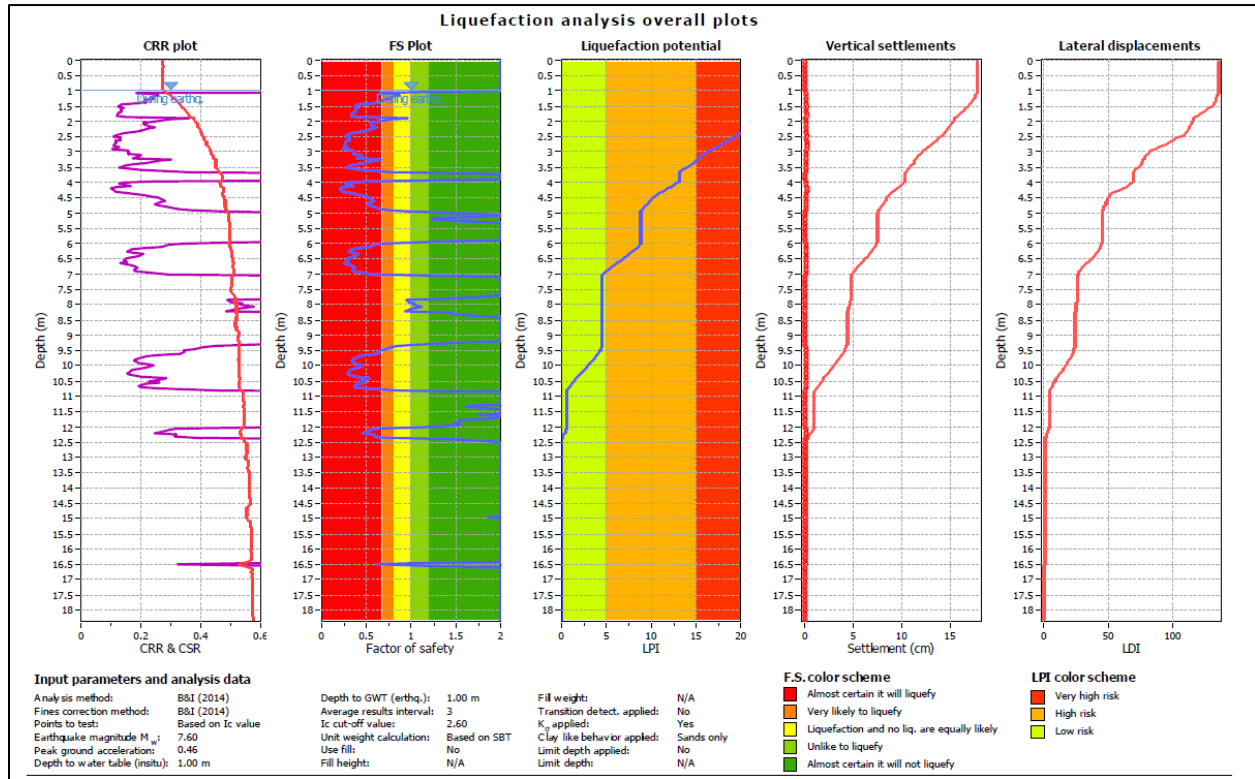
Figure 52. Liquefaction Analysis of CLY001.

Figure 53 shows the summary of the liquefaction analysis for the CPT sounding site CLY001. The centroid of the soil matrix in the CPT classification chart in Figure 53 is 0.6% and 171.1. The normalized friction ratio has a range of 0.13 to 2.17%. The COV of the data points is 0.47. Similarly, for the CPT penetration resistance, the range is 40 to 291. The COV of the resistance is 0.32. For the data points based on the upper 5 m of the soil column, the centroid of the chart becomes 0.5 and 111.2, which indicates the shifting of datasets towards the third quadrants, which reduces the prospect of liquefaction. The average values of the normalized friction ratio and CPT penetration resistance for the lower 15 m of the soil is 0.64 % and 193.85, which imparting in the major shifting of the centroid of the classification chart. So, the soil layers below five meters have a major influence on the overall probability of liquefaction of the location. In the liquefaction triggering relationship graph in Figure 53, the coordinates of the centroid of the data points are 176.57 and 0.593. The average values of data points for the upper 5m are 116.79 and 0.74, which indicates all the points are susceptible to liquefaction. On the other hand, the centroid of the data points generated based on the soil layers below 5 m are 199 and 0.54, these points are in no liquefaction zone of the liquefaction triggering graph.

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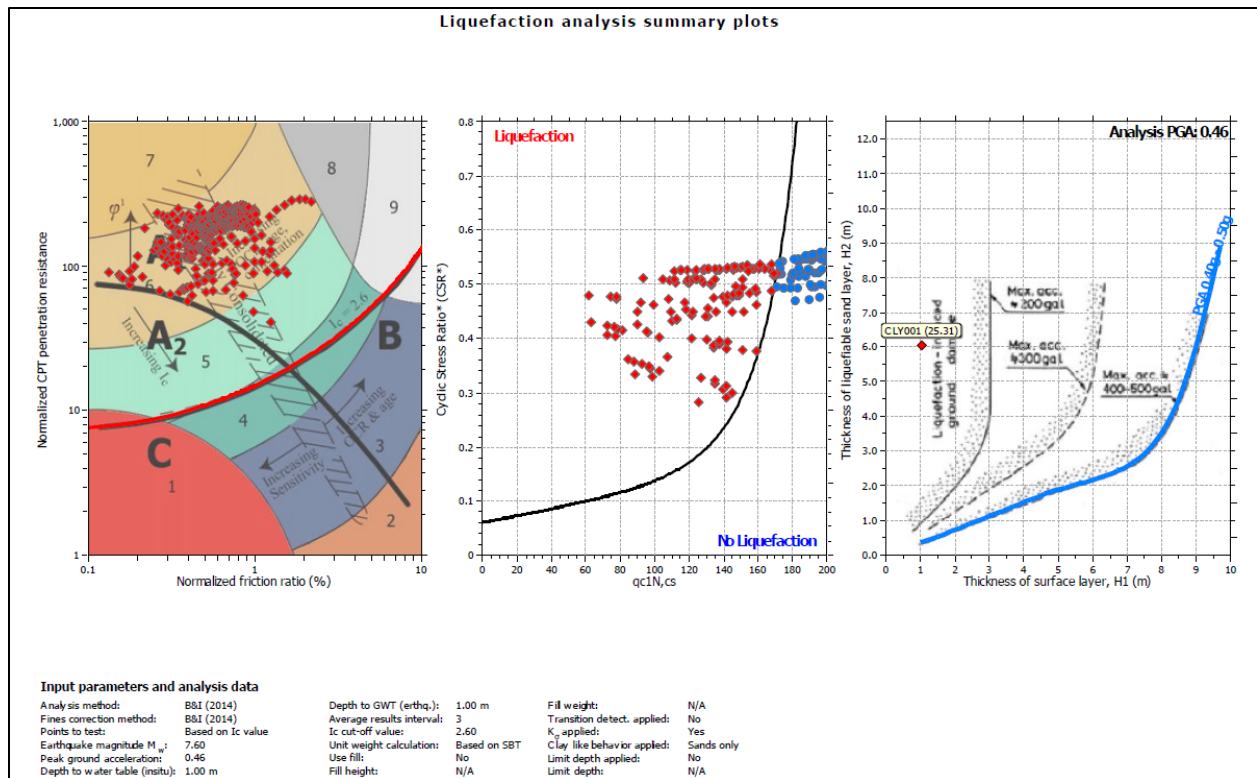
**Figure 53. Summary of LP Analysis of CLY001.**

After comparing the LPI values of the five sites at Clay and Green County, it can be concluded that with the progression of CPT sounding locations toward the northeast direction the LPI values of GEE004, GEE003, GEE002, GE001, and CLY001 soundings are 22.146, 19.736, 25.34, 32.64 and 25.31. The values increased at the beginning and then dropped after crossing the boundary of Green County, and all five sites have a very high risk of liquefaction. The LSN values in a similar direction also followed a similar pattern. The values of LSN for a similar order of sounding locations are 41.783, 40.174, 46.81, 55.63, and 50.35. Sixty percent of the sounding points have major liquefaction risk, and forty percent of the locations have severe liquefaction risks. The vertical settlement and horizontal displacement also increased while progressing towards the northeast direction, and then dropped at the location in Clay County. The overall probability of liquefaction is low for the GE003 site, but for all other four locations, the values increased with the progression towards the northeast direction from Green County, and then started to drop once the location fall in Clay County.

The overlay of all five sites CPT data shows several common scenarios applicable along the line connecting the CPT sounding locations. The average cone resistance of less in the upper 4 m than lower 16 m of the soil columns. However, the depth range of 4 to 12 m has the maximum variability in cone resistance. The average SBT Index, Ic, for all the sites are less than 2, however for the upper 3 m of the mantle, Ic lies within 2 to 2.6. In terms of settlements, the GEE001 site is more sensitive to PGA values. With the increment of PGA, the site can experience

In this analysis, a method developed by Boulanger and Idriss (2014) *(34)* has been adopted for the wider acceptability of the method among the practitioner. However, there are several cases where other methods are suitable as well. A comparative analysis of the estimated values of overall LPI,

LSN, vertical settlement, horizontal displacement, and overall probability shows very consistent results with the estimated values using other available methods. In the case of LPI estimation, the value of overall LPI is smaller than the estimated value using the method adopted by the same researcher's previously published method from 2008, only – which signifies the usage of the new method. The estimated values are also consistent with the values estimated using methods developed my Robertson *(50)*.

Craighead County

In total, twelve CPT bore logs were found within Craighead County. The LPI values of all 12 sites are greater than 15, which indicates a very high risk of liquefaction. All twelve sites have CPT soundings results up to the depth of 20 m. All the sites were analyzed for assigned peak seismic ground acceleration coefficient modified by a short-site factor from the nearest seismic site study sites.

CGD001

CGD001 is located at the intersection of the AR 148 Hwy E and highway 139. Based on the interpretation of the CPT cone resistance, and friction ratios, the top most soils layers are mostly of clay and silty clay with sand and silty sands. The soil layers below 5 m are all sand and silty sand. The percent fine content (FC) of the topmost 2.5 m of soils varies approximately 10 to 50 percent. FC in other layers within the depth of 5 m approximately varies between 5 to 12 percent. In the sandy soil layers below 5 m have percent FC values around 2 to 5 percent. The unit weight of soils varies from 13.73 kN/m^3 to 21.5 kN/m^3. The soil layer with the highest unit weight lies at

the depth of 15 to 16 m. The average I_c of the soil layers is 1.78. A detailed depth-wise summary of the basic CPT data interpretation results is shown in Figure 54.

Figure 54. Basic Interpretation of CPT Data of CGD001.

Figure 55 shows the CPT based SBT classification chart and liquefaction triggering curve of based on the liquefaction analysis for the CPT sounding site, CGD001. The centroid of the soils metrics in CPT based SBT classification is 0.98% and 170.24. The normalized friction ratio has a range of 0.17 to 5.73%. The COV of the data points is 0.56. Similarly, for the normalized CPT penetration resistance, the range is 16.93 to 345.2. The COV of the resistance is 0.49. For the data points based on the upper 5 m of the soil column, the coordinates of the centroid of the chart have become 1.19% and 59.68, which indicates the shifting of datasets towards the fourth quadrants of a Cartesian coordinate system, which signifies the characteristics upper layer soils. The upper layer soils are comparatively less sensitive and more of clay to silty clay. The green points in the SBT classification charts belong to the upper portion of the soils. The average values of the normalized friction ratio and CPT penetration resistance for the lower 15 meters of the soil are 0.91 % and 206.13, which imparting in the major shifting towards the second quadrant of the on the classification chart. In the liquefaction triggering curve in Figure 55, the centroid of the data points is 180.61 and 0.77, which indicates a significant portion of the soils are not prone to liquefaction. The average values of the equivalent clean-sand value of the normalized cone penetration resistance and modified cyclic stress ratio for the upper 5 m are 92.97 and 0.86, which imparts to a higher risk of liquefaction. On the other hand, the centroid of the data points generated based on the soil layers below 5 m are 208.8 and 0.74, these points mostly lie on the no liquefaction side of the liquefaction triggering curve. So, the bottom layers are not susceptible to higher liquefaction, whereas the upper portion is highly susceptible.

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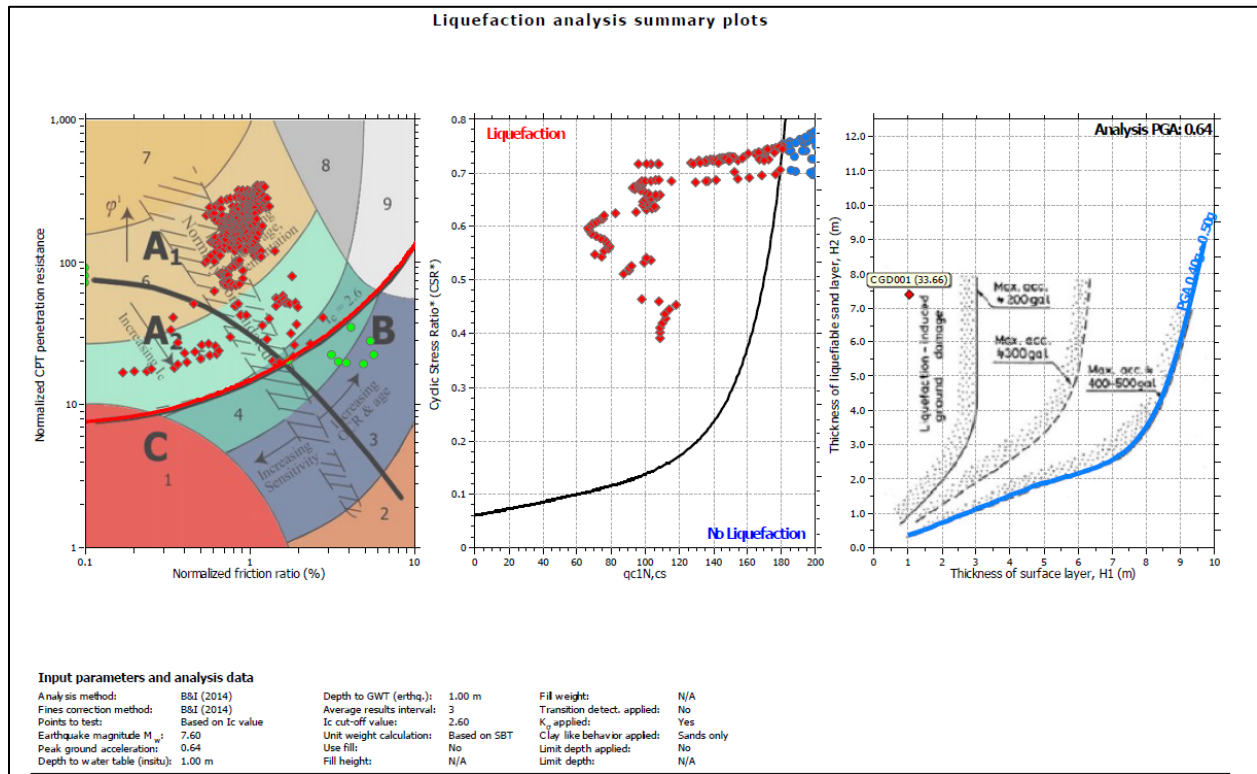
Figure 55. Summary Plots of the Liquefaction Analysis of Sounding CGD001.

A detailed graph of the summary of the cyclic liquefaction analysis is shown in Figure 56, below. According to the analysis, the CSR is greater than CRR for the soil layers of depth up 6 m. There are several other layers around a depth of 10 m, 12 m, 13 m, 14 m, and 15 m, which are also prone to liquefaction. The FS values against liquefaction for layers with the depth range of 1.05 to 1.50m, 1.9 to 5.5 m, 9.35 to 10.20 m, 11.40 to 12.35 m, 12.75 to 13.95 m, and 15.05 to 15.50 m are less than 1. The overall LPI for the site is around 33.658, and the site is prone to a vertical settlement of 22 cm and a lateral displacement index of 190 cm after earthquakes.
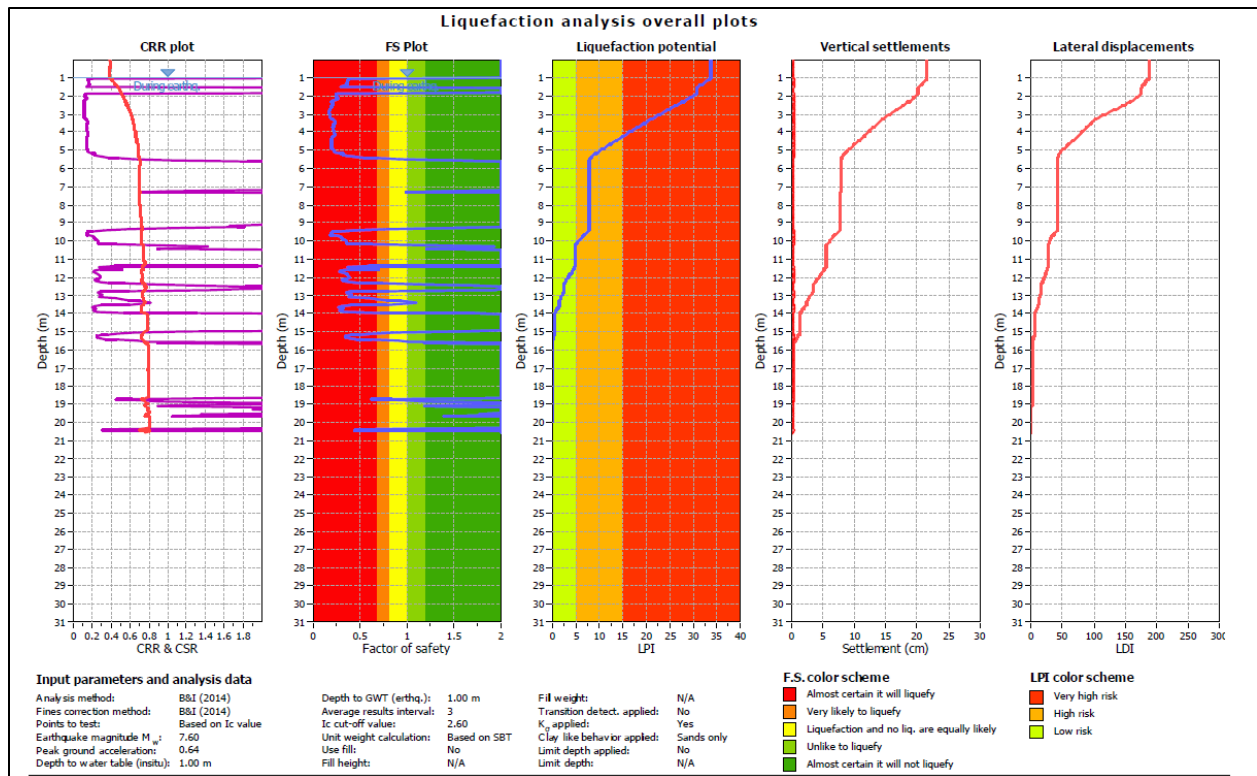
Figure 56. Overall Liquefaction Analysis Results of CGD001.

CGD003

This CPT sounding location is located on County Road 515, beside Saint Francis River, near Macey at Craighead County. The site is located on the eastern bank of the Saint Francis River. Based on the interpretation of the CPT cone resistance at CPT sounding site, CGD003, and friction ratios, the top most soils layers are mostly of clay, clay and silty clay, silty sand, and sandy silt, and sand and silty sand. The soil layers below 4 m from the ground level are all sand and silty sand. Percent fine contents of the topmost five meters of soils vary approximately 2.3 to 45.33 percent. However, from the depth of 15.5 m to 20 m, percent FC in the sand is comparatively higher. The unit weight of soils varies from 15.89 kN/m^3 to 20.91 kN/m^3 with an average of 19.36

kN/m^3. The soil layer with the highest unit weight lies at the depth of 9.5 m from the ground. A detailed depth-wise summary of the basic CPT data interpretation results is shown in Figure 57.
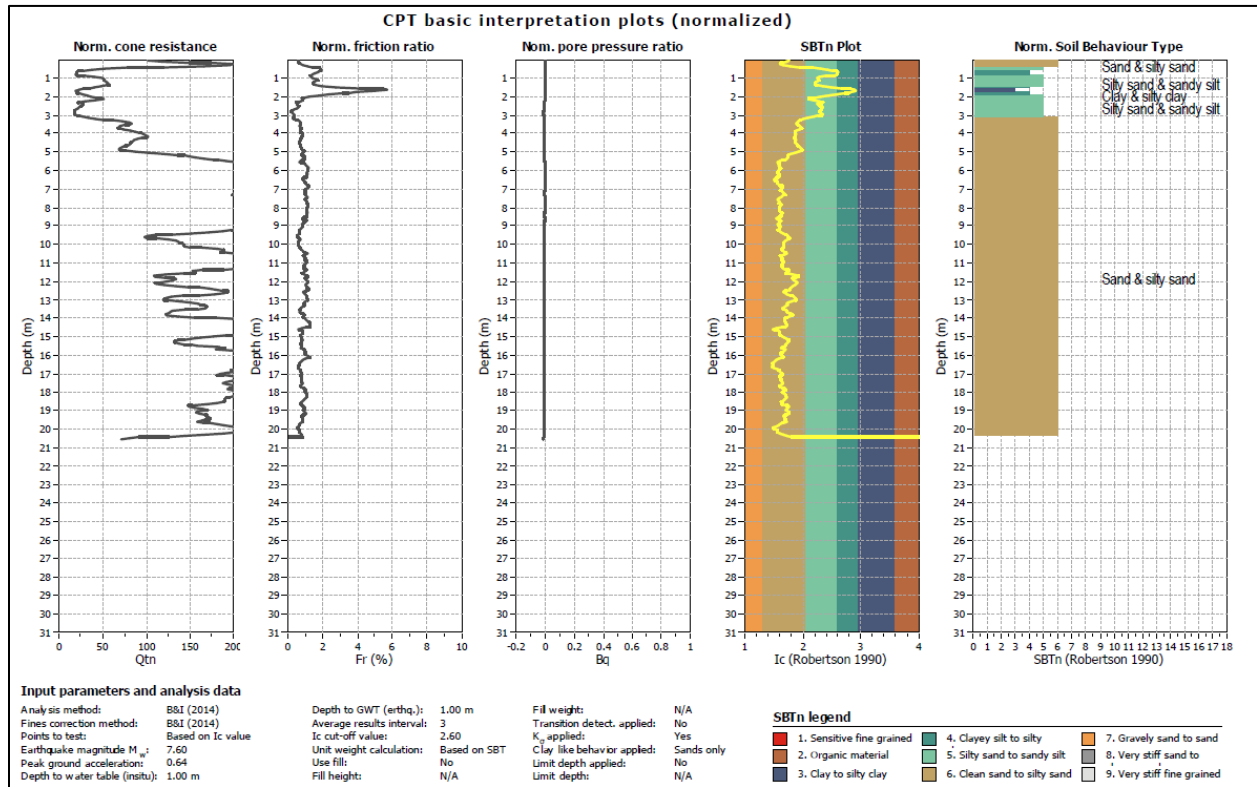

Figure 57. Basic Interpretation of CPT Data of CGD003.

