Penetration *vs.* Shear Wave Velocity for Evaluation of Liquefaction Potential in Northeast Arkansas

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Abstract: Shear wave velocity profiles at 16 bridge sites in Northeast Arkansas (NEA) were determined using a hybrid, non-invasive technique. These profiles were used to evaluate the liquefaction resistance at the selected sites using the simplified procedure by Seed and Idriss (V_s approach). The liquefaction resistance was also evaluated using the Standard Penetration Test (SPT approach) results from the geotechnical investigations at these sites that were conducted by the Arkansas State Highway and Transportation Department (AHTD). The Liquefaction Potential Index (LPI), as introduced by Iwasaki, was used to evaluate the severity of liquefaction. The results of both approaches were then compared. Recommendations were made to AHTD personnel for liquefaction evaluation of future bridge projects based on the results of this research.

Keywords: Earthquake; Liquefaction; Northeast Arkansas; NMSZ; Shear Wave Velocity.

1. INTRODUCTION

NEA is considered part of the Mississippi embayment. The embayment is a trough-like depression that plunges southward along an axis that approximates the course of the Mississippi River. The embayment is filled with sediments of clay, silt, sand, and gravel to depths reaching 500 meters to 1000 meters. According to Broughton, Arsdale, and Broughton [1], most of the study area has surficial deposits, which include Holocene artificial fill, alluvium Holocene deposits along river channels, Pleistocene Loess and Terrace deposits, and Lafayette Gravel. Based on the work by Rix and Romero [2], many of these deposits are susceptible to liquefaction.

NEA is also expected to experience significant damage from earthquakes in the New Madrid seismic zone (NMSZ). The NMSZ, which extends from southeastern Missouri to northwestern Tennessee and northeastern Arkansas, generated three large events in 1811-1812. According to Bakun and Hopper [3], the best estimates of the moment magnitudes for the three large events range from 7.5 to 7.8. The recurrence interval for events similar to 1811-1812 is estimated by Tuttle, Schweig, Sims, Lafferty, Wolf, and Haynes [4] to be 500± 30 years based on geologic data.

Liquefaction is the result of excess porewater pressure generated in saturated granular soils from ground shaking during earthquakes. Several cases of liquefaction-induced damage to bridges and other structures have been documented by Kramer [5]. The method referred to as the simplified procedure, which was introduced by Seed and Idriss [6], is typically used for evaluating the occurrence of liquefaction based on SPT results. The method was revised several times (Youd, Idriss, Andrus, *et al.* [7]) and it now includes a method based on shear wave velocity (V_s) measurements.

The objective of this research was to obtain site-specific V_s profiles to depths of at least 30-meters at selected bridge locations in Northeast Arkansas. The V_s profiles were to be used to determine the soil liquefaction potential at each bridge location using the simplified V_s procedure. These V_s liquefaction evaluations were to be compared with Standard Penetration Test (SPT) liquefaction evaluation procedures using blow count (N) data already available from the AHTD.

2. SITE CHARACTERIZATION AND SURVEY PROCEDURE

Surface wave tests were performed on 16 sites located throughout NEA. Sites were selected based primarily on their proximity to pre-existing highway bridges. All of these sites are north of Latitude 35.0° and west of Longitude 90.0°. The test site locations are summarized in Table **1** below. The test locations are shown in Fig. (**1**).

Noninvasive methods for determining in situ soil velocity profiles are rapidly becoming popular in the engineering field. Their low cost when compared to that of traditional invasive methods, such as downhole and Crosshole, make them attractive in today's economy. Surface wave methods take advantage of the dispersive nature of Raleigh waves. By measuring the wavelength and velocity of propagating Raleigh waves, the stiffness properties of a soil profile can be characterized with respects to depth. These properties can then be utilized in evaluating site response, soil-structure interaction, and liquefaction potential.

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Table 1. Testing Site Locations

Site	Latitude	Longitude	Site	Latitude	Longitude
110337	35.200	-90.246	100105	35.600	-90.214
BR1108	36.391	-90.399	100153	35.821	-90.433
BR1110	36.460	-90.357	100303	35.823	-90.501
BR4706	35.600	-90.269	100478	36.052	-90.360
110434	35.415	-90.284	100522	35.519	-90.413
R00059	35.841	-90.753	100523	35.481	-90.358
110401	35.392	-90.273	100547	35.475	-90.333
110358	35.163	-90.224	110288	35.273	-90.559