Figure 58 shows the CPT based SBT classification chart and liquefaction triggering curve of based on the liquefaction analysis for the CPT sounding site, CGD003. The centroid of the soils metrics in CPT based SBT classification is 1.305% and 137.05. The normalized friction ratio has a range of 0.53 to 6.03%. The COV of the data points is 0.79. Similarly, for the normalized CPT penetration resistance, the range is 17.21 to 295.52. The COV of the resistance is 0.40. For the data points based on the upper 5 m of the soil column, the centroid of the chart has become 1.75% and 91.23, which indicates the shifting of datasets towards the fourth quadrants, Zone B, of a Cartesian coordinate system and signifies increasing OCR, age and decreasing sensitivity of soils. Average values of the normalized friction ratio and CPT penetration resistance for the lower 15 m of the soil are 0.82 % and 151.97, which imparting in the major shifting towards the second quadrant, zone A$_1$, of the classification chart. In the liquefaction triggering curve (Figure 57), the coordinates of the centroid of the data points are 150.26 and 0.75, which indicate major parts of the soils are prone to liquefaction. The average values of the equivalent clean-sand value of the normalized cone penetration resistance and modified cyclic stress ratio for the upper 5 m are 110.9 and 0.85, which implies almost all five meters of the soils will trigger liquefaction. The centroid of the data points generated based on the soil layers below 5 m are 162.964 and 0.72, these points are also mostly in the liquefaction zone side of the liquefaction triggering curve. So, the total soil column triggers liquefaction in this case.
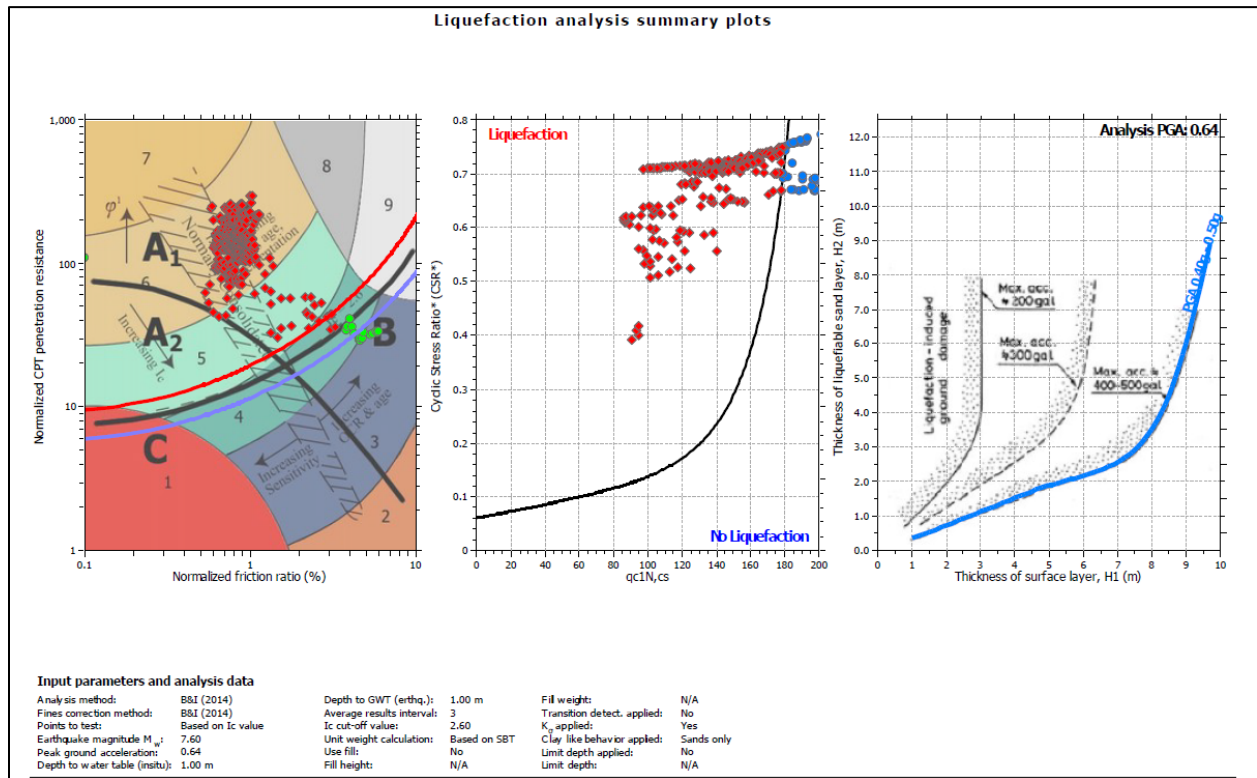
Figure 58. Summary Plots of the Liquefaction Analysis of Sounding CGD003.

The results of the cyclic liquefaction analysis for different soil layers of the site is shown in Figure 59, below. According to the analysis, the CSR is greater than CRR for almost all the soil layers of depth up to 20 m, except layers at a depth of 6 to 7 m, 14 to 15.5 m. The overall LPI for the site is around 39.68, which is very high in comparison with the other locations within the county. For the assumed analysis criteria, the ground surface of the site is prone to a vertical settlement of 32.96 cm and a lateral displacement of 217.01 cm after earthquakes. Figure 59 shows the summary of the liquefaction analysis for the CPT sounding site CGD003.
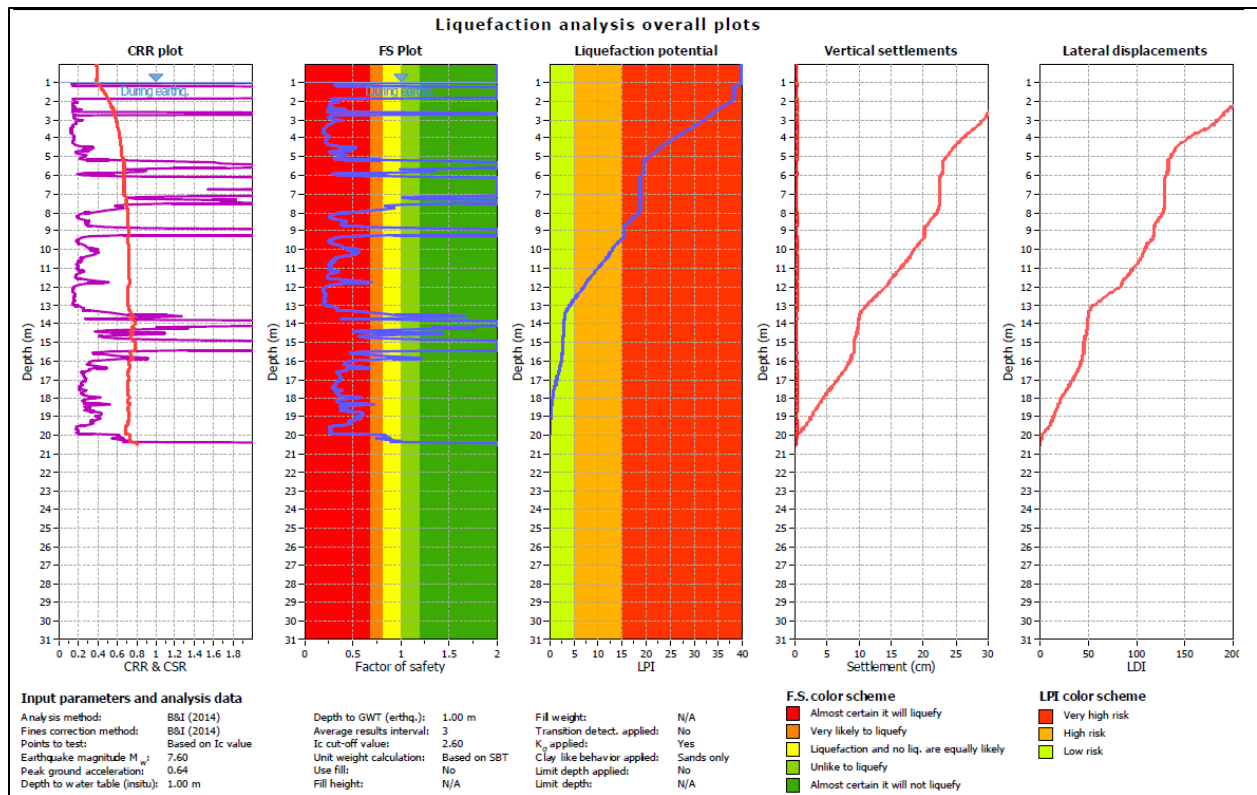
Figure 59. Overall Liquefaction Analysis Results of CGD003.

CGD005

This CPT Sounding is located near Macey, and in between highway AR-139 N and County Road 518. The SBT analysis shows several layers of clay, clay and silty clay, silty sand and sandy silt layers, one thin layer of silty sand and another layer of sandy silt layer exist within the top-most five meters. All the other soils in the 20 m depth columns are sand and silty sand, except two very thin silty sand and sandy silt layers near the depth of 11 m and a very dense stiff soil layer at the bottom of the total soil column. The total depth of the sounding is 20.35 m. The average I_c of the soil column is 1.94. The percent fine content (FC) of the upper 5 m of soils varies from 5.0% to 57.32%, with an average of 23.62% and COV of 0.63. The FC in other layers below the depth of 5 m approximately varies from 4.3% to 18.45%, with an average of 9.05% and COV of 0.25. The unit weight of soils varies from 15.44 kN/m^3 to 21.58 kN/m^3. The average unit weight of the soil

column is 19.51 kN/m³ and the COV of unit weight of the soils is 0.06 kN/m³. A detailed depth-wise summary of the basic CPT data interpretation results is shown in Figure 60.
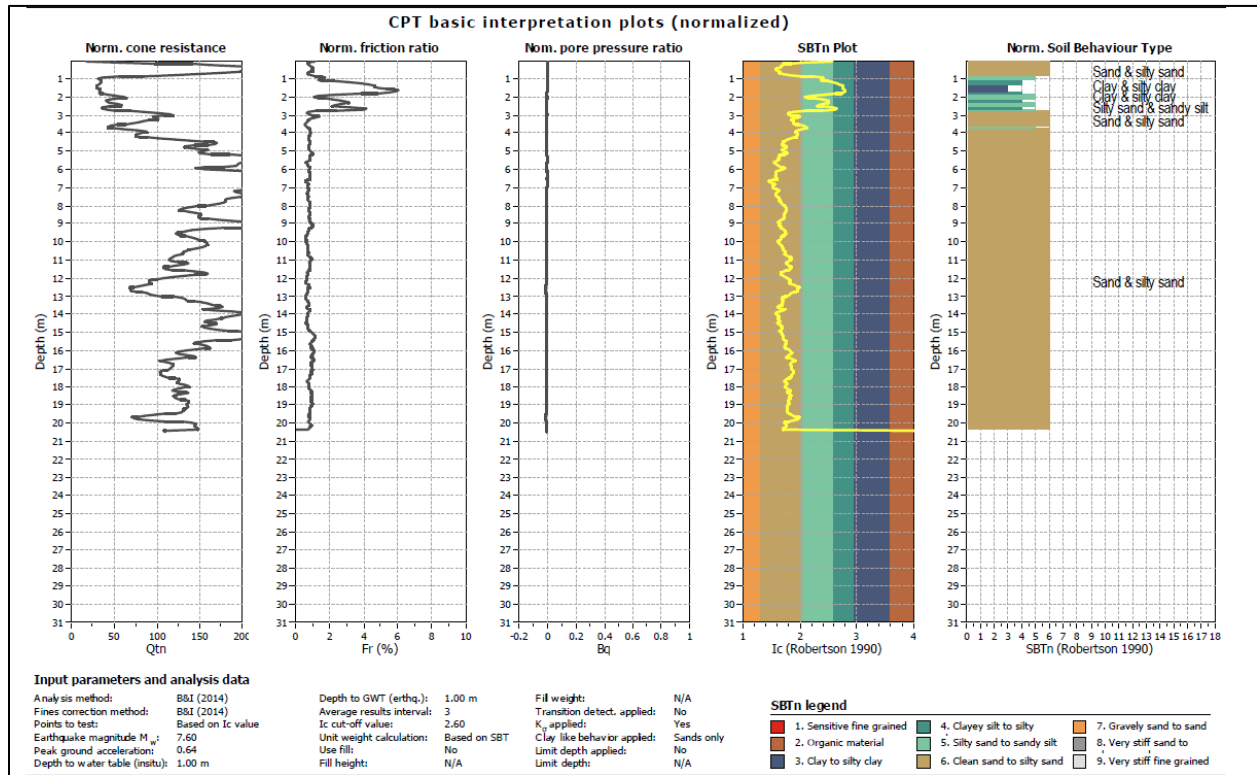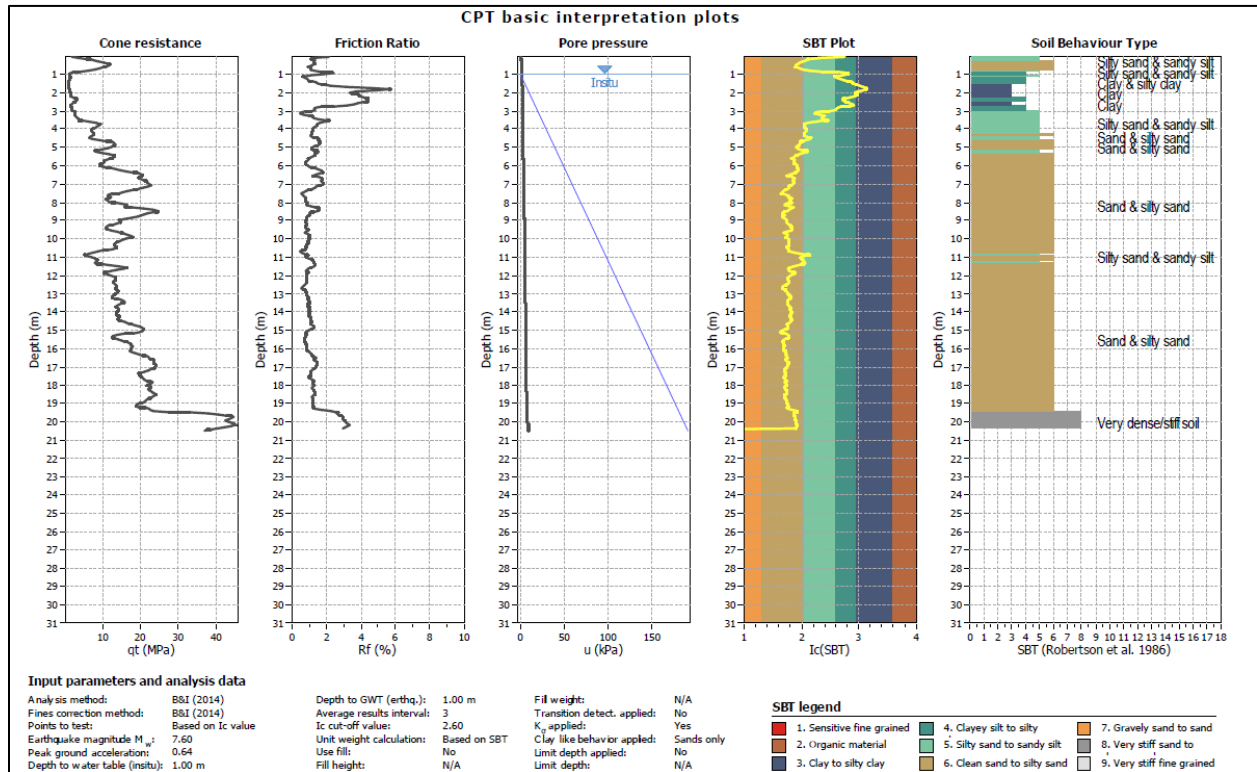

Figure 60. Basic Interpretation of CPT Data of CGD005.

Figure 61 shows the CPT based SBT classification chart and liquefaction triggering curve of based on the liquefaction analysis for the CPT sounding site, CGD005. The centroid of the soil matrix in CPT based SBT classification is 1.38% and 132.82. The normalized friction ratio has a range of 0.46 to 5.89%. The COV of the data points is 0.61. Similarly, for the normalized CPT penetration resistance, the range is 13.0 to 273.22. The COV of the resistance is 0.46. For the data points based on the upper 5 m of the soil column, the centroid of the chart has become 1.97% and 80.49, which indicates the shifting of datasets towards the fourth quadrants, Zone B, of a Cartesian coordinate increment of OCR, age, and reduction of sensitivity. The average values of the normalized friction ratio and CPT penetration resistance for the lower 15 m of the soil are 1.2 % and 149, which imparting in the major shifting towards the second quadrant of the on the classification chart. In the liquefaction triggering curve, the centroid of the data points is 151.732 and 0.75, which indicates major parts of the soils are prone to liquefaction. The average values of the equivalent clean-sand value of the normalized cone penetration resistance and modified cyclic stress ratio for the upper 5 m are 100.02 and 0.86, which means all the upper layers will trigger liquefaction of this location. On the other hand, the centroid of the data points generated based on the soil layers below 5 m are 168.41 and 0.73, these soil layers will also trigger liquefaction.
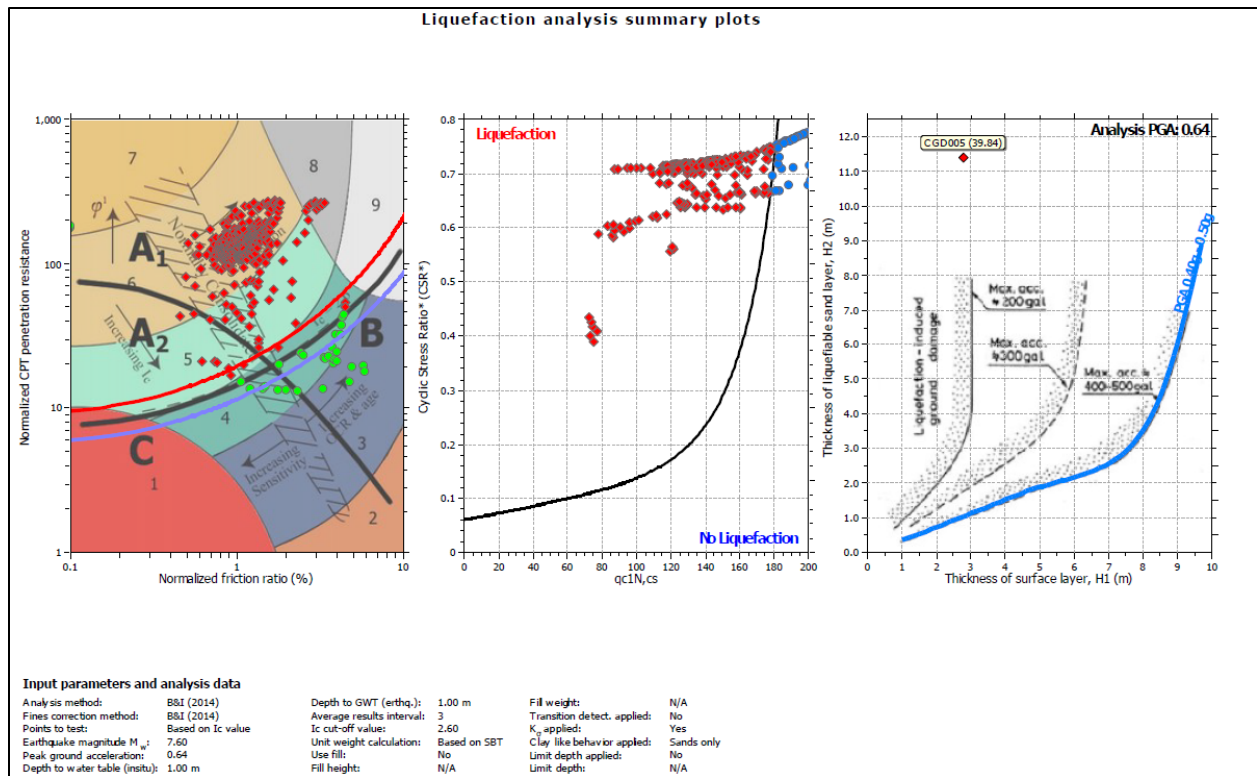
Figure 61. Summary Plots of the Liquefaction Analysis of Sounding CGD005.

According to the cyclic liquefaction analysis, the CSR is greater than CRR for several different layers within 20.5 m. Depth ranges of the soil layers with a probability of failure greater than 1 are 3 m to 6 m, 7.5 to 8.0 m, 9.0 to 17.0 m, and 17.0 to 17.5 m. The sand and silty sand layer starting from the depth of 9.0 m most vulnerable part of the soils in this site. The overall LPI for the site is around 39.839, which is highest among all other points within the county. The LSN value of the location is 49.265, which is less than 50, which indicates the possibility of showing a major expression of liquefaction. For the given criteria, the ground surface of the site is prone to a vertical settlement of 29.413 cm and a lateral displacement of 199.286 cm after earthquakes. The overall probability of liquefaction of the site is 99.63%, which is highest among any other points within the county. The parametric analysis of vertical settlements against input PGA shows that PGA values greater than 0.35g do not have any major impact on the overall vertical settlements. The maximum possible value of the vertical settlement is around 32 cm. A significant portion of the settlements can occur for a ground motion with a PGA value of less than 0.35g. The LPI for the site keeps increasing with the increment of PGAs of ground motion, with a faster pace up to 0.5g, and at a slower pace for greater values. Any ground motion with the PGA values greater than 0.27g is critical for this location and can cause "very high risk" of overall liquefaction. According to sensitivity analysis, the PGA values above 0.35g do not affect the values of LSN significantly, the major surface damages will occur before reaching out to the threshold value of 0.35g. However, this site will not face severe damage for ground motion with higher PGA values, but it can show a major risk of expression of liquefaction at a PGA value of 0.24g. The depth-wise distribution liquefiable layers of the CGD001 site along with different scenarios of liquefaction potential, probability of liquefaction, vertical settlements, and lateral displacements are shown in Figure 62, below. For this site, the depth of GWT from the ground surface reduces overall vertical settlement

at an approximate rate of 0.03 cm/cm. Similar to settlements, the depth of GWT also reduces the LSN and LPI values. But the rate of change in the decreasing of LSN for increasing the depth of GWT is very high when the GW exists within 1.5 m depth from the ground surface.
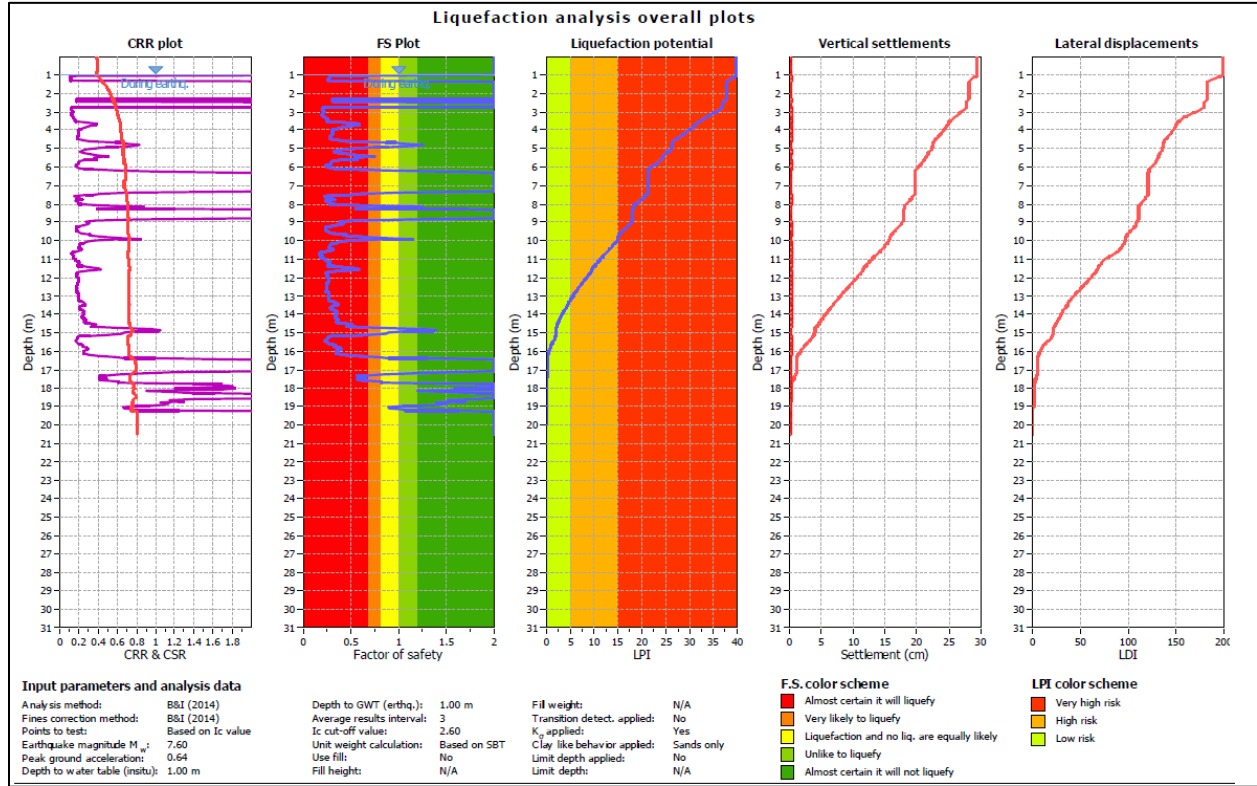
Figure 62. Overall Liquefaction Analysis Results of CGD005.

CGD010

Nearest CPT soundings near the city of Jonesboro, CGD010, CGD012, and CGD013, are located on the county road 664 near Gum Point, near the intersection of County Road 664 and County Road 655. This CPT Sounding is located on the county road 664 near Gum Point, one of the CPT nearest to the city of Jonesboro. SBT Analysis gives one thin clay layer, in the middle of two silty sand and sandy silt layers, at the top three meters of the soil column. All the other soils up to the depth of 19 m are sand and silty sand. The total depth of the sounding is 19 m. The overall cone tip resistance increased with depth. The average I_c of the soil column is 1.77. The soil layers below 2.5 m are all sand and silty sand. The percent fine content (FC) of the upper 5 m of soils varies from 3.99% to 43.84%, with an average of 12.52% and COV of 0.58. The FC in other layers below the depth of 5 m approximately varies from 3.78% to 11.72%, with an average of 6.35% and COV of 0.21. The unit weight of soils varies from 15.21 kN/m^3 to 21.29 kN/m^3. The average unit weight

of the soil column is 19.75 kN/m^3, and the COV of unit weight of the soils is 0.05 kN/m^3. A detailed depth-wise summary of the basic CPT data interpretation results is shown in Figure 63.
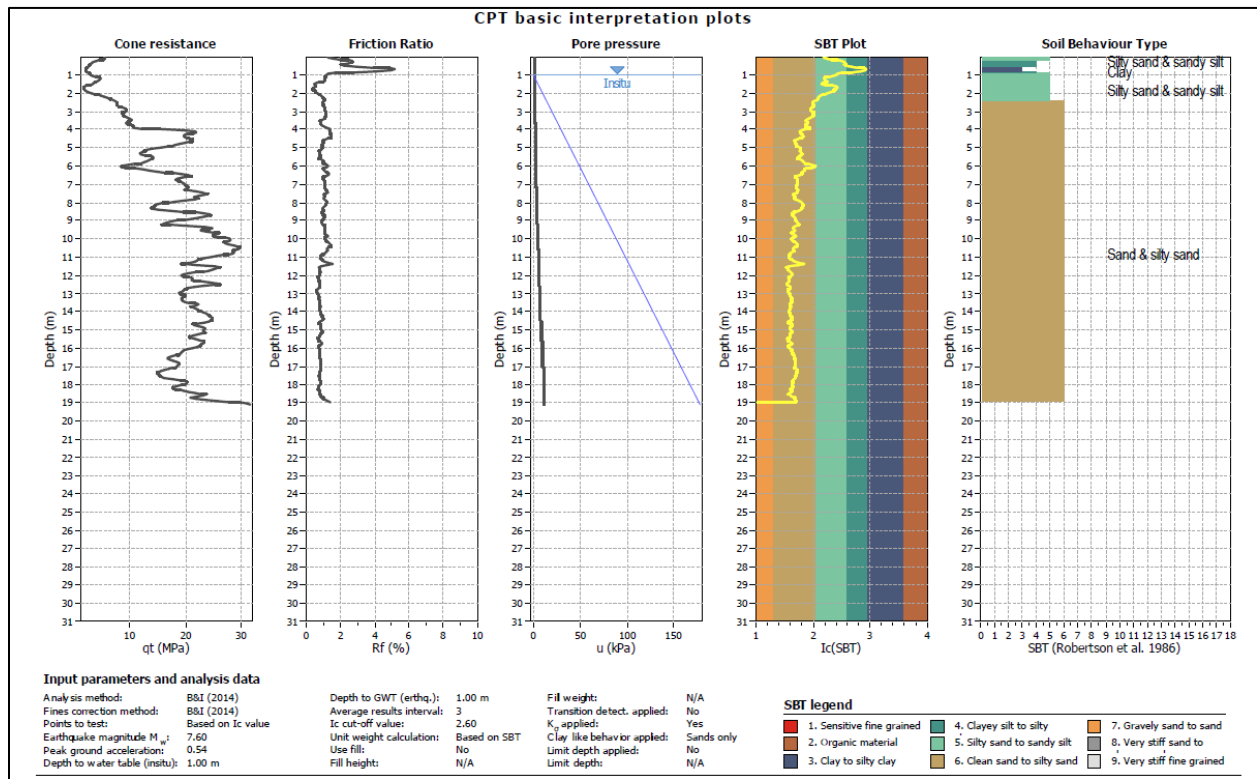
Figure 63. Basic Interpretation of CPT Data of CGD010.

Figure 64 shows the CPT based SBT classification chart and liquefaction triggering curve of based on the liquefaction analysis for the CPT sounding site, CGD010. The centroid of the soils metrics in CPT based SBT classification chart is 1.05% and 169.95. The normalized friction ratio has a range of 0.38 to 5.23%. The COV of the data points is 0.50. Similarly, for the normalized CPT penetration resistance, the range is 26.16 to 326.71. The COV of the resistance is 0.38. For the data points based on the upper 5 m of the soil column, the centroid of the chart has become 1.34% and 133.058, which indicates the shifting of datasets towards the fourth quadrants or Zone B. So, the data points close to zone B in the charts are originated from the upper part of the soil column. The average values of the normalized friction ratio and CPT penetration resistance for the lower 15 m of the soil is 0.79 % and 145, which imparting in the major shifting towards Zone A$_1$ of the classification chart. According to the liquefaction triggering curves, the coordinates of the centroid of the data points are 178.68 and 0.65, which indicates a significant part of these soils are does not trigger liquefaction. The average values of the equivalent clean-sand value of the normalized cone penetration resistance and modified cyclic stress ratio for the upper 5 m are 141.54 and 0.79, which imparts to a higher risk of liquefaction. On the other hand, the centroid of the data points generated based on the soil layers below 5 m is 19.86 and 0.60, these points lie below the liquefaction triggering curve. So, the bottom layers of the soil are not susceptible to liquefaction here, so the overall liquefaction potential is also low for this location.

Figure 64. Summary Plots of the Liquefaction Analysis of Sounding CGD010.

According to the cyclic liquefaction analysis, the CSR is greater than CRR for approximately four to five different layers within the total depth of the soundings. The depth ranges of the soil layers with FS less than 1.00 are 1 to 4.0 m, 5.0 to 6.5 m, 8.0 to 8.5 m, 13.0 to 14.0 m, and 16.0 to 19.0 m. The overall LPI for the site is around 22.281, which indicates a very high risk of liquefaction. The LSN value of this location is 48.753, which indicates the possibility of showing major expression of liquefaction of the upper layer of the soils. Given the criteria, the ground surface of the site is prone to a vertical settlement of 18.681 cm and a lateral displacement of 116.59 cm after earthquakes. The overall probability of liquefaction of the site is 85.38%. A detailed depth-wise summary of the liquefaction analysis results is shown in Figure 65.

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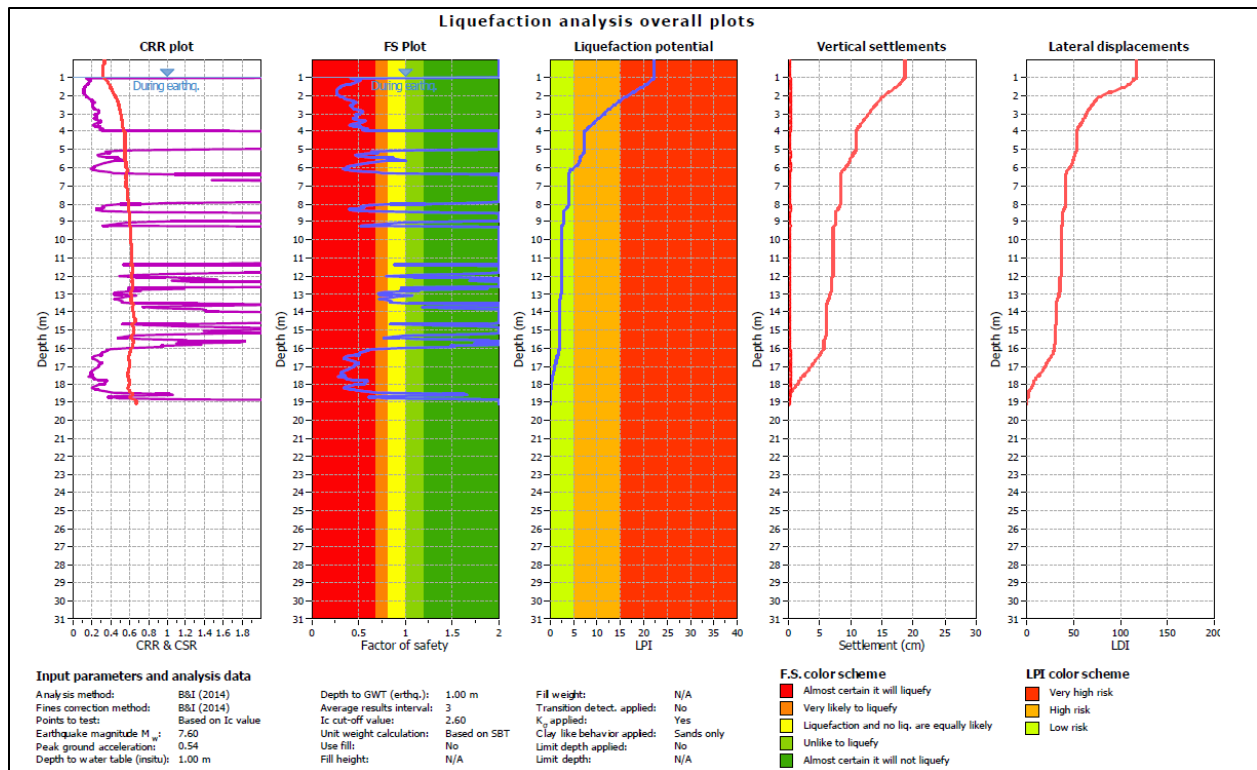
Figure 65. Overall Liquefaction Analysis Results of CGD010.

CGD012

This CPT Sounding is also located on the county road 664 near Gum Point, one of the CPT nearest to the city of Jonesboro, near the intersection of County Road 664 and County Road 655. The SBT Analysis shows a clay and silty clay layer in the middle of two silty sand and sandy silt layers in the top 2 m of the soil column. All the other soils in the 20.2 m depth columns are sand and silty sand. The average I_c of the soil column is 1.77. The soil layers below 1.5 m are all sand and silty sand. The percent fine content (FC) of the upper 5 meters of soils varies from 02.05% to 53.64%, with an average of 12.17% and COV of 1.11. The FC in other layers below the depth of 5 m approximately varies from 2.95% to 12.59%, with an average of 7.13% and COV of 0.30. The unit weight of soils varies from 16.51 kN/m³ to 21.58 kN/m³. The average unit weight of the soil column is 19.69 kN/m³, and the COV of unit weight of the soils is 0.049 kN/m³. A detailed depth-wise summary of the basic CPT data interpretation results is shown in Figure 66.

CPT basic interpretation plots (normalized)

| Norm. cone resistance | Norm. friction ratio | Nom. pore pressure ratio | SBTn Plot | Norm. Soil Behaviour Type |

Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	1.00 m
Fines correction method:	B&I (2014)	Average results interval:	3
Points to test:	Based on Ic value	Ic cut-off value:	2.60
Earthquake magnitude M_w:	7.60	Unit weight calculation:	Based on SBT
Peak ground acceleration:	0.54	Use fill:	No
Depth to water table (insitu):	0.00 m	Fill height:	N/A

Fill weight:	N/A
Transition detect. applied:	No
K_o applied:	Yes
Clay- like behavior applied:	Sands only
Limit depth applied:	No
Limit depth:	N/A

SBTn legend

1. Sensitive fine grained	4. Clayey silt to silty	7. Gravely sand to sand
2. Organic material	5. Silty sand to sandy silt	8. Very stiff sand to
3. Clay to silty clay	6. Clean sand to silty sand	9. Very stiff fine grained

Figure 66. Basic Interpretation of CPT Data of CGD012.

Figure 67, below, shows the CPT based SBT classification chart and liquefaction triggering curve of based on the liquefaction analysis for the CPT sounding site, CGD012. The centroid of the soil matrix in CPT based SBT classification chart is 1.08% and 170.54. The normalized friction ratio has a range of 0.39 to 7.42%. The COV of the data points is 0.68. Similarly, for the normalized CPT penetration resistance, the range is 16.37 to 400.64, which is significantly higher in comparison with surrounding locations. The COV of the resistance is 0.44. For the data points based on the upper 5 m of the soil column, the centroid of the chart has become 1.4% and 179.28, which indicates the shifting of datasets towards the first quadrants, the upper region of A_1 and region 8 of the chart. These results imply the presence of very stiff sand to clayey sand at the top layers of the location, which also signifies lower sensitivity, higher OCR, and aged soil layers. The average values of the normalized friction ratio and CPT penetration resistance for the lower 15 m of the soil are 0.97 % and 167.69, which contributes to the shifting of the data points towards the third quadrant of the classification chart. So, the soil layers below five meters comparatively loose soils with higher sensitivity. In the liquefaction triggering curve (Figure 67), the centroids of the data points are 174.33 and 0.65, which indicates major parts of the soils are not susceptible to liquefaction. The average values of the equivalent clean-sand value of the normalized cone penetration resistance and modified cyclic stress ratio for the upper 5 m are 165.91 and 0.79, which indicates a certain a significant risk of liquefaction. On the other hand, the centroid of the data points is generated based on the soil layers below 5 m are 177.04 and 0.61, these points lie mostly on the "no liquefaction part triggering curve." So, the bottom layers of the soil column have an overall low risk of triggering liquefaction.
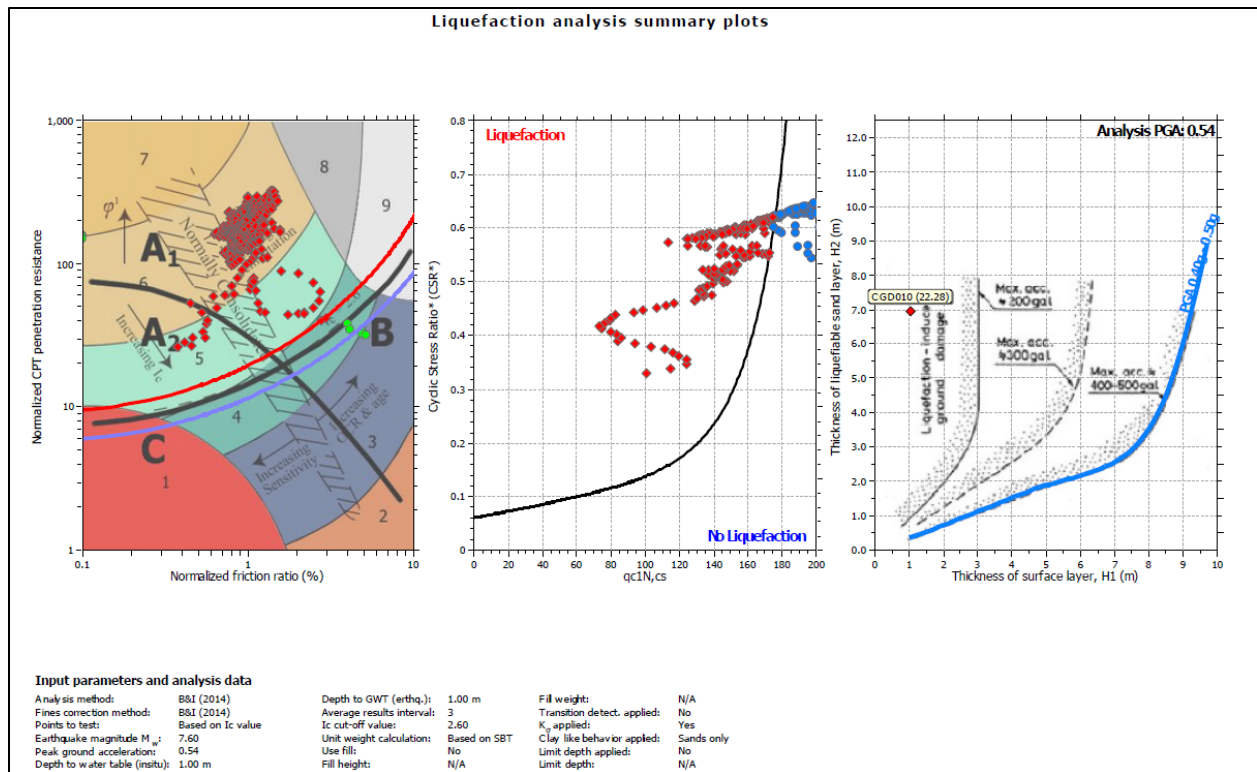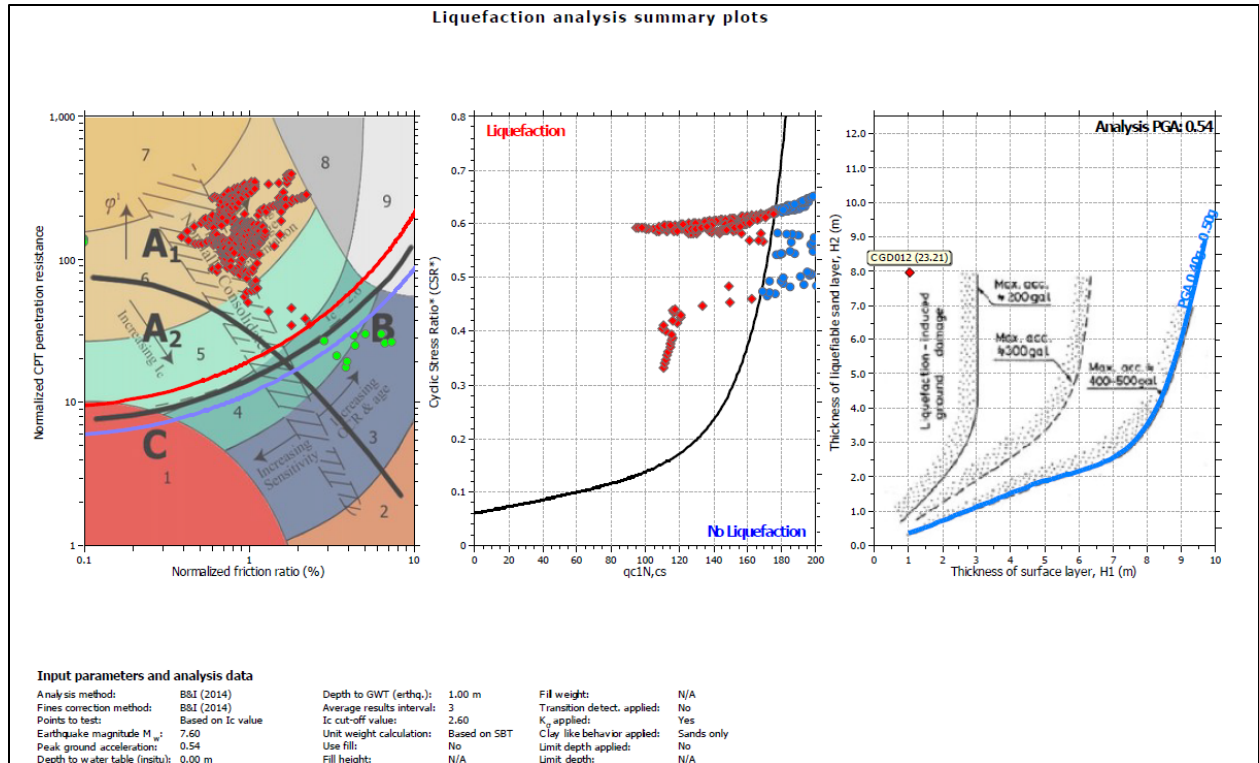
Figure 67. Summary Plots of the Liquefaction Analysis of Sounding CGD012.

According to the cyclic liquefaction analysis, the CSR is greater than CRR for approximately five different layers within 20.5 m. The depth ranges of the soil layers with an FS less than 1.0 are 0 to 2.5 m, 7.5 to 11 m, 12.0 to 13.8 m, 15.5 to 18.0 m, and 18.2 to 19.5 m. The overall LPI for the site is around 23.21, the value is greater than 15 which indicates a very high risk of liquefaction. The LSN value of the location is 37.77, which indicates moderate to a severe expression of liquefaction to the upper layer of the soils. The ground surface of the site is prone to a vertical settlement of 21.642 cm and a lateral displacement of 135.744 cm after earthquakes. The overall probability of liquefaction of the site is 87.74%. The depth-wise distribution liquefiable layers of the site along with different scenarios of liquefaction potential, probability of liquefaction, vertical settlements, and lateral displacements is shown in Figure 68.
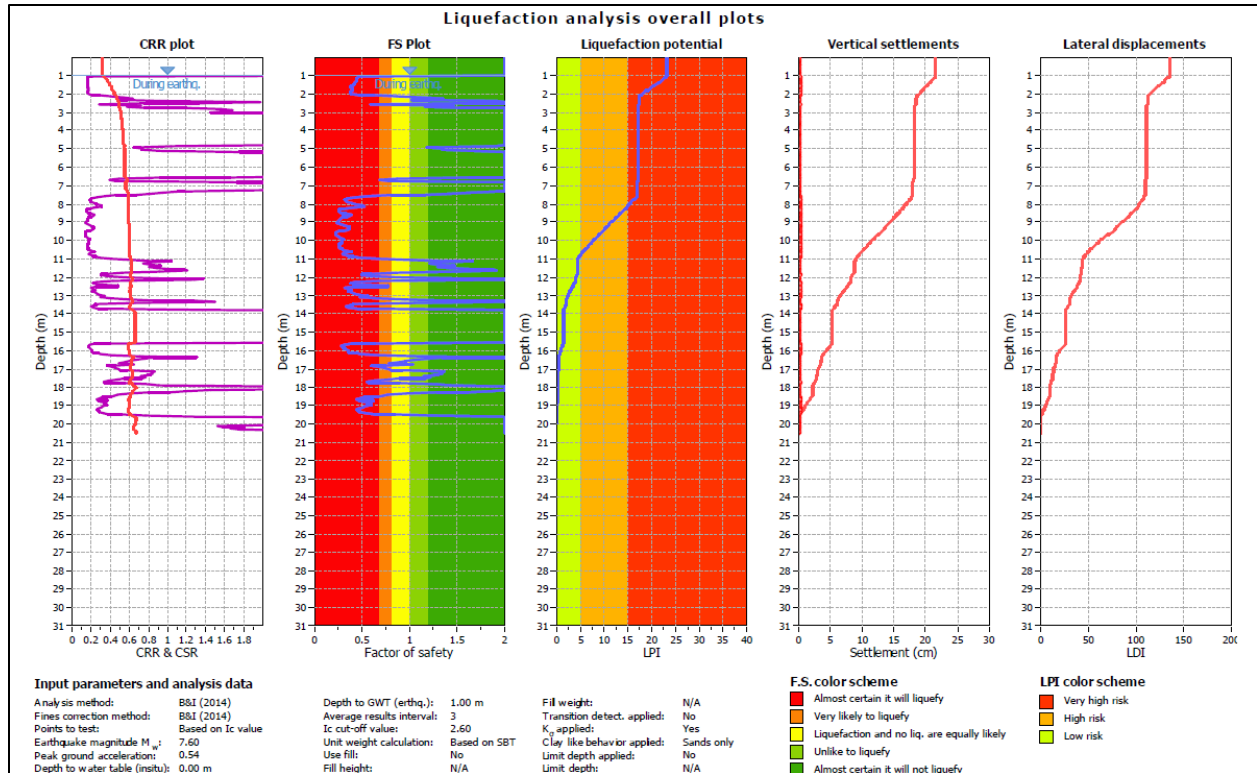
Figure 68. Overall Liquefaction Analysis Results of CGD012.

CGD013

This CPT Sounding is also located on the county road 664 near Gum Point, one of the CPT is located nearest to the city of Jonesboro, near the intersection of County Road 664 and County Road 673. The SBT Analysis shows one thick clay layer in the middle of one silty sand and sandy silt layer, and one thin clay and silty clay within the top 2.2 m of the soil column. All the other soils in the 20.4 m thick columns are mainly sand and silty sand, except one very thin silty sand and sandy silt layer at a depth of 13 m. The total depth of the sounding is 20.4 m. The cone tip resistance is comparatively low up to the depth of 2.5 m and almost uniform at higher depth. The average I_c of the soil column is 2.04. The soil layers below 2.2 m are all sand and silty sand. The percent fine content (FC) of the upper 5 m of soils varies from 2.22% to 70.12%, with an average of 21.17% and COV of 1.15. The FC in other layers below the depth of 5 m approximately varies from 2.99% to 35.3%, with an average of 6.79% and COV of 0.45. The unit weight of soils varies from 16.96 kN/m^3 to 21.58 kN/m^3. The average unit weight of the soil column is 19.88 kN/m^3 and the COV of unit weight of the soils is 0.049. A detailed depth-wise summary of the basic CPT data interpretation results is shown in Figure 69.
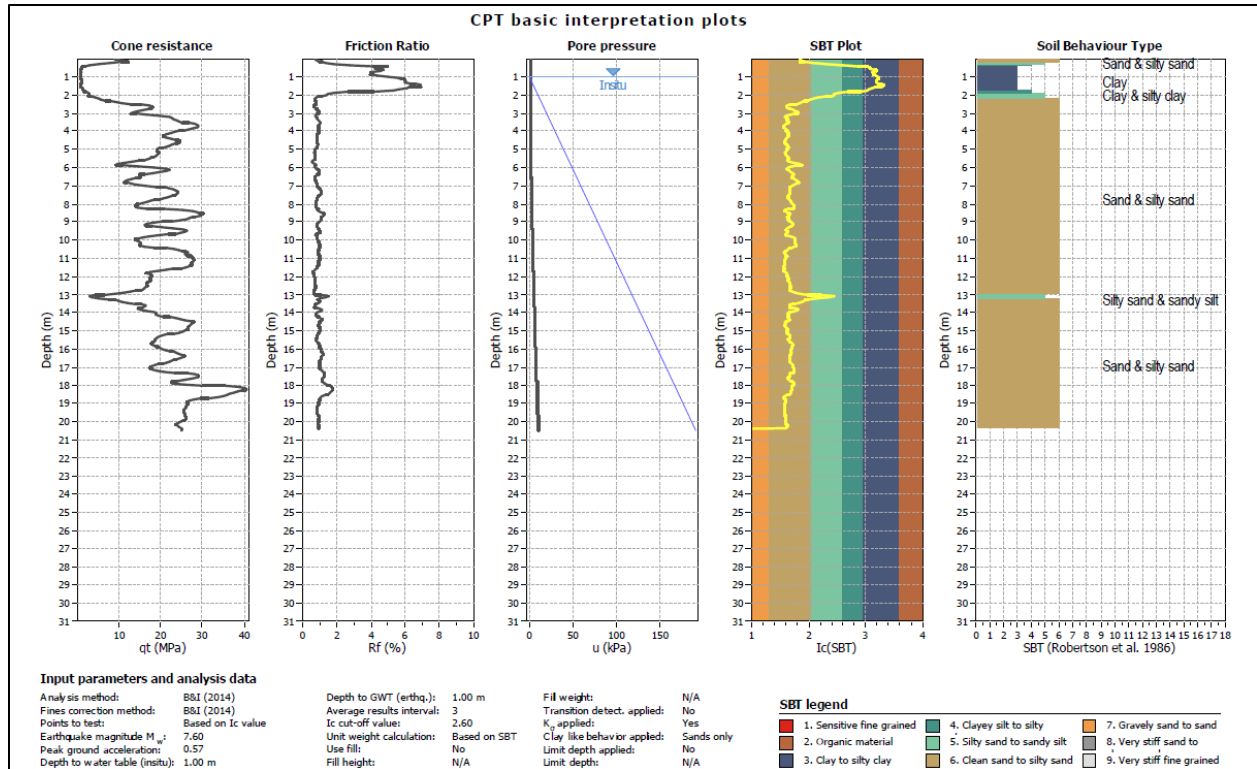
Figure 69. Basic Interpretation of CPT Data of CGD013.

Figure 70, below, shows the CPT based SBT classification chart and liquefaction triggering curve of based on the liquefaction analysis for the CPT sounding site, CGD013. The centroid of the soil matrix in CPT based SBT classification chart is 1.28% and 182.14. The normalized friction ratio has a range of 0.54 to 7.15%. The COV of the data points is 0.93. Similarly, for the normalized CPT penetration resistance, the range is 12.03 to 417.05. The COV of the resistance is 0.46. For the data points based on the upper 5 m of the soil column, the centroid of the chart has become 2.26% and 185.87, which indicates the shifting of datasets towards the first quadrants, Zone A_1 and region 8 of the chart. The data indicates the presence of clay and stiff clays in the upper layers of the soil column. The average values of the normalized friction ratio and CPT penetration resistance for the lower 15 m of the soil are 0.96 % and 180.92, which imparting in the major shifting towards the third quadrant, towards region A_2 and region C, of the classification chart. So, the soil layers below 5 meters are more sensitive. In the liquefaction triggering curve in Figure 70, the centroid of the data points is 184.32 and 0.69, which indicates a major portion of the soils are not prone to liquefaction in this location. The average values of the equivalent clean-sand value of the normalized cone penetration resistance and modified cyclic stress ratio for the upper 5 m are 154.82 and 0.81, which imparts to major risk of liquefaction. On the other hand, the average of the data points generated based on the soil layers below 5 m are 193.84 and 0.65, these points are mostly in the "no liquefaction" zone of the liquefaction triggering curve. For this site, the upper soil layers are facing liquefaction due to the presence of sand layers at the bottom, not because of the vulnerability of the upper layer soils. So, the replacement of upper layer soils would not impact the chances of liquefaction, which is sometimes critical for foundation design.
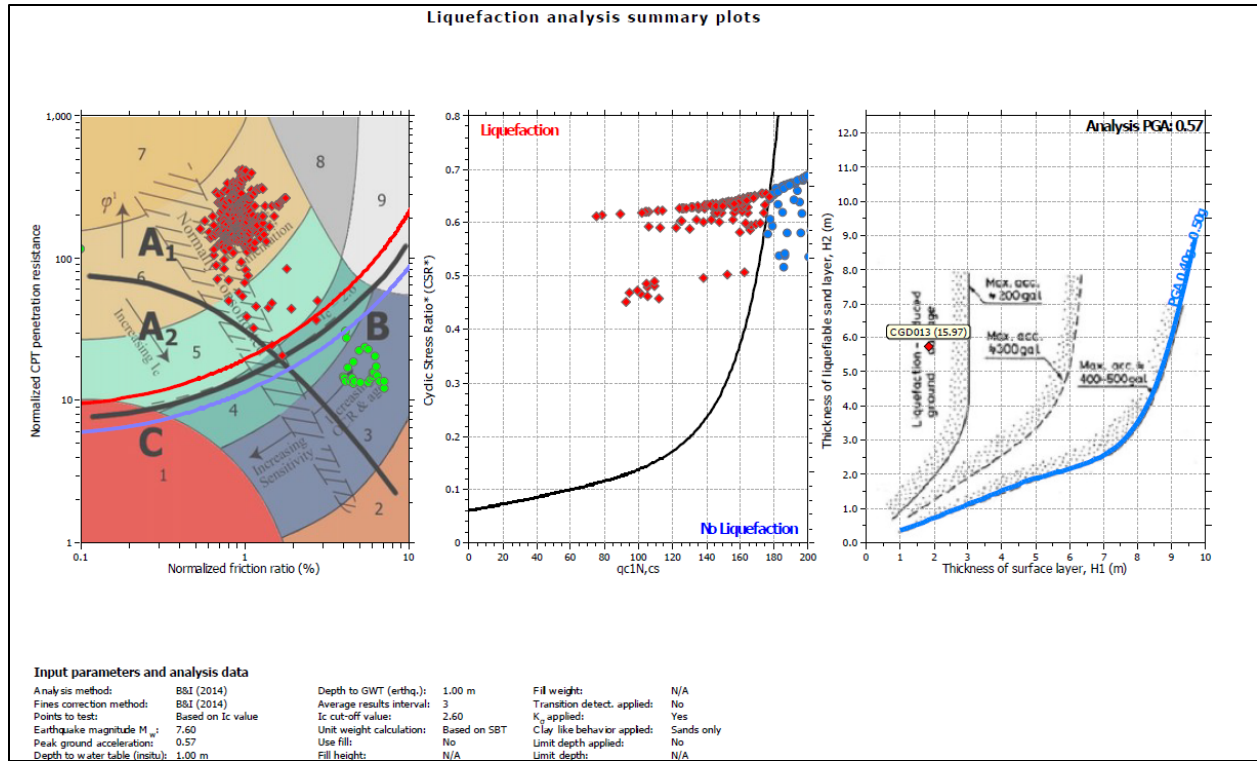
Figure 70. Summary Plots of the Liquefaction Analysis of Sounding CGD013.

According to the cyclic liquefaction analysis, the CSR is greater than CRR for several different layers of varied thickness. Depth ranges of the soil layers with an FS less than 1.0 are 2 m to 2.8 m, 5.8 m to 6.0 m, 6.6 m to 7.2 m, a very thin layer at 8.0 m, another layer at a depth between 9.9 m to 10.2 m, 12.0 m to 14.0 m, and 15 m to 17 m. The overall LPI for the site is around 15.967, the value is slightly greater than 15, which indicates a very high risk of liquefaction. However, this is the location with the lowest LPI value inside the Craighead County. The LSN value of the location is 20.382, which indicates the possibility of moderate expression of liquefaction at the upper layer of the soils. The ground surface of the site is prone to a vertical settlement of 14.53 cm and a lateral displacement of 87.08 cm after earthquakes. The overall probability of liquefaction of the site is 59.59%, which is lowest among all estimated values for other sites considered within the county. The depth-wise distribution liquefiable layers of the site along with different scenarios of liquefaction potential, probability of liquefaction, vertical settlements, and lateral displacements is shown in Figure 71.

95
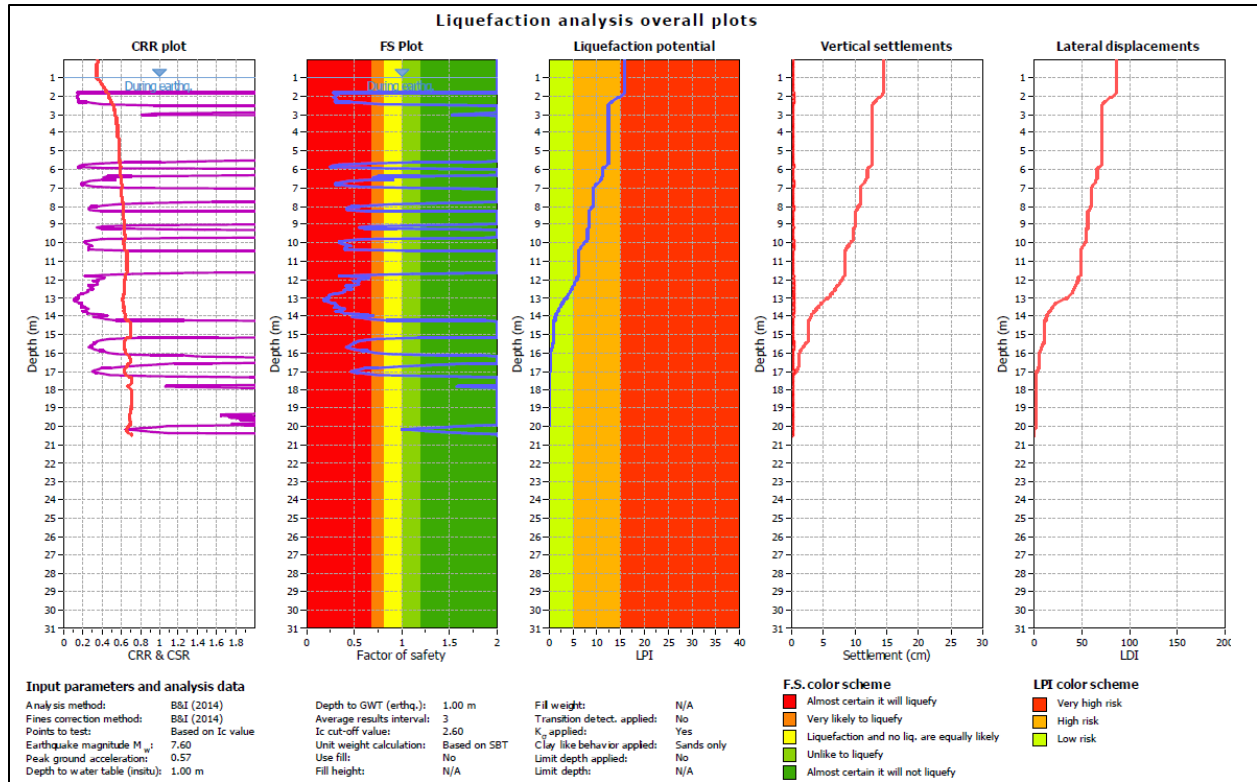
Figure 71. Overall Liquefaction Analysis Results of CGD013.

CGD014

This CPT Sounding is located at the intersection of AR-135 S and County Road 834, near Mangrum in Craighead. The SBT Analysis shows one thin clay and silty clay inside two sand and silty sand layers within the top two meters of the soil column. All the other soils up to the depth of 20.2 m are sand and silty sand. The cone tip resistance is comparatively low up to the depth of 4.0 m and fluctuates significantly at higher depth. The average I_c of the soil column is 1.71. The soil layers below 2 m are all sand and silty sand. The percent fine content (FC) of the upper 5 m of soils varies from 02.35% to 35.65%, with an average of 9.17% and COV of 0.75. The FC in other layers below the depth of 5 m approximately varies from 2.55% to 12.98%, with an average of 5.87% and COV of 0.33. The unit weight of soils varies from 17.49 kN/m³ to 21.03 kN/m³. The average unit weight of the soil column is 19.53 kN/m³ and the COV of unit weight of the soils is

0.034. A detailed depth-wise summary of the basic CPT data interpretation results is shown in Figure 72.
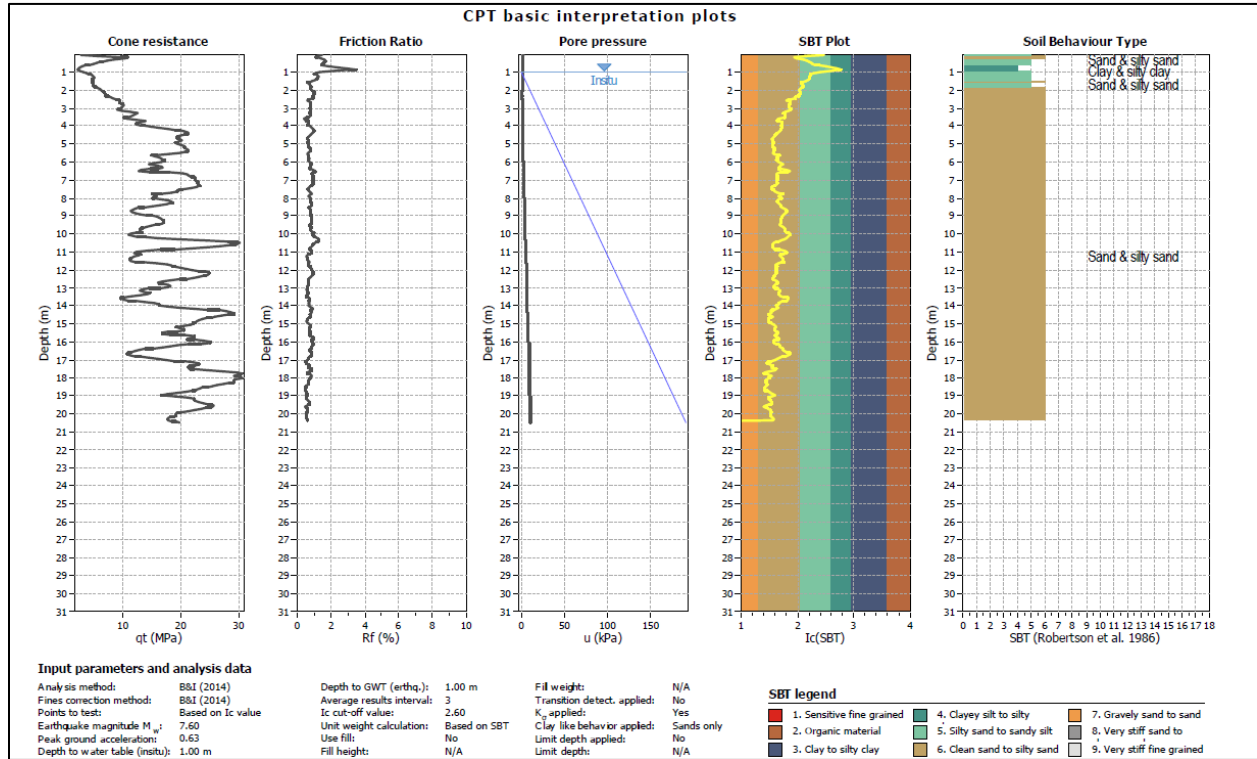
Figure 72. Basic Interpretation of CPT Data of CGD014.

Figure 73 shows the CPT based SBT classification chart and liquefaction triggering curve of based on the liquefaction analysis for the CPT sounding site, CGD014. The centroid of the soil matrix in CPT based SBT classification chart is 0.82% and 163.005. The normalized friction ratio has a range of 0.42 to 3.48%. The COV of the data points is 0.37. Similarly, for the normalized CPT penetration resistance, the range is 35.47 to 295. The COV of the resistance is 0.35. For the data points based on the upper 5 m of the soil column, the centroid of the chart has become 1.01% and 150.85, which indicates the shifting of datasets towards the fourth quadrants, region B, of signifies increment of OCR, age, and cementation and shifting towards less sensitivity. The average values of the normalized friction ratio and CPT penetration resistance for the lower 15 meters of the soil are 0.77 % and 166.966, which imparting in the major shifting towards the second quadrant, zone A_1, of the classification chart. In the liquefaction triggering curve in Figure 73, the centroid of the data points is 171.57 and 0.74, which indicates a major portion of the soils are prone to liquefaction. The average values of the equivalent clean-sand value of the normalized cone penetration resistance and modified cyclic stress ratio for the upper 5 m are 156.45 and 0.85, which indicates these upper layers trigger severe liquefaction. the centroid of the data points generated based on the soil layers below 5 m are 176.44 and 0.70, these points are also mostly in the liquefaction zone side of the liquefaction triggering curve. Again, further analysis of the penetration resistance and cyclic stress ration shows that the probability of the liquefaction of the upper 10 m and bottom 10 m of the soils are also similar. So, it can be concluded that the overall soil column triggers liquefaction for this site.

Figure 73. Summary Plots of the Liquefaction Analysis of Sounding CGD014.

According to the cyclic liquefaction analysis, the CSR is greater than CRR for several different layers within the total depth of the sounding. Depth ranges of the distinguishable soil layers with a probability of failure greater than 1.0 are 1m to 4.0 m, 8.0 to 10 m, 11.0 to 12.0 m, 12.5 to 14.0 m, and 15.0 to 17.0 m. The overall LPI for the site is around 28.969, which indicates a very high risk of liquefaction. The LSN value of the location is 46.495, which means the site has the possibility of showing major expression of liquefaction in at the ground surface of the soils. For the given criteria, the ground surface of the site is prone to a vertical settlement of 24.531 cm and lateral displacement of 152.942 cm after earthquakes. The overall probability of liquefaction of the site is 96.17%. The depth-wise distribution liquefiable layers of the site along with different scenarios of liquefaction potential, probability of liquefaction, vertical settlements, and lateral displacements is shown in Figure 74.
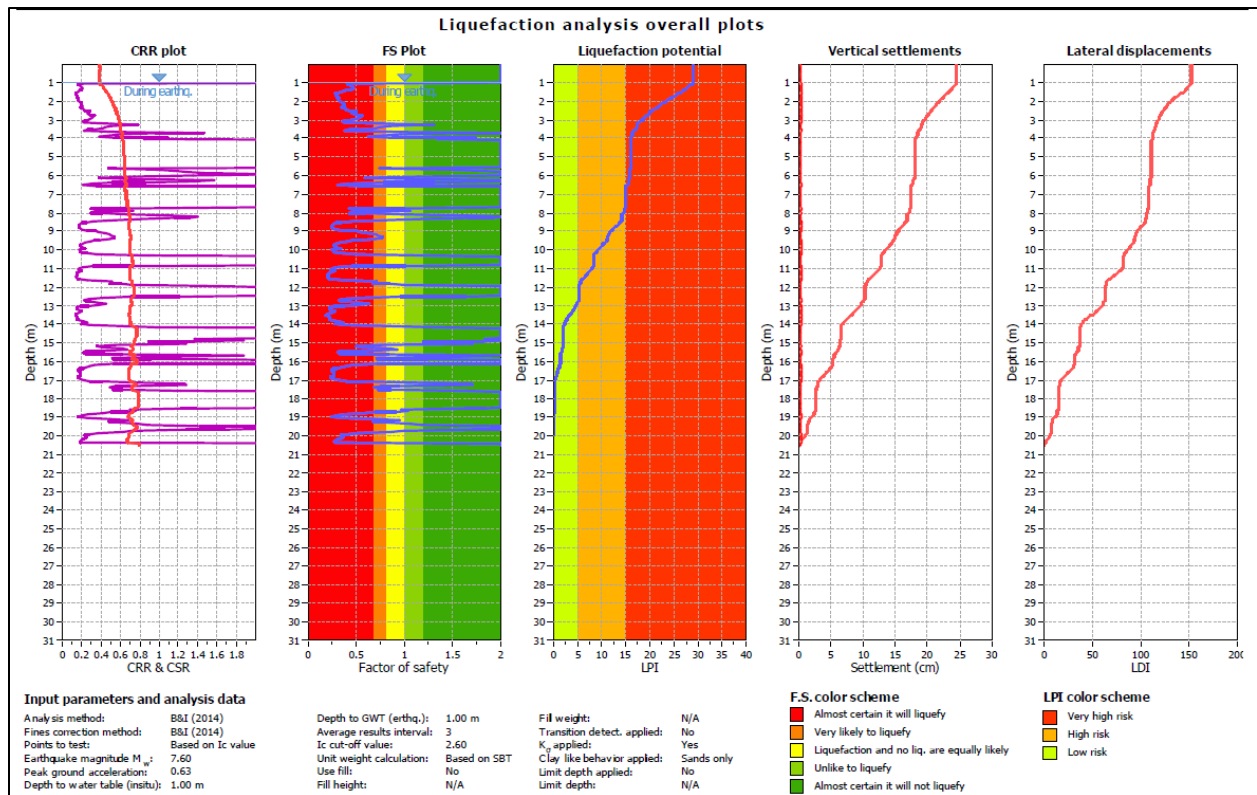
Figure 74. Overall Liquefaction Analysis Results of CGD014.

CGD015

This CPT Sounding is located at the intersection of Highway 139 S and County Road 834, near Hancock. The SBT Analysis shows all 20 m depth of soil columns is sand and silty sand, except one to two very thin silty sand and sandy silt layer at a depth of 2 m. The average I_c of the soil column is 1.66. The percent fine content (FC) of the upper 5 m of soils varies from 2.26% to 41.32%, with an average of 8.0% and COV of 1.01. The FC in other layers below the depth of 5 m approximately varies from 2.07% to 9.4%, with an average of 4.89% and COV of 0.312. The unit weight of soils varies from 15.45 kN/m³ to 21.58 kN/m³. The average unit weight of the soil

column is 19.76 kN/m^3 and the COV of unit weight of the soils is 0.06. A detailed depth-wise summary of the basic CPT data interpretation results is shown in Figure 75.
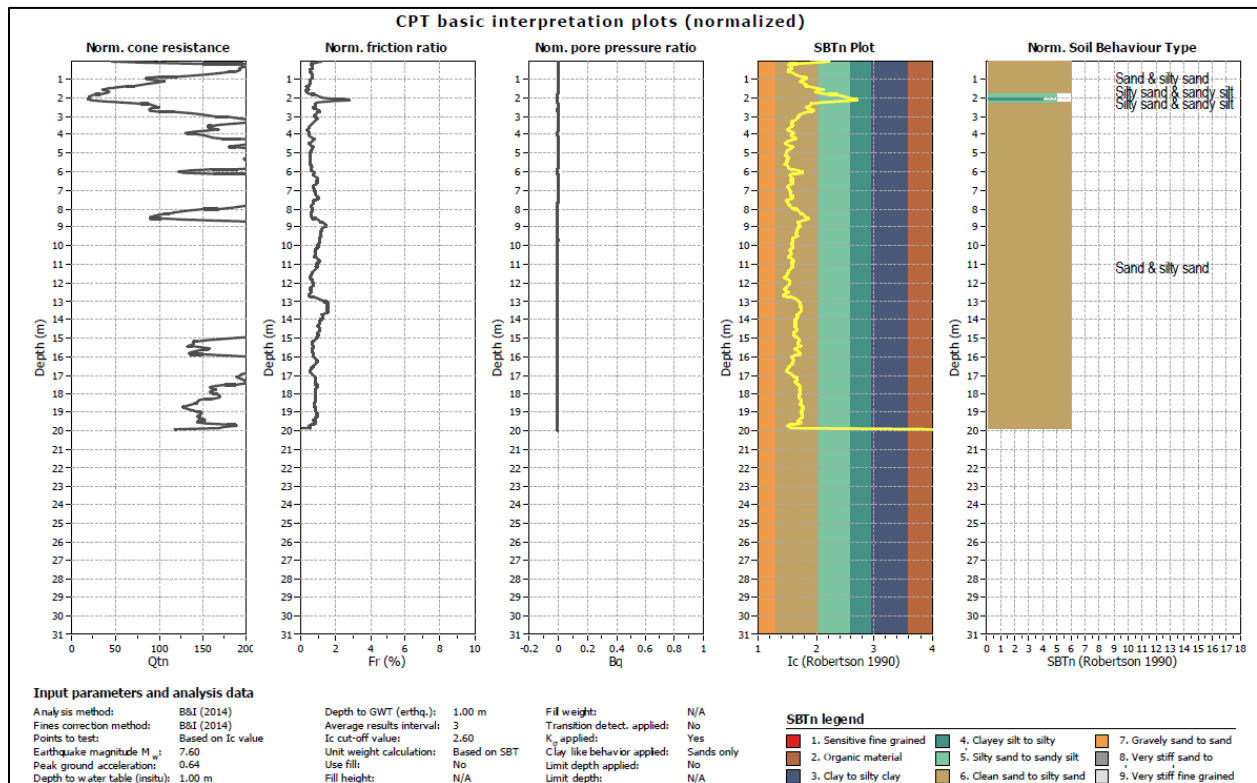
Figure 75. Basic Interpretation of CPT Data of CGD015.

Figure 76 shows the CPT based SBT classification chart and liquefaction triggering curve of based on the liquefaction analysis for the CPT sounding site, CGD015. The centroid of the soil matrix in CPT based SBT classification chart is 0.83% and 197.23. The normalized friction ratio has a range of 0.31 to 2.8%. The COV of the data points is 0.35. Similarly, for the normalized CPT penetration resistance, the range is 18.59 to 301.2. The COV of the resistance is 0.34. For the data points based on the upper 5 m of the soil column, the centroid of the chart has become 0.72% and 134.34, which indicates the shifting of datasets towards the third quadrants, towards A_2 and C region of the Chart, of a Cartesian coordinate system and signifies liquefaction reduction of OCR, age, and cementation and shifting towards higher sensitivity of soils in region 6. The average values of the normalized friction ratio and CPT penetration resistance for the lower 15 m of the soil are 0.87% and 218.48, which imparting in the major shifting towards the first quadrant, towards stiffer soil types, of the on the classification chart. In the liquefaction triggering curve in Figure 76, the coordinates of the centroid of the data points are 199.781 and 0.77, which means most of the soil layers do not trigger liquefaction. The average values of the equivalent clean-sand value of the normalized cone penetration resistance and modified cyclic stress ratio for the upper 5 m are 138.05 and 0.86, which indicates the possibility of triggering liquefaction of these soil layers. On the other hand, the centroid of the data points generated based on the soil layers below 5 m are 220.43 and 0.74, these bottom layers do not trigger liquefaction. In this location, the bottom layers are not susceptible to liquefaction, by replacing the topsoil or using deep foundations liquefaction damage can be reduced.
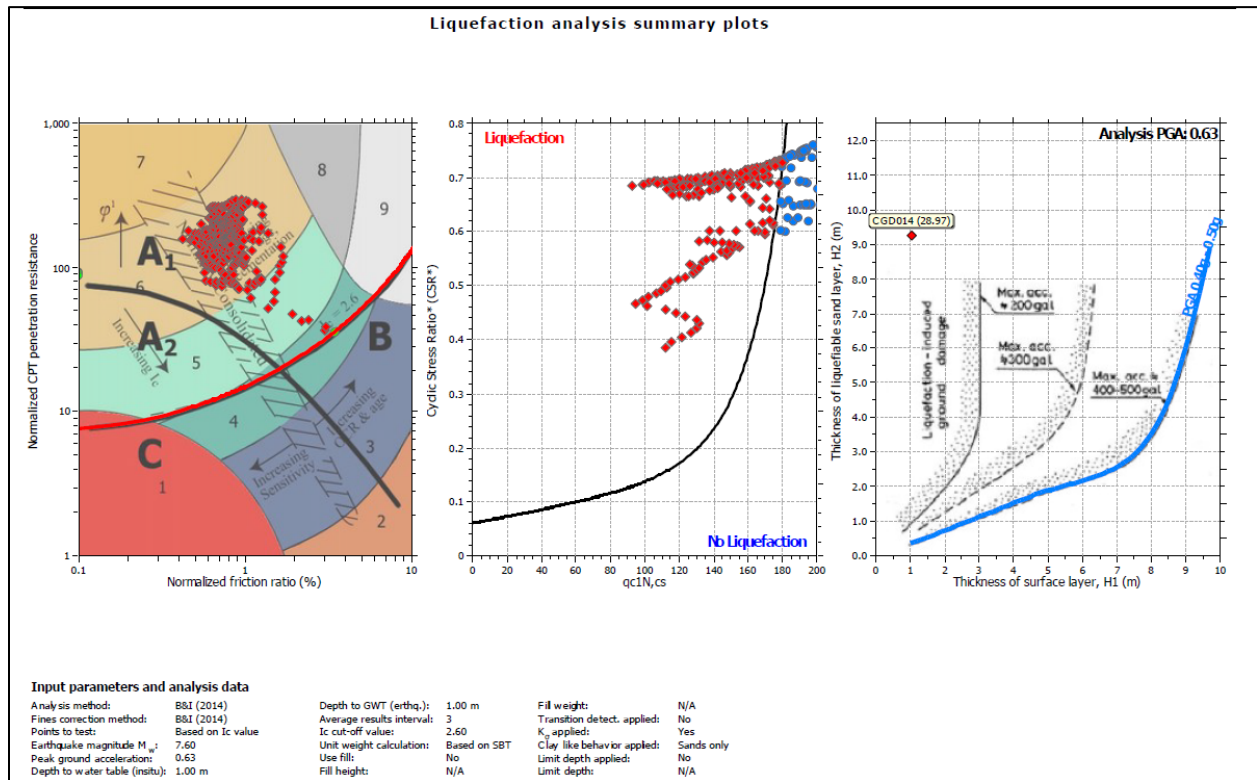
Figure 76. Summary Plots of the Liquefaction Analysis of Sounding CGD015.

According to the cyclic liquefaction analysis, the CSR is greater than CRR for four different layers within 20.5 m. Depth ranges of the soil layers with a probability of failure greater than 1.0 are 1 to 4.0 m, 8.0 to 9.0 m, 15.0 to 16.0 m, and 18.2 to 19.8 m. The overall LPI for the site is around 21.426, which indicates a very high risk of liquefaction. However, this site has second-lowest values of LPI in comparison with the other locations within the county. The LSN of this location is 46.54, which indicates the possibility of showing a major expression of liquefaction. For the given criteria, the ground surface of the site is prone to a vertical settlement of 14.282 and a lateral displacement of 110.366 after earthquakes. The estimated values of surface settlement and lateral displacement are lowest among all other sites with the county. The overall probability of liquefaction of the site is 82.90%. The depth-wise distribution liquefiable layers of the site along with different scenarios of liquefaction potential, probability of liquefaction, vertical settlements, and lateral displacements is shown in Figure 77.
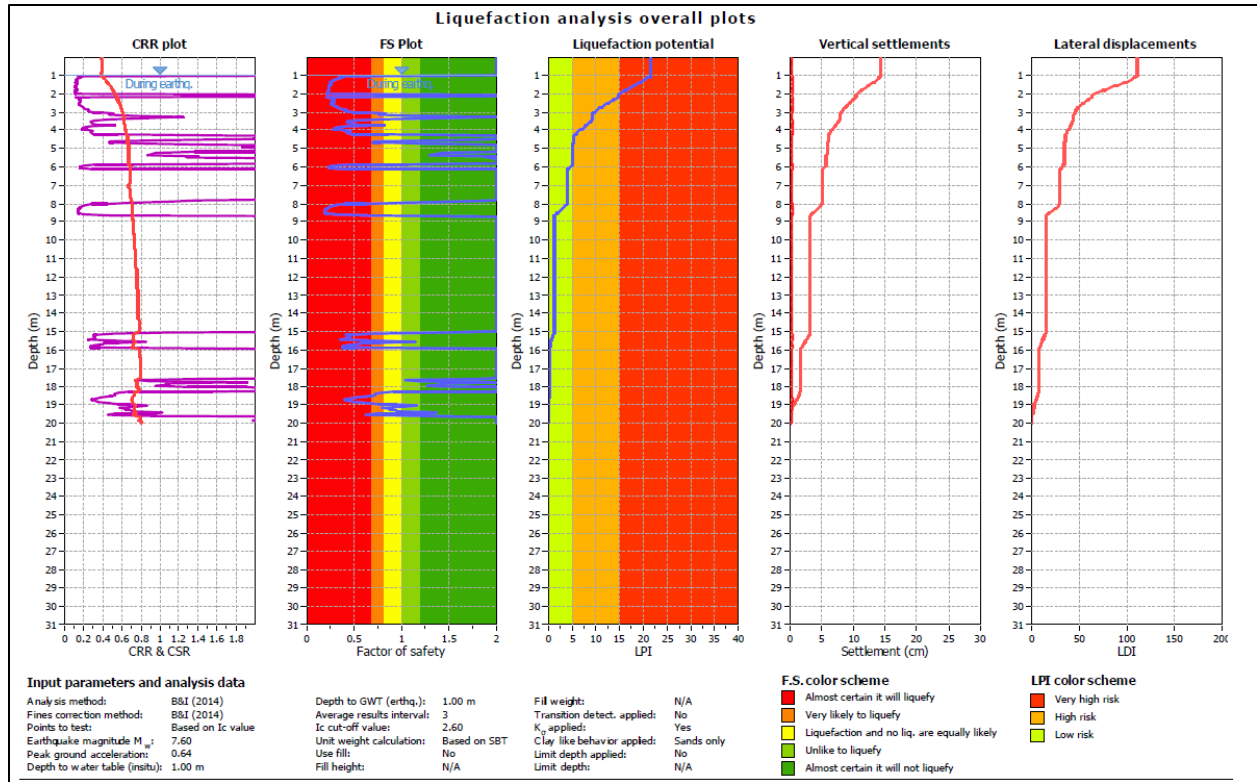
Figure 77. Overall Liquefaction Analysis Results of CGD015.

CGD016

This CPT Sounding is located in the east of County Road 817 near Lake City. The site is located at the west of Ditch No 60 and Saint Francis River bank near Lake City. The SBT Analysis shows the presence of clay and silty clay, silty sand, and sandy silt layer overlapped one another within the top 2 m of the soil column. All the other soils in the 20.4 m thick columns are sand and silty sand, except a very dense or stiff soil layer at a depth of 15.2 m. The cone tip resistance is comparatively low up to the depth of 4.0 m. The average I_c of the soil column is 1.78. The percent fine content (FC) of the upper 5 m of soils varies from 5.34% to 41.58%, with an average of 13.53% and COV of 0.76. The FC in other layers below the depth of 5 m approximately varies from 3.24% to 12.32%, with an average of 6.59% and COV of 0.34. The unit weight of soils varies from 17.52 kN/m^3 to 21.58 kN/m^3. The average unit weight of the soil column is 19.53 kN/m^3 and

the COV of unit weight of the soils is 0.042. A detailed depth-wise summary of the basic CPT data interpretation results is shown in Figure 78.
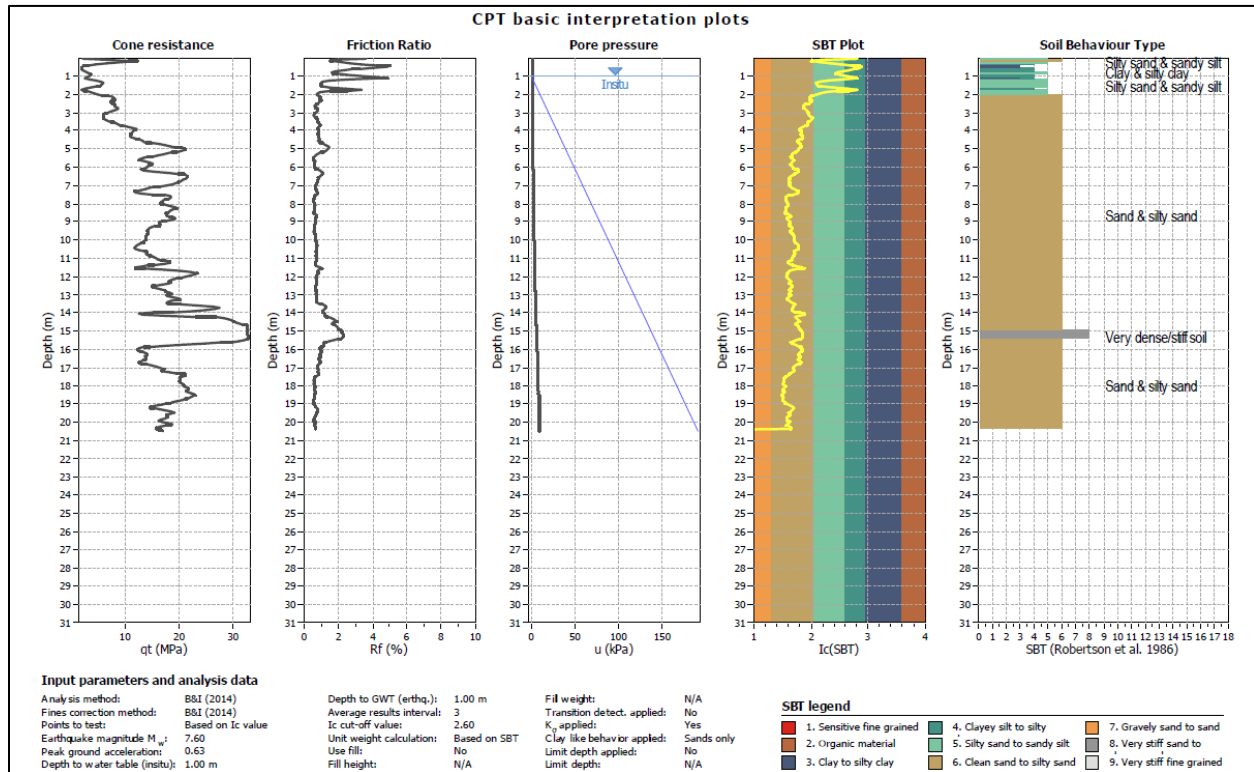
Figure 78. Basic Interpretation of CPT Data of CGD016.

Figure 79 shows the CPT based SBT classification chart and liquefaction triggering curve of based on the liquefaction analysis for the CPT sounding site, CGD016. The centroid of the soils metrics in CPT based SBT classification chart is 1.02% and 150.55. The normalized friction ratio has a range of 0.55 to 5.06%. The COV of the data points is 0.68. Similarly, for the normalized CPT penetration resistance, the range is 30.61 to 289.17, which is comparatively low in comparison with the other locations within the county. The COV of the resistance is 0.35, which is consistent in terms of indicating the variability in other locations within the county. For the data points based on the upper 5 m of the soil column, the centroid of the chart has become 1.47% and 119.64, which indicates the shifting of datasets towards the fourth quadrants of the chart. The datasets representing these layers lie in the upper part of the region for sand mixtures and silt mixtures, adjacent to Zone B of the chart. The average values of the normalized friction ratio and CPT penetration resistance for the lower 15 m of the soil are 0.87 % and 160, which imparting in the major shifting towards the second quadrant, towards normally consolidated sands, of the classification chart. In the liquefaction triggering curve in Figure 78, the coordinates of the centroid of the data points are 160.56 and 0.74, which indicates a significant portion of these soil layers are prone to liquefaction. The average values of the equivalent clean-sand value of the normalized cone penetration resistance and modified cyclic stress ratio for the upper 5 meters are 129.17 and 0.85, which shows all most all parts of these 5 m of soils trigger liquefactions. The centroid of the data points generated based on the soil layers below 5 m are 170.69 and 0.70, these points are also mostly in the liquefaction zone side of the liquefaction triggering curve. For this location, the upper

segment of the soil column is highly susceptible to liquefaction and the lower parts also contribute to triggering it.
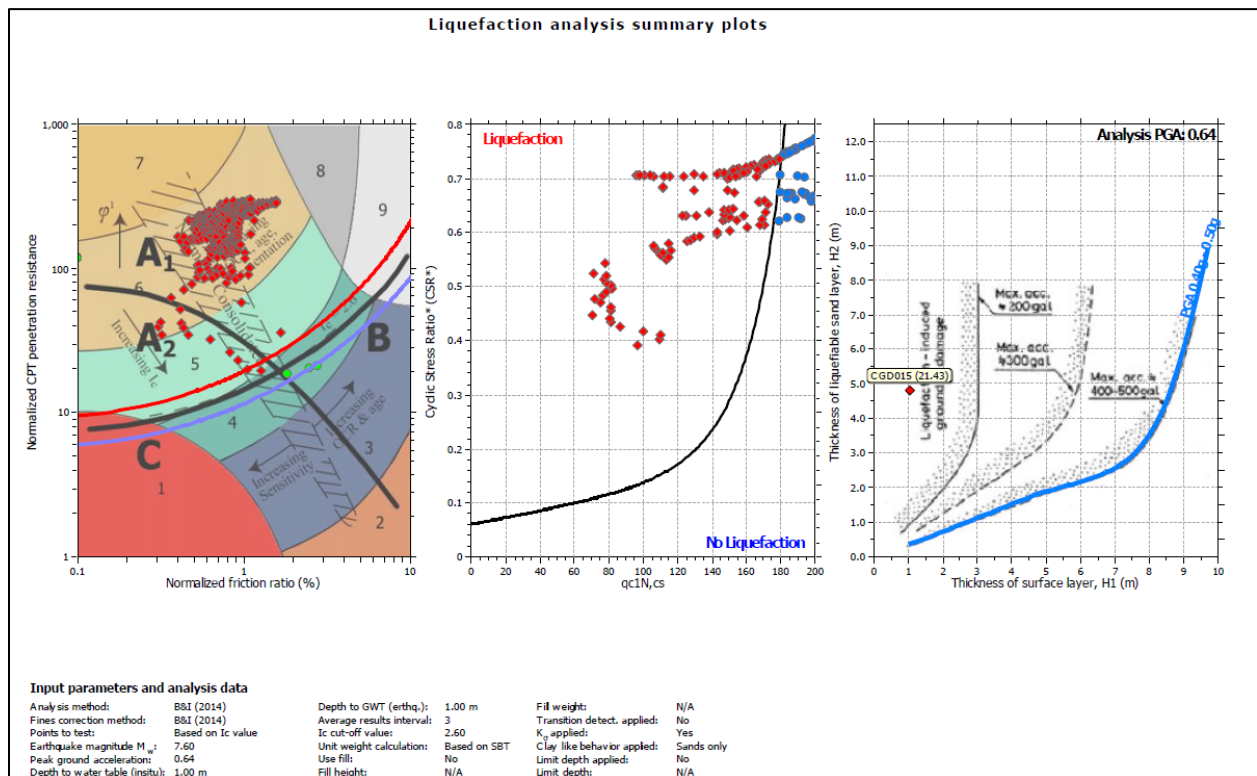

Figure 79. Summary Plots of the Liquefaction Analysis of Sounding CGD016.

According to the cyclic liquefaction analysis results, the CSR is greater than CRR for approximately several different layers within the total depth of the CPT sounding. Depth ranges of the soil layers with a probability of failure greater than 1.0 are 1.0 to 4.8 m, 9.0 to 11.8 m, 12.0 to 14.0 m, and 16.0 to 20.5 m. The overall LPI for the site is around 34.967, which indicates a very high risk of liquefaction. The LSN value of this location is 52.672, which indicates the possibility of severe damage to the upper layer of the soils during the design ground motion. For the given criteria, the ground surface of the site is prone to a vertical settlement of 29.619 cm and a lateral displacement of 176.50 cm after earthquakes. The overall probability of liquefaction of the site is 98.93%. The depth-wise distribution liquefiable layers of the site along with different scenarios of liquefaction potential, probability of liquefaction, vertical settlements, and lateral displacements is shown in Figure 80.
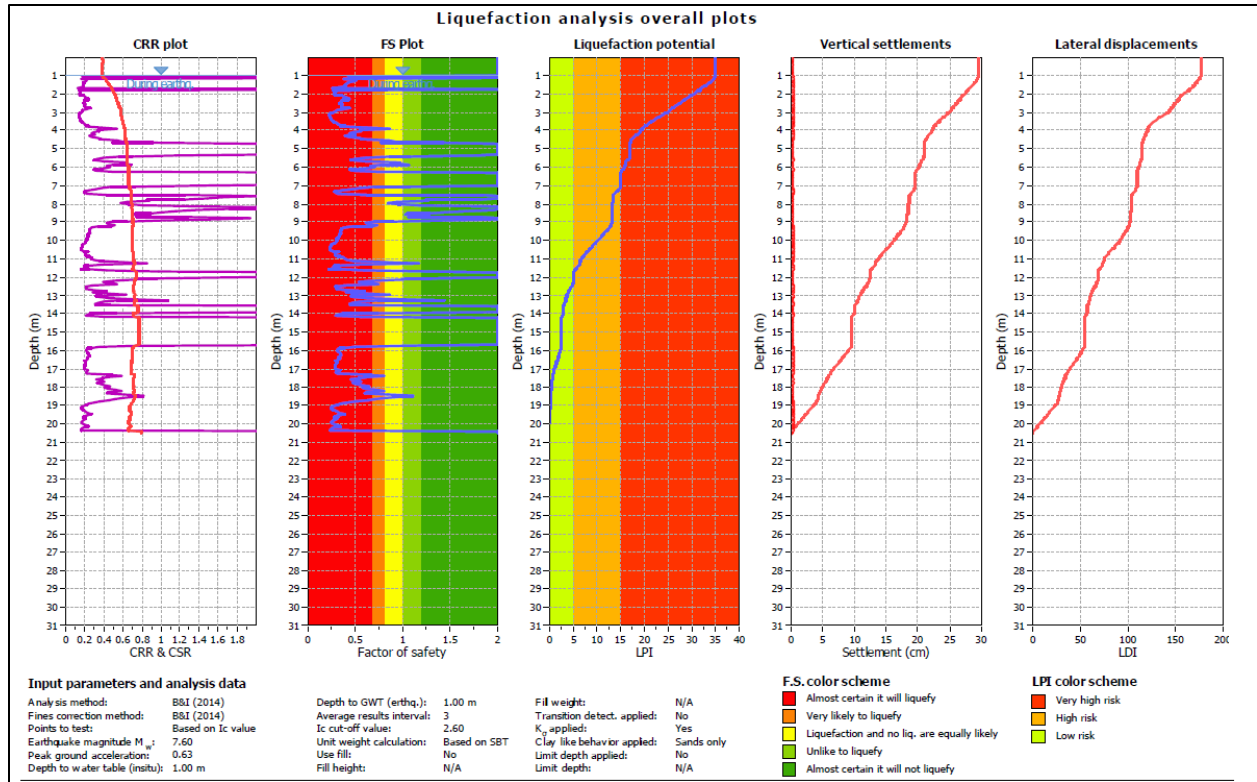
Figure 80. Overall Liquefaction Analysis Results of CGD016.

CGD017

This CPT Sounding is also located at the east of Highway 158 connecting Lunsford and Southland. The SBT Analysis shows all the other soils within the first 20 m are sand and silty sand, except a very thin silty sand and sandy silt layer, presents at a depth of 1.4 m. The total depth of the sounding is 20.4 m. The cone tip resistance is comparatively less up to the depth of 5.0 meters, and almost consistent at higher depth. The average I_c of the soil column is 1.69. The soil layers below 2 m are all sand and silty sand. The percent fine content (FC) of the upper 5 m of soils varies from 3.7% to 11.52%, with an average of 7.28% and COV of 0.25. The FC in other layers below the depth of 5 m approximately varies from 2.37% to 11.56%, with an average of 5.65% and COV of 0.25. The unit weights of soil vary from 16.59 kN/m^3 to 21.50 kN/m^3. The average unit weight of the soil

column is 19.51 kN/m^3, and the COV of unit weight of the soils is 0.041. A detailed depth-wise summary of the basic CPT data interpretation results is shown in Figure 81.
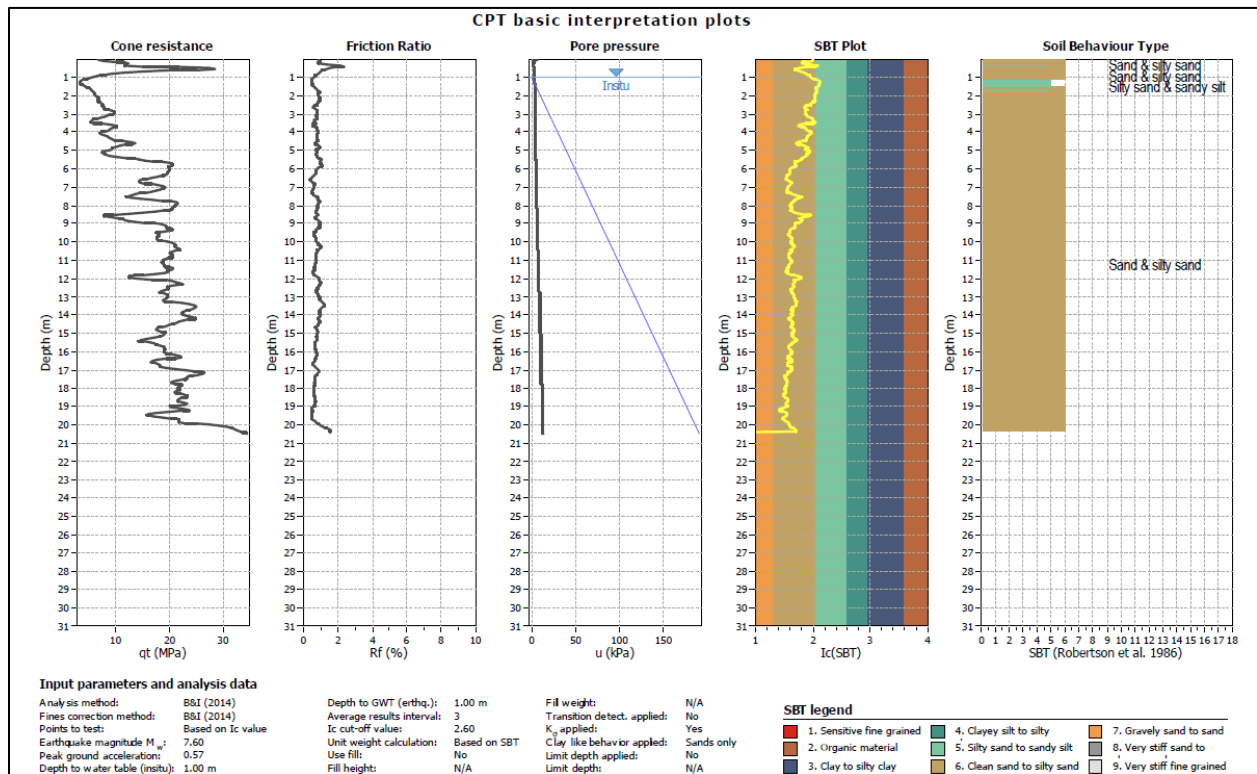
Figure 81. Basic Interpretation of CPT Data of CGD017.

Figure 82 shows the CPT based SBT classification chart and liquefaction triggering curve of based on the liquefaction analysis for the CPT sounding site, CGD017. The centroid of the soils metrics in CPT based SBT classification chart is 0.79% and 162.22. The normalized friction ratio has a range of 0.39 to 2.33%. The COV of the data points is 0.27. Similarly, for the normalized CPT penetration resistance, the range is 57 to 475.86, which is comparatively higher than the other locations within the county. The COV of the resistance is 0.29, which indicates better consistency of stiffness throughout the soil layers. For the data points based on the upper 5 m of the soil column, the centroid of the chart has become 0.83% and 142.40, which indicates the shifting of datasets towards the fourth quadrants or towards region 5 of the chart. This shifting also signifies increment of OCR, age, and cementation and shifting towards lower sensitivity of within region 6 of the chart. The average values of the normalized friction ratio and CPT penetration resistance for the lower 15 m of the soil are 0.78 % and 168.67, which imparting in the major shifting towards the second quadrant, towards the normally consolidated sands, of the classification chart. In the liquefaction triggering curve in Figure 82, the coordinates of the centroid of the data points are 168.46 and 0.64, which indicates significant parts of the soil are prone to liquefaction. The average values of the equivalent clean-sand value of the normalized cone penetration resistance and modified cyclic stress ratio for the upper 5 m are 135.06 and 0.80, which indicates a higher risk of liquefaction of these upper layers of soils. On the other hand, the centroid of the data points generated based on the soil layers below 5 m are 179.24 and 0.64, these points are in no liquefaction zone the liquefaction triggering curve. Similarly, for the upper 10 m, normalized cone penetration resistance, and modified cyclic stress ratio are 157.681 and 0.71 and for the lower 10

m, the values are 178.73 and 0.66, these indicate a higher probability of triggering liquefaction by the upper 10 m. The average FS against liquefaction for the upper 5 m is 0.71, for the upper 10 m is 1.006, and for total depth, the FS is equal to 1.083.
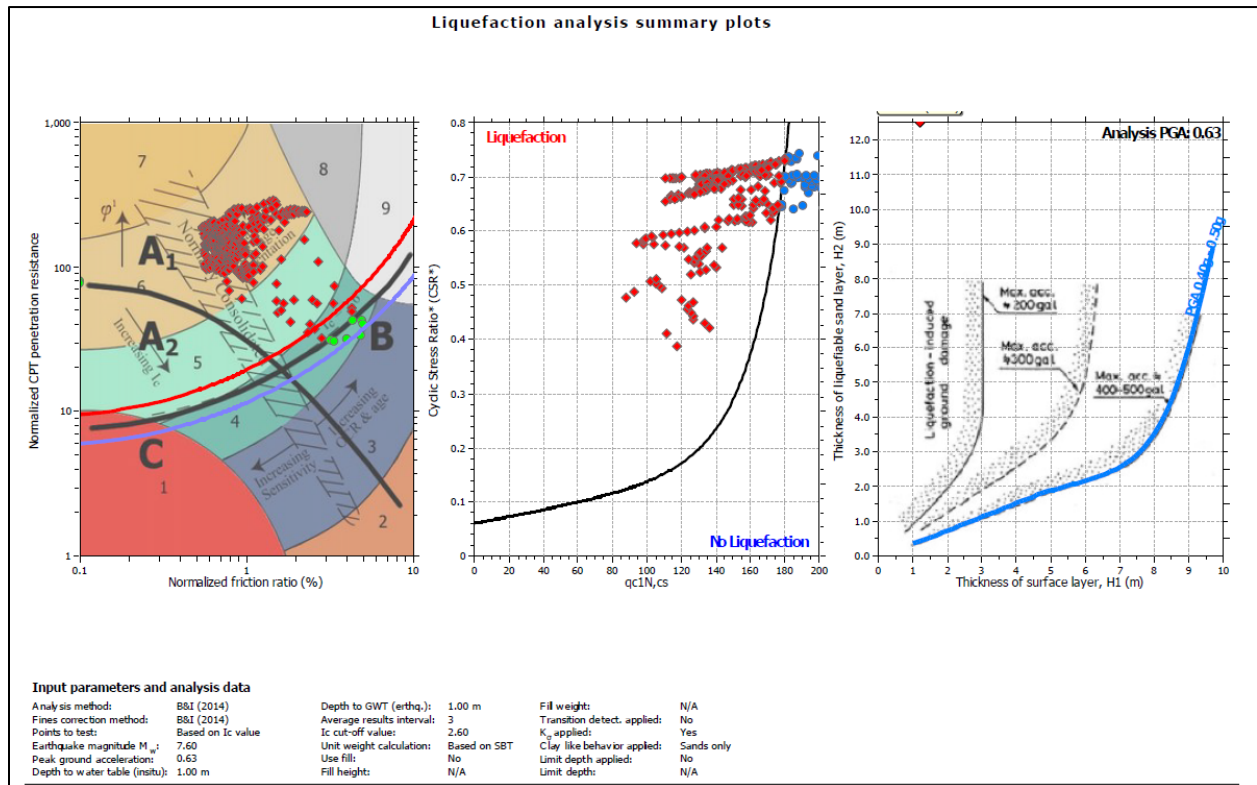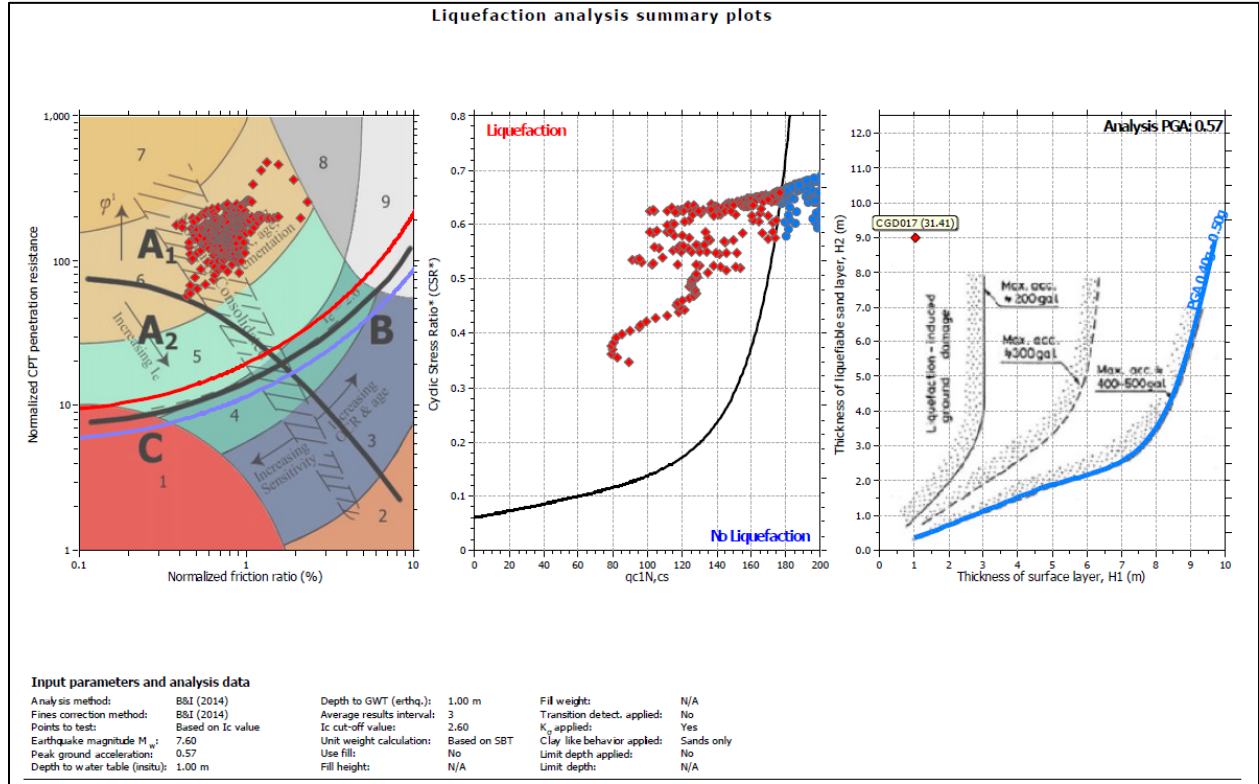

Figure 82. Summary plots of the liquefaction analysis of sounding CGD017.

According to the cyclic liquefaction analysis, the CSR is greater than CRR for several thin layers and two thick layers within the sounding depth. The depth ranges of the two thick soil layers with an FS less than 1.00 are 1.0 to 5.8 m, and 14.5 to 17.0 m. The overall LPI for the site is around 31.409, which indicates a very high risk of liquefaction. The LSN value of this location is 58.732, which insinuates severe damage to the upper layer of the soils. This site has the highest risk of the surface level of damage in comparison with the other sites considered within this county for this research. For the given criteria, the ground surface of the site is prone to a vertical settlement of 25.242 cm and lateral displacement of 166.138 cm after earthquakes. The overall probability of liquefaction of the site is 97.714%. The depth-wise distribution liquefiable layers of the site along with different scenarios of liquefaction potential, probability of liquefaction, vertical settlements, and lateral displacements is shown in Figure 83.
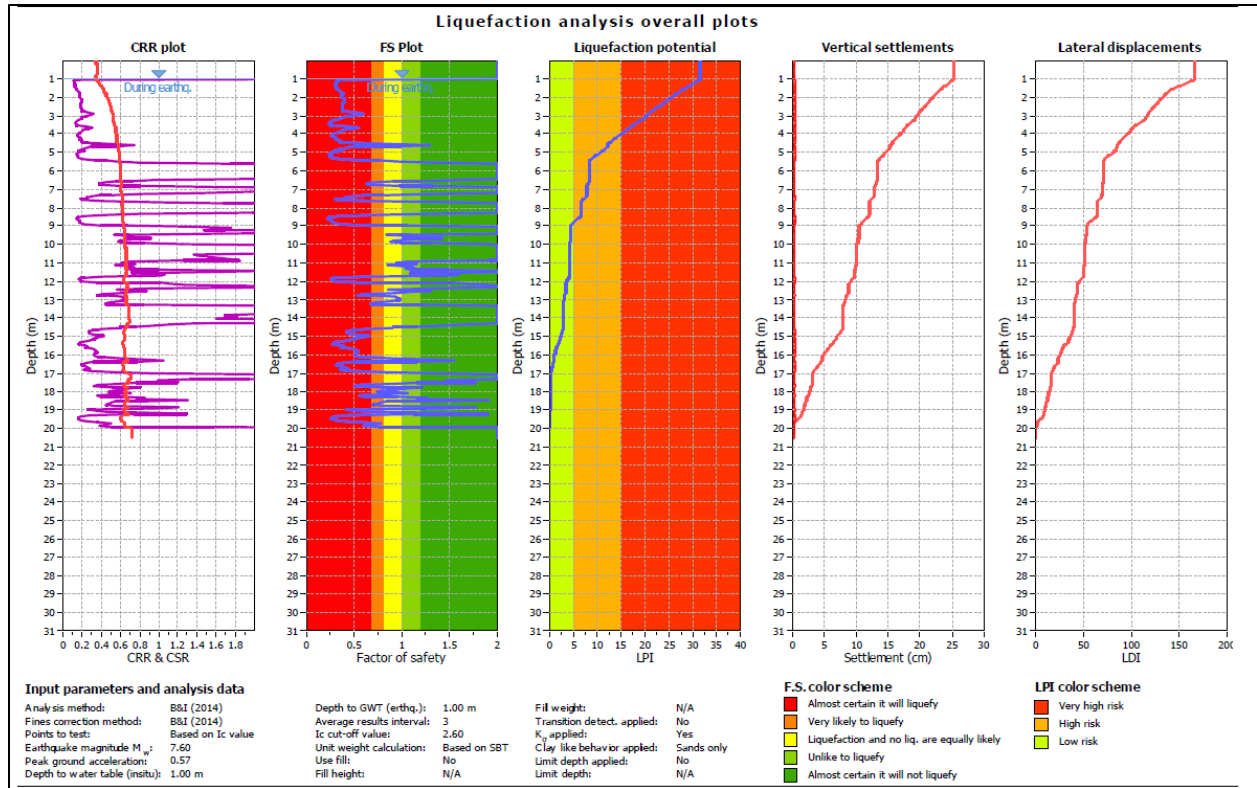
Figure 83. Overall Liquefaction Analysis Results of CGD017.

CGD018

This CPT Sounding is located at Lake View beside AR135-N. The location is on the west side of Saint Francis River. The SBT Analysis shows the presence of one-meter thick silty sand and silty sand layer on top of 20 m thick sand and silty sand layer. The cone tip resistance is comparatively low up to the depth of 3.0 m and varies significantly at higher depth. The average I_c of the soil column is 1.81. The soil layers below one meter are all sand and silty sand. The percent fine content (FC) of the upper 5 m of soils varies from 2.97% to 17.34%, with an average of 7.73% and COV of 0.39. The FC in other layers below the depth of 5 m approximately varies from 4.22% to 17.53%, with an average of 8.6% and COV of 0.27. The unit weight of soils varies from 17.03 kN/m^3 to 20.99 kN/m^3. The average unit weight of the soil column is 19.49 kN/m^3 and the COV

CPT basic interpretation plots

Cone resistance	Friction Ratio	Pore pressure	SBT Plot	Soil Behaviour Type

(Insitu marked on Pore pressure plot)

Soil Behaviour Type labels: Silty sand & sandy silt; Silty sand & sandy silt; Sand & silty sand; Silty sand & sandy silt; Sand & silty sand

x-axes: qt (MPa); Rf (%); u (kPa); Ic(SBT); SBT (Robertson et al. 1986)

y-axis: Depth (m)

Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	1.00 m	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	Kσ applied:	Yes
Earthquake magnitude M_w:	7.60	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.63	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	1.00 m	Fill height:	N/A	Limit depth:	N/A

SBT legend

1. Sensitive fine grained
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravely sand to sand
8. Very stiff sand to
9. Very stiff fine grained

Figure 84. Basic Interpretation of CPT Data of CGD018.

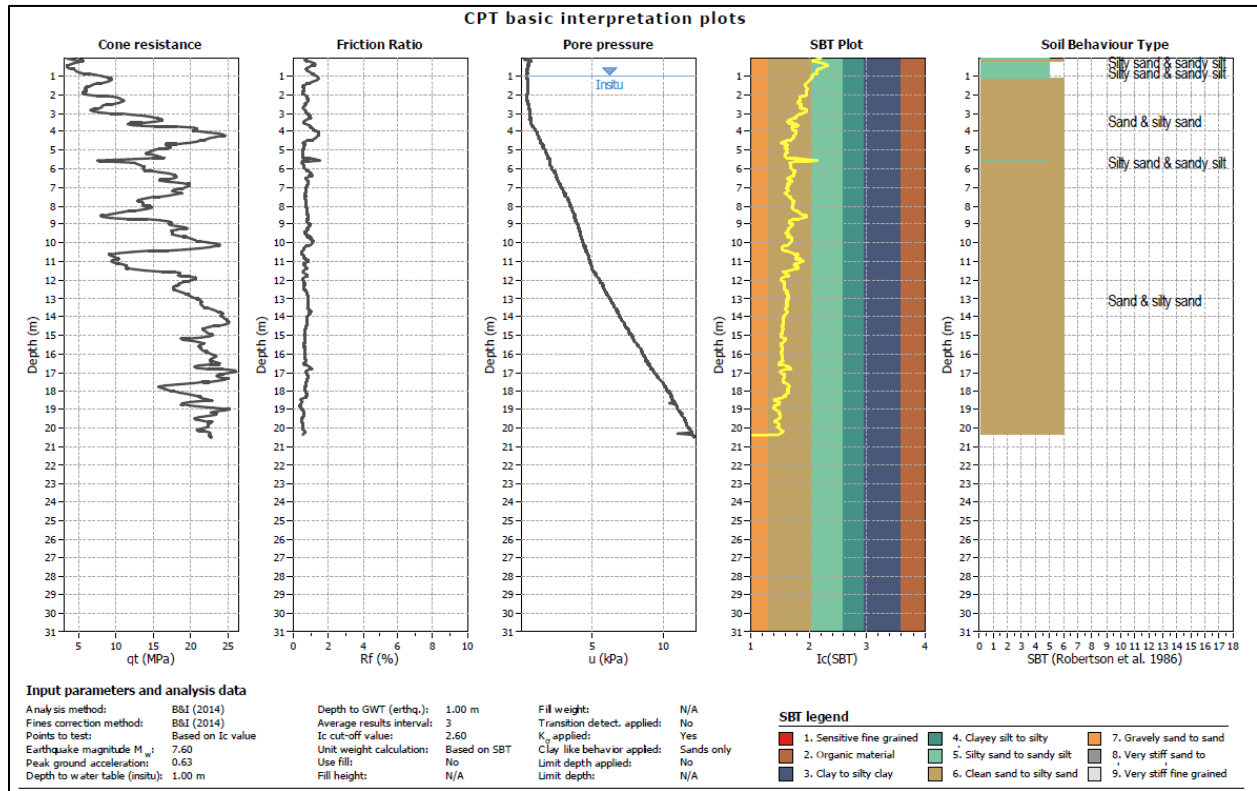Figure 85, below, shows the CPT based SBT classification chart and liquefaction triggering curve of based on the liquefaction analysis for the CPT sounding site, CGD018. The centroid of the soils metrics in CPT based SBT classification chart is 0.77% and 115.21. The normalized friction ratio has a range of 0.39 to 1.59%. The COV of the data points is 0.26, which indicates less variability in comparison with the other locations within the county. Similarly, for the normalized CPT penetration resistance, the range is 51.3 to 277.1. The COV of the resistance is 0.36. For the data points based on the upper 5 meters of the soil column, the centroid of the chart has become 0.89% and 148.64, which indicates the shifting of datasets towards the first quadrants, towards region 8 in the chart, which indicates increased OCR, age and cementation property and shifting towards lower sensitivity of soils in region 6 of the chart. The average values of the normalized friction ratio and CPT penetration resistance for the lower 15 meters of the soil are 0.74 % and 104, which imparting in the major shifting towards the third quadrant of the classification chart. However, the coefficient of variation of these two parameters below 5 m is 0.21 and 0.26, which indicates the presence of similar types of soils. In the liquefaction triggering curve in Figure 85, the centroid of the data points is: 143.59 and 0.48, which indicates major parts of the soils in the total column are prone to liquefaction. The average values of the equivalent clean-sand value of the normalized cone penetration resistance and modified cyclic stress ratio for the upper 5 m are 151.11 and 0.71, which indicates to trigger liquefaction. On the other hand, the centroid of the data points generated based on the soil layers below 5 m are 141.16 and 0.41, these points are also mostly in the liquefaction zone side of the liquefaction triggering curve. In summary, the total soil column highly

triggers liquefaction. The analysis of the FS against liquefaction shows the values are 1.3, 0.7 for the upper 5 m, and subsequent layers. Similarly, the FS against liquefaction values are 0.99 and 0.71 for the upper ten meters and lower ten meters. So, the selection of foundation layers to avoid liquefaction is difficult for this location and area. The replacement of upper layers will not improve the performance of the foundation. Other ground improvement techniques will be required.
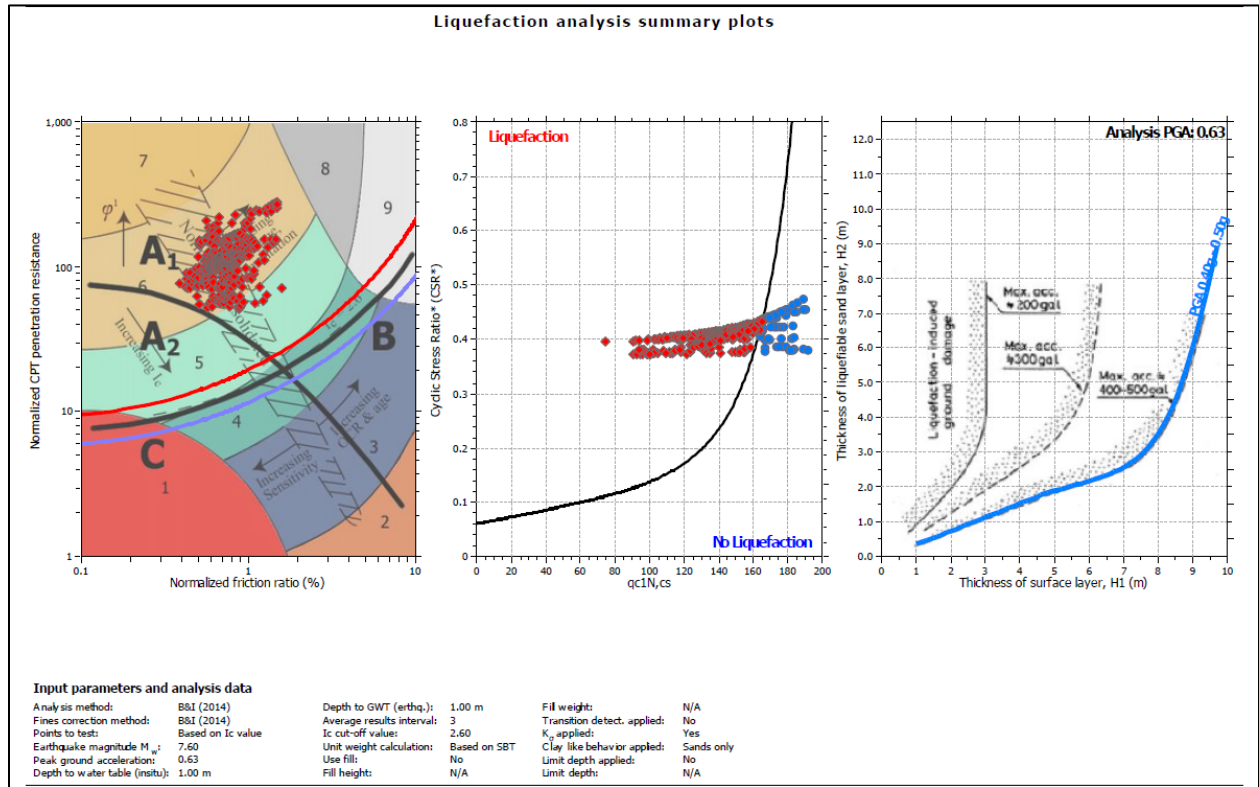
Figure 85. Summary Plots of the Liquefaction Analysis of Sounding CGD018.

According to the cyclic liquefaction analysis, the FS against liquefaction is less than 1.0 for almost throughout the depth, except an approximately 1.0-meter-thick layer at a depth of 4.5 m. The overall LPI for the site is around 29.469, which indicates a very high risk of liquefaction. The LSN value of the location is 53.27, which indicates the possibility of severe damage to the upper layers of the soils. The ground surface of the site is prone to a vertical settlement of 35.78 cm, and lateral displacement of 230.78 cm after earthquakes. This site is susceptible to experience the highest vertical settlement and lateral displacement among all the sites considered for analysis within Craighead County. The site has an overall probability of liquefaction of 96.55%. The depth-wise distribution of the liquefiable layers of the site along with different scenarios of liquefaction potential, probability of liquefaction, vertical settlements, and lateral displacements are shown in Figure 86.
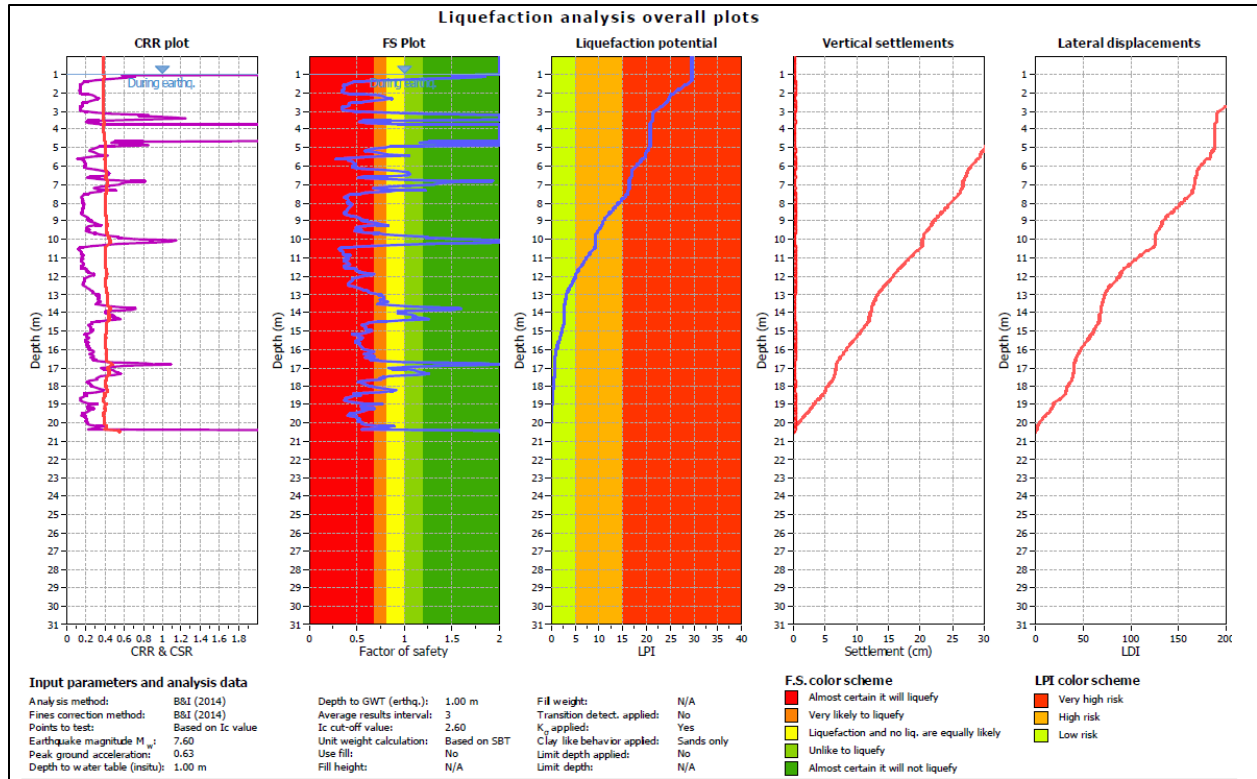
Figure 86. Overall Liquefaction Analysis Results of CGD018.

CGD019

This CPT Sounding is located at the southeast corner of the county near Lepanto Junction. The SBT Analysis shows the presence of a clay layer in the middle of two silty sand and sandy silt layers within the top 1.5 m of the soil column. Under these layers, approximately 12-m thick sand and silty sand layers exist. In the soils starting from a depth of 10.5 meters to 20.5 meters, several sand layers exist which are present as sheet inside each other. The cone tip resistance increases with depth. The average I_c of the soil column is 1.50. The percent fine content (FC) of the upper 5 m of soils varies from 0.97% to 44.98%, with an average of 9.18% and COV of 1.31. The FC in other layers below the depth of 5 m approximately varies from 0% to 7.49%, with an average of 2.0% and COV of 0.68. The unit weight of soils varies from 15.38 kN/m^3 to 21.41 kN/m^3. The average unit weight of the soil column is 18.75 kN/m^3 and the COV of unit weight of the soils is

0.05. A detailed depth-wise summary of the basic CPT data interpretation results is shown in Figure 87.
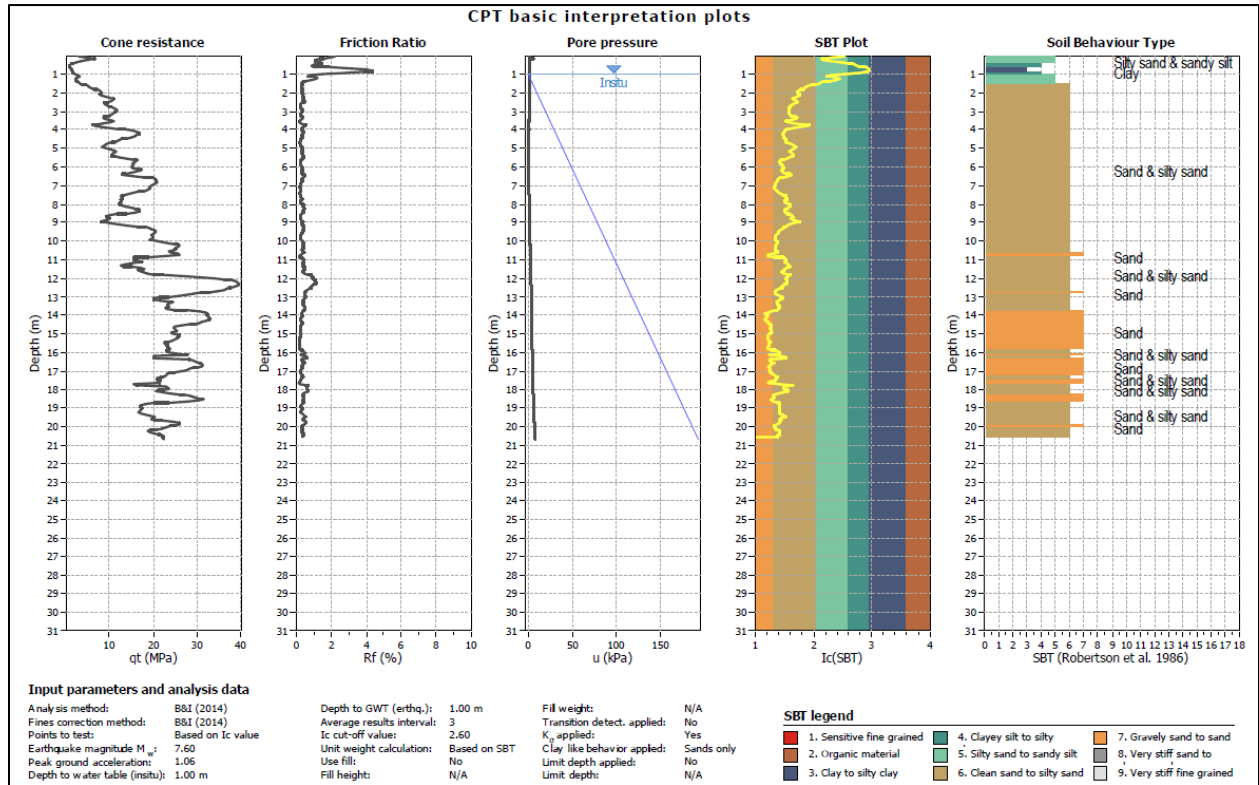

Figure 87. Basic Interpretation of CPT Data of CGD019.

Figure 88 shows the CPT based SBT classification chart and the liquefaction triggering curve of based on the liquefaction analysis for the CPT sounding site, CGD019. The centroid of the soils metrics in CPT based SBT classification chart is 0.47% and 178.82. The normalized friction ratio has a range of 0.17 to 4.42%. The COV of the data points is 1.002. Similarly, for the normalized CPT penetration resistance, the range is 15.26 to 365. The COV of the resistance is 0.37. For the data points based on the upper 5 m of the soil column, the centroid of the chart has become 0.73% and 121.144, which indicates the shifting of datasets towards the fourth quadrants, towards Zone B and the soil behavior type 5 and 4. It also signifies increment of OCR, age, and cementation and shifting towards less sensitive soils. The average values of the normalized friction ratio and CPT penetration resistance for the lower 15 m of the soil are 0.39 % and 197.36, which imparting in the major shifting towards the second quadrant of the on the classification chart. So, the soil layers below 5 m are mostly normally consolidated clean sand to silty sand. In the liquefaction triggering curve in Figure 88, the coordinates of the centroid of the data points are 180.39 and 1.22, which indicates a significant portion of the soil triggers liquefaction. The average values of the equivalent clean-sand value of the normalized cone penetration resistance and modified cyclic stress ratio for the upper 5 m are 129.60 and 1.17, which confirms the possibility of severe liquefaction. On the other hand, the centroid of the data points generated based on the soil layers below 5 m are 196.566 and 1.24, these parts of soils have several points in the "No liquefaction" side of the liquefaction triggering curve.
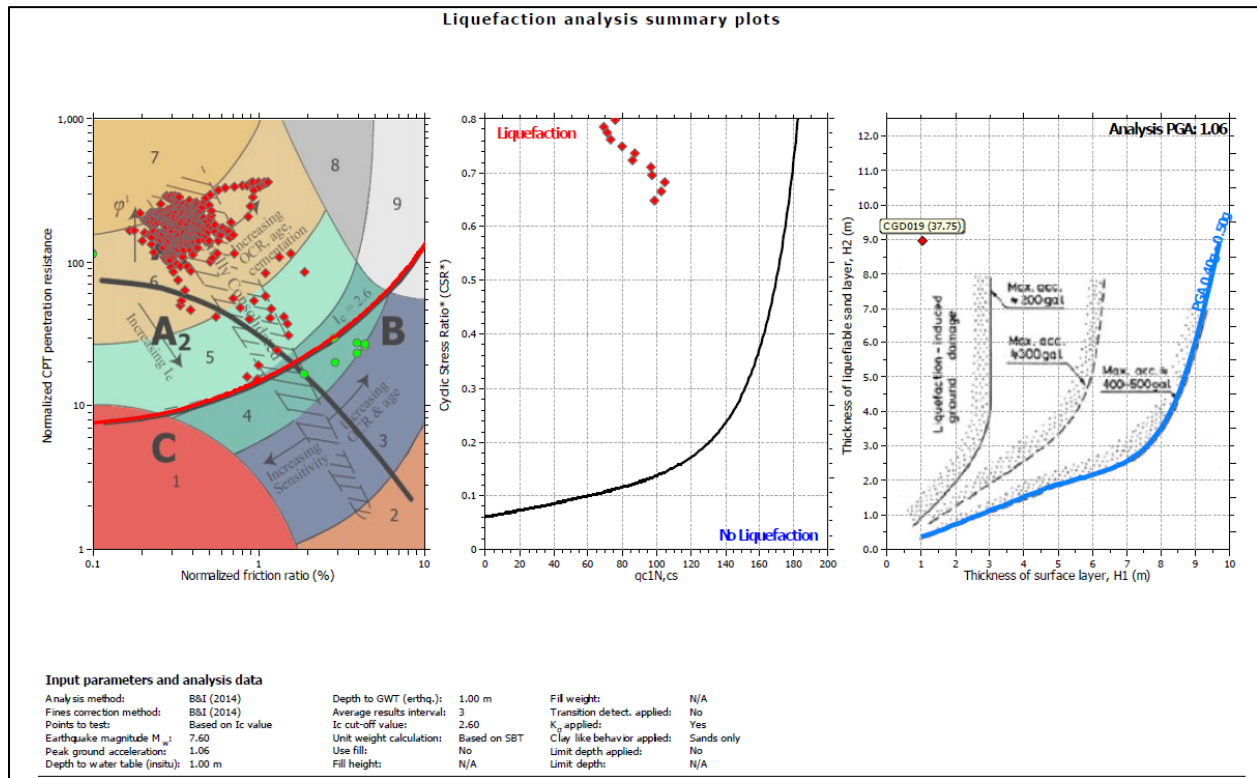
Figure 88. Summary Plots of the Liquefaction Analysis of Sounding CGD019.

According to the cyclic liquefaction analysis, the CSR is greater than CRR for approximately five different layers within 20.5 m. The depth ranges of the soil layers with an FS less than 1.0 are 1 to 4.2 m, 4.5 to 6.0 m, 7.1 to 9.2 m, 11.0 to 12.0 m, 17.0 to 18.0 m, and 19.0 to 20.8 m. The overall LPI for the site is around 37.752, which indicates a very high risk of liquefaction. The value is comparatively higher than most of the sites within the county. The LSN value of the location is 50.76, which insinuates the possibility of severe damage to the ground surfaces in these locations. For the given criteria, the ground surface of the site is prone to a vertical settlement of 21.143 cm and a lateral displacement of 149.42 cm after earthquakes. The overall probability of liquefaction of the site is 99.42%. The depth-wise distribution liquefiable layers of the site along with different scenarios of liquefaction potential, probability of liquefaction, vertical settlements, and lateral displacements is shown in Figure 89.
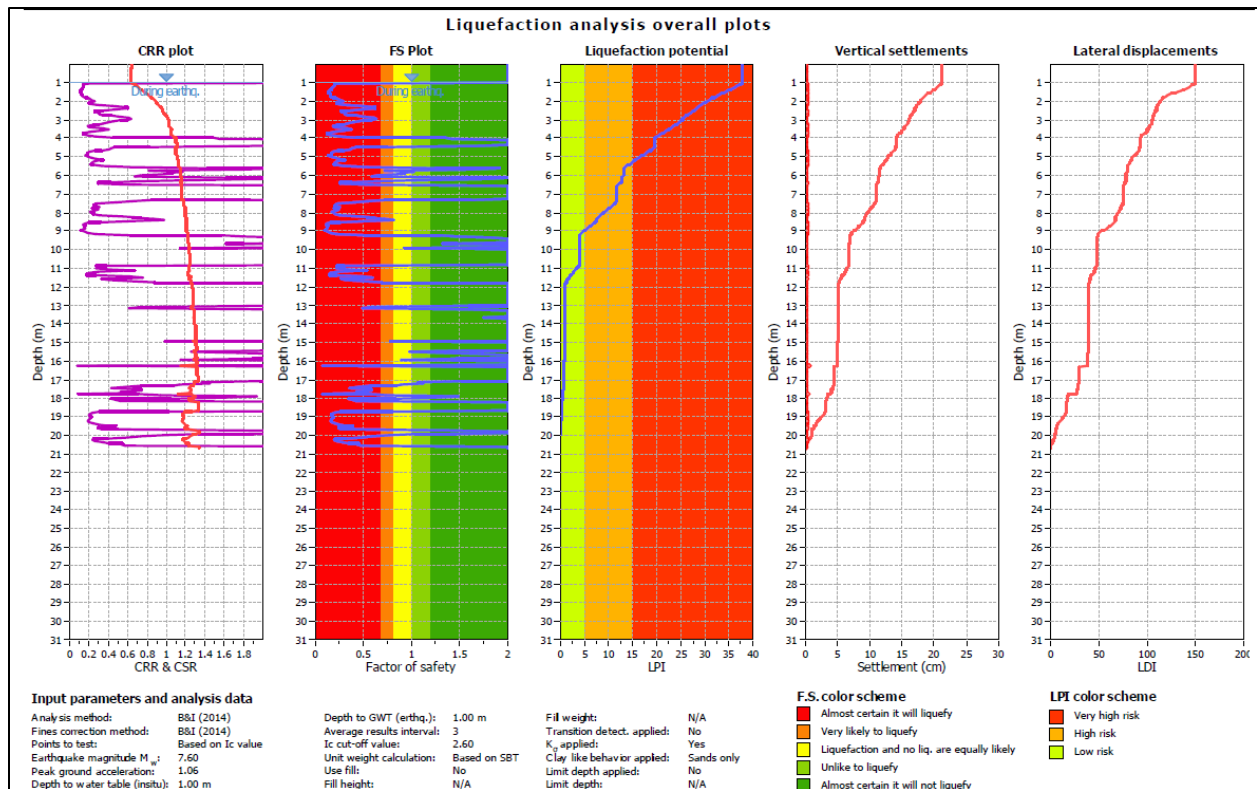
113

Figure 89. Overall Liquefaction Analysis Results of CGD019.

Analysis of all the sites inside the county shows that all the sites are at a very high risk of liquefaction. Concerning LSN, about 50% of the locations of the analysis area are susceptible to severe liquefaction hazards, 34% of the locations are suspected to have major liquefaction hazards, 8% percent of the locations have moderate liquefaction hazards, and the rest of the locations have minor liquefaction hazards.

Summary for NEA

Mississippi, Poinsett, Cross, Crittenden, Lee, and St. Francis Counties also lie within Northeast Arkansas. Therefore, in addition to 17 sites from Clay, Green, and Craighead counties, data of 38 sites from Crittenden County, 8 sites from Cross County, 52 sites from Mississippi County, 10 sites from Poinsett County, 4 sites in Lee County, and 2 more in St. Francis from County were analyzed for cyclic liquefaction resistances. The SBT and cyclic liquefaction analysis results of these additional 114 sites are provided with the *Project Data* of this project. The results of all 131 sites are summarized based on the actual locations of the sites and corresponding LPI, LSN, vertical settlement, lateral displacement, and the overall probability of liquefaction. The maps for these parameters with graduated symbols are shown in Figures 90 through 94.
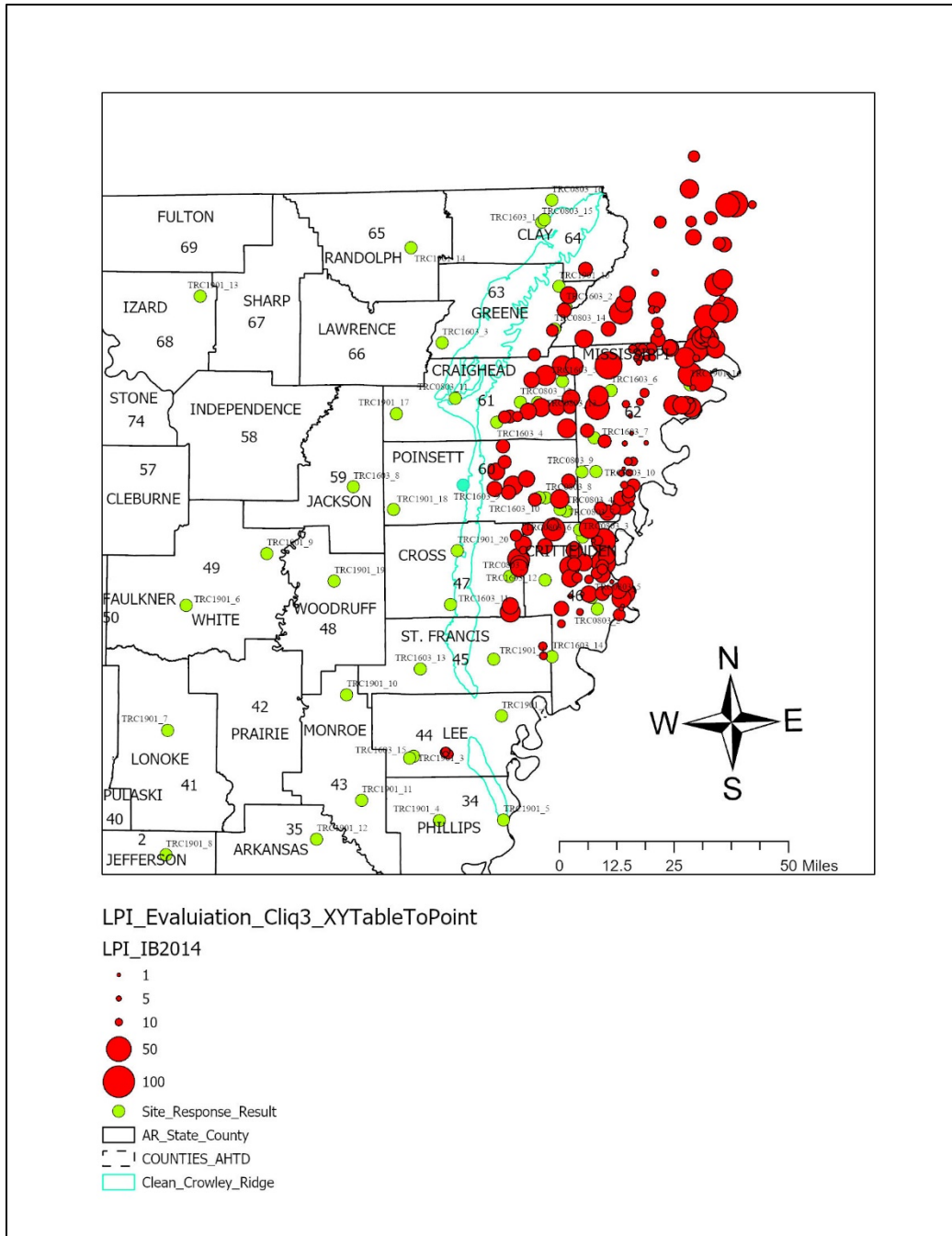
Figure 90. Liquefaction Potential Index for Different Locations in NEA.

Figure 90 shows the spatial distribution of liquefaction potentials around different counties in northeast Arkansas. Graduated red circles show the increment of LPI with the increase of diameters of the balls. The diameter of the circle indicates the LPI increases with toward the east from the Crowley's Ridge area from NEA. However, the LPI values get reduced towards the middle portion, which indicates a huge area with lower liquefaction potentials in the middle of Mississippi County. Similarly, Figure 91 shows the distribution of LSN over the NEA. The yellow graduated circles show the values of LSN on the location. The analysis shows a strip of area passes through the Mississippi County has very low LSN values, which indicate the prospects of lower risk of surface

damages. This strip has less risk of surface damage during earthquakes. The selection of heavy loaded structures and highways in these routes can expect less surface damage.
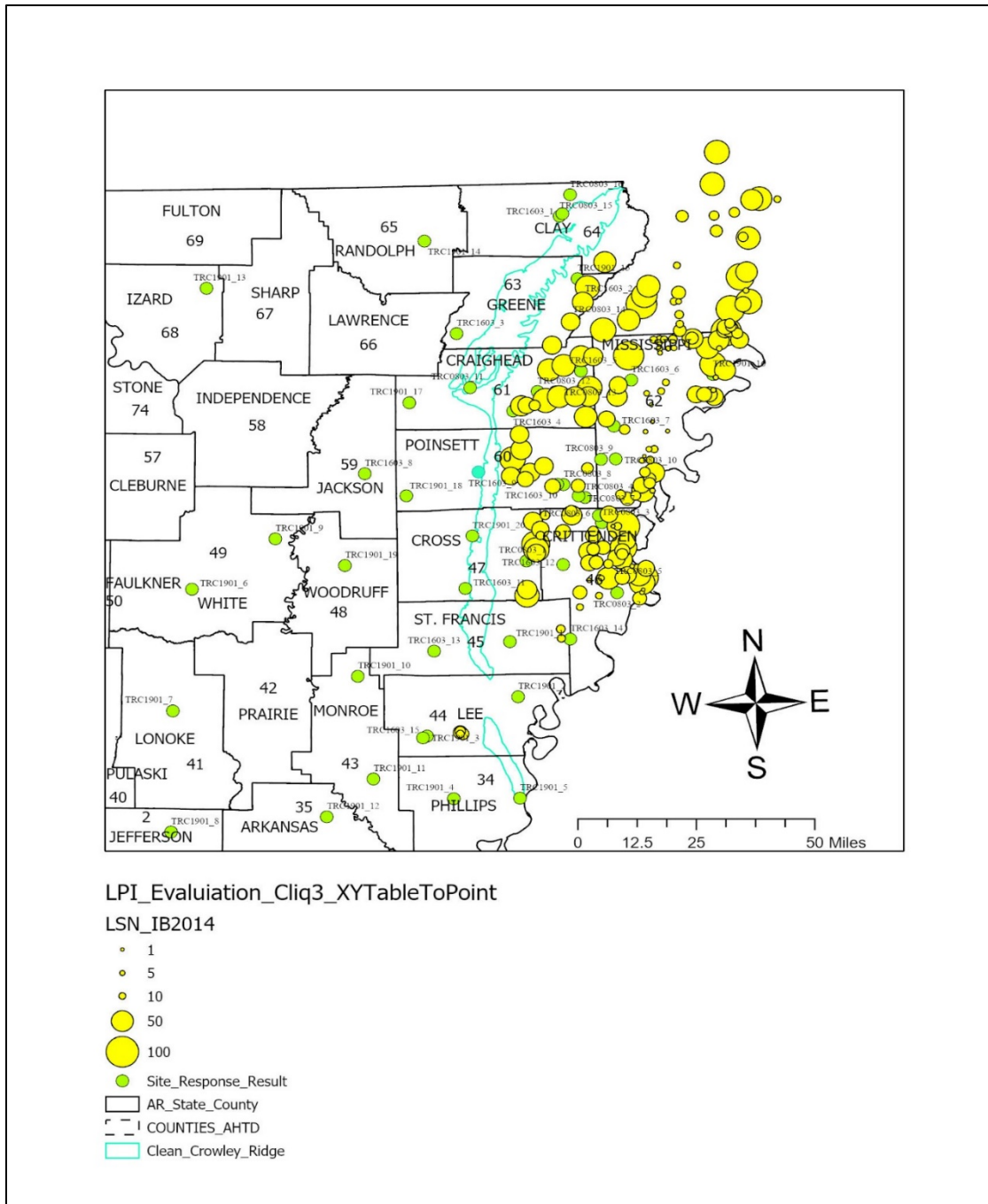
Figure 91. Liquefaction Severity Number Graduated With Numbers for NEA.

Vertical settlements are shown similar trends in Figure 92. Estimated values of vertical settlements of those locations based on the considered ground motion are mapped with graduated circles. Areas with less probable vertical settlements are marked with smaller circles. Figure 93 shows the estimated values of lateral spreading or displacements of these locations based on the considered ground motion. Finally, Figure 94 shows the overall probability of liquefaction of different points

in NEA. Some of the effects of liquefaction are self-explanatory in terms of risk and damages, all five parameters act proportionately with the risk and damages.

Figure 92. Estimated Vertical Settlements for Different Locations in NEA.

Figure 93. Estimated Lateral Displacements for Different Locations in NEA.

Figure 94. Probability of Liquefaction of Soil Layers for Different Locations in NEA.

6. CONCLUSIONS

The ARDOT recommends using the AASHTO LRFD bridge design specifications for the seismic design of bridges. According to the definition of the Site Classes of AASHTO, most of the parts of NEA are within the site cla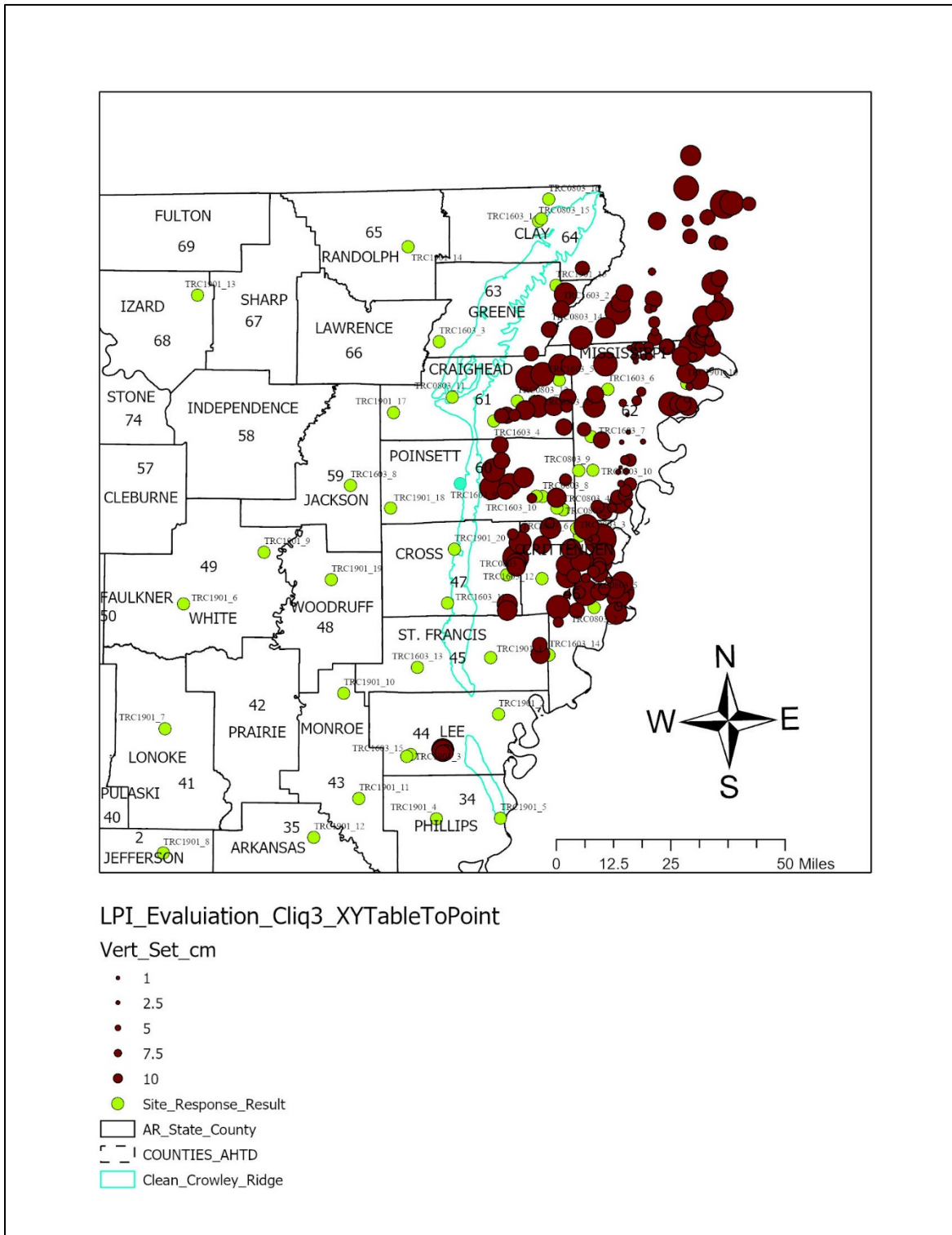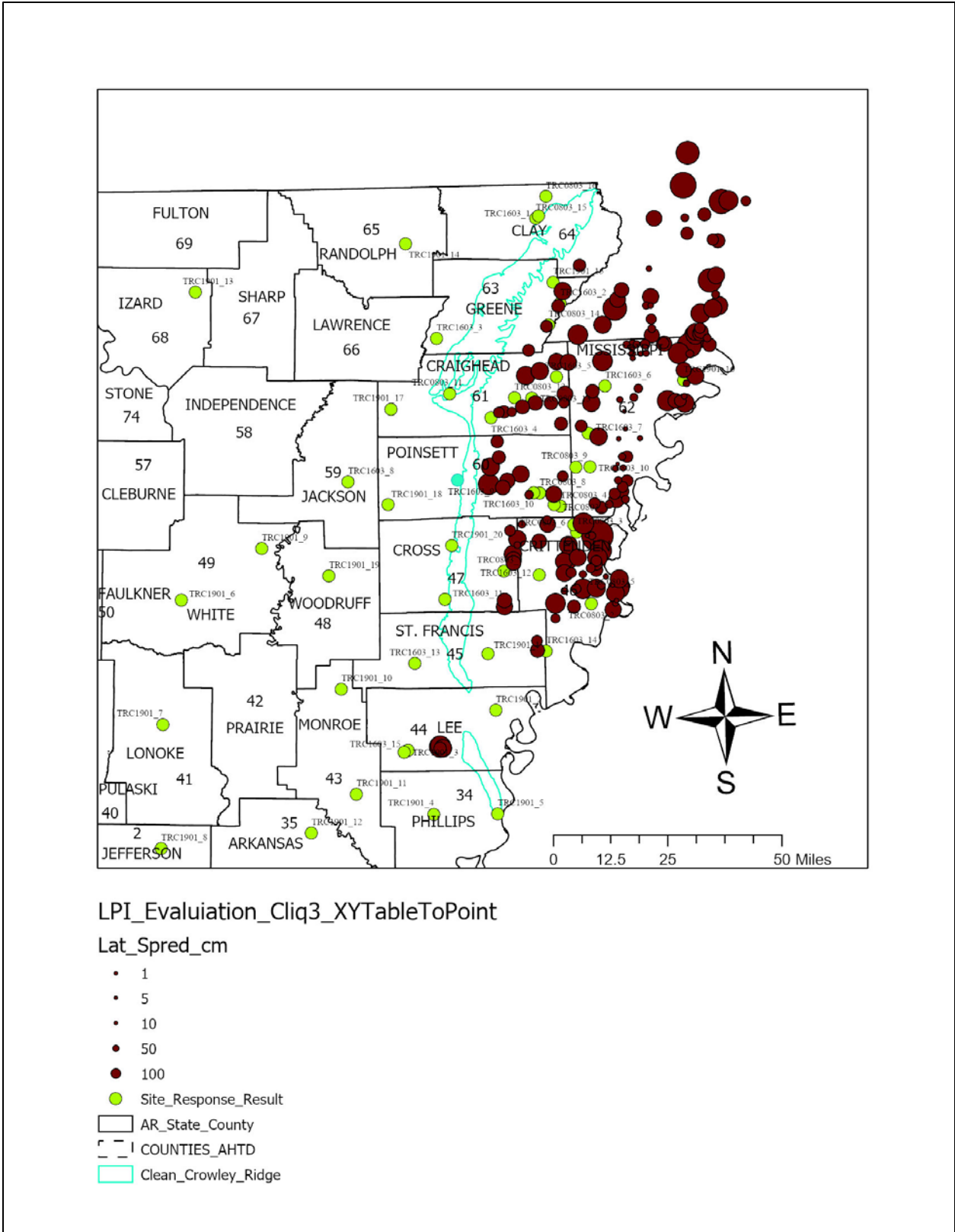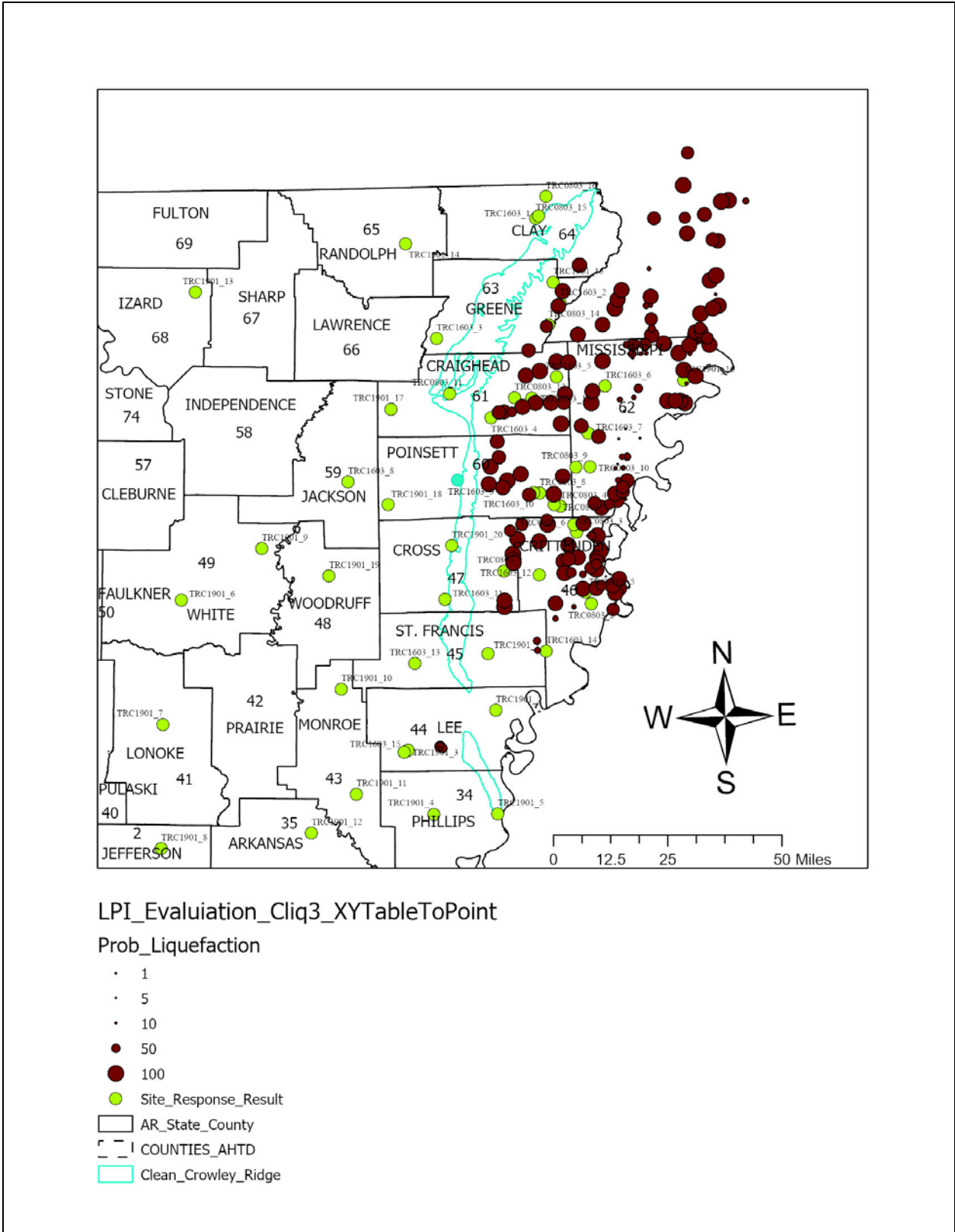ss 'F". In general, AASHTO established procedures for developing design response spectrums based on the Site Classes. However, for site class "F", the AASHTO does not recommend any value for developing the spectrum. Site specific analysis is suggested before developing the design spectrum. Additionally, liquefaction potential analysis is also required for each site. Previously, a significant number of efforts have been made to understand different factors associated with the determination of ground responses for different locations. Extensive geophysical investigations have been carried out at a certain interval for all the part of NEA. In this project, ground responses have been analyzed for five different sites near the city of Jonesboro based on the previous shear wave velocity profile analysis, liquefaction triggering. Afterward, seismic hazards were analyzed for 131 sites within the NEA, based on the available CPT along with other geological data.

Delineated design response spectra for five sites developed for selected matched ground motions with the target ground motions by performing seismic hazard analysis. Based on the upper 30m shear wave velocity (V_S) profile obtained from ARDOT's previous projects, these five sites were found as Site Class "D" instead of "F". The AASHTO suggests Seismic site factors such as PGA (0.01 sec), Fa (0.2 sec), Fv (1.0 sec) can be obtained from the AASHTO documented design spectrum for different Site Class. These factors may lead to the overdesigning of a structure at significant cost and under designing of the structure at significant risk. The AASHTO does not count the impact of deep deposited soft soil down to bedrock (i.e. ME embayment) for these recommended seismic factors. It is suggested to perform seismic site specific ground motion Response Analysis (SSGMRA) before designing structures such as bridges where deep deposited soft soil profile down to bedrock exists. Based on site specific analysis, these PGAs gets deamplified about 33% except Fontaine. Its PGA gets deamplified less than 33% compared to that of the AASHTO. Four out of Five site shows 33% reduction in short period (0.2 sec) design acceleration (SS) than that of the AASHTO. Fontaine shows a lower reduction of short period (0.2 sec) design acceleration (SS) compared to the other four sites. The probable reasons behind this exception are Fontaine has the lowest depth of soil profile down to bedrock and it has the lowest target spectrum (i.e. PGA, SS, S1) among five sites. As a result, seismic energy is not quickly released into the soil layer and this results in the amplification of the seismic design spectrum compared to the other four sites.

For Harrisburg, Manila, Fontaine, Bay, and Marked Tree the corresponding estimated PGAs are 0.69g, 1.222g 0.557g, 0.656g, and 1.119g. respectively. According to the analysis of this project, the reported values of Ss, S1, Fa, and Fv for Harrisburg are 1.025g, 0.801g, 1.894, and 4.5213, respectively. Shorter period structures (up to 0.345s) are seemed to be overdesigned if AASHTO design guidelines are followed, so significant cost can be saved in case of using short period bridges in this site. A similar analysis of for Manila shows the values of Ss, S1, Fa, and Fv as 1.633g, 1.369g, 1.6196, and 4.354, respectively. Shorter period structures (up to 0.775 sec) are seemed to be overdesigned if the AASHTO design guidelines are followed, so significant cost can be saved for this site based on the site specific analysis. This analysis results can also be applied to the nearest sites to this location. For Fontaine, the values of the coefficients are 1.009g, 0.55g,

and 3.063 and 4.6023, respectively. Similarly, the values of Ss, S1, Fa, and Fv for Bay are 0.984g, 0.764g, 1.885, and 4.455, respectively. That means bridges may be overdesigned up to 0.443 sec if the AASHTO documented values are used for this site. For Marked Tree, the values of Ss, S1, Fa, and Fv are 1.612g, 1.343g, 1.545, and 4.089, respectively. That means bridges may be overdesigned up to 0.502 sec if the AASHTO documented values are used for this site. On the other hand, spectral acceleration values are getting amplified after the above mention period than that of the AASHTO. Hence, it can be concluded that SSGMRA shows a lower acceleration of about a 33% reduction in design spectra for short period (0.1~0.5 sec) than that of the AASHTO. On the other hand, the SSGMRA shows amplification in design spectra for periods after 0.5 sec. In conclusion, the analysis results wide class of structures are getting overdesigned for short period (0.1~ 0.5 sec) in these areas, if we follow AASHTO LRFD design guidelines. Following the developed design response spectrums during replacement and construction of new bridges can save a significant amount of money for short period (0.1~ 0.5 sec).

Besides site specific ground response analysis for locations around the city of Jonesboro, 131 CPT sites were analyzed for liquefaction analysis under this project. CPT sounding results were analyzed and LPI, LSN and vertical settlement, lateral displacement, and overall probability of liquefaction have been estimated besides several other properties. Description of individual site specific liquefaction analysis results were included in the findings part of this report. A geographical information system-based summary of LPIs, LSNs, and site specific vertical settlements, horizontal settlements, and overall liquefaction were mapped for all the dispersed points in northeast Arkansas. The generated maps using these parameters show the spatial distribution of liquefaction potentials around different counties in northeast Arkansas. The most critical and comparatively safe zones in northeast Arkansas are shown in the generated maps. The findings of this study are expected to help ARDOT and other agencies in the region to incorporate liquefaction hazards in their designs and avoid catastrophic surface damages and triggering liquefaction in the event of a major earthquake in the region. Other uses of the maps are the installation of pipes lines and finding suitable foundation layers for safe structures.

In site specific ground response analysis, five sites were covered around the city of Jonesboro. However, a limited number of sites with detailed geotechnical data available inside of the city area. For example, there are very few data inside of the Jonesboro city area. So, city intensive geophysical investigations are required for these areas to analyze the ground response analysis and liquefaction analysis. Again, to ease up the ground response analysis, ground motion predictions need to be developed for easing up the ground response analysis process. The Pacific Earthquake PEER Center does not have enough data to select the most suitable ground motion for the NMFZ and the surrounding area. Emphasize should be made to enrich this database with locally extracted ground motion data recorded in different seismic stations. Additionally, the ARDOT should prepare a guideline for a better plan and analysis of regional earthquake engineering problems.

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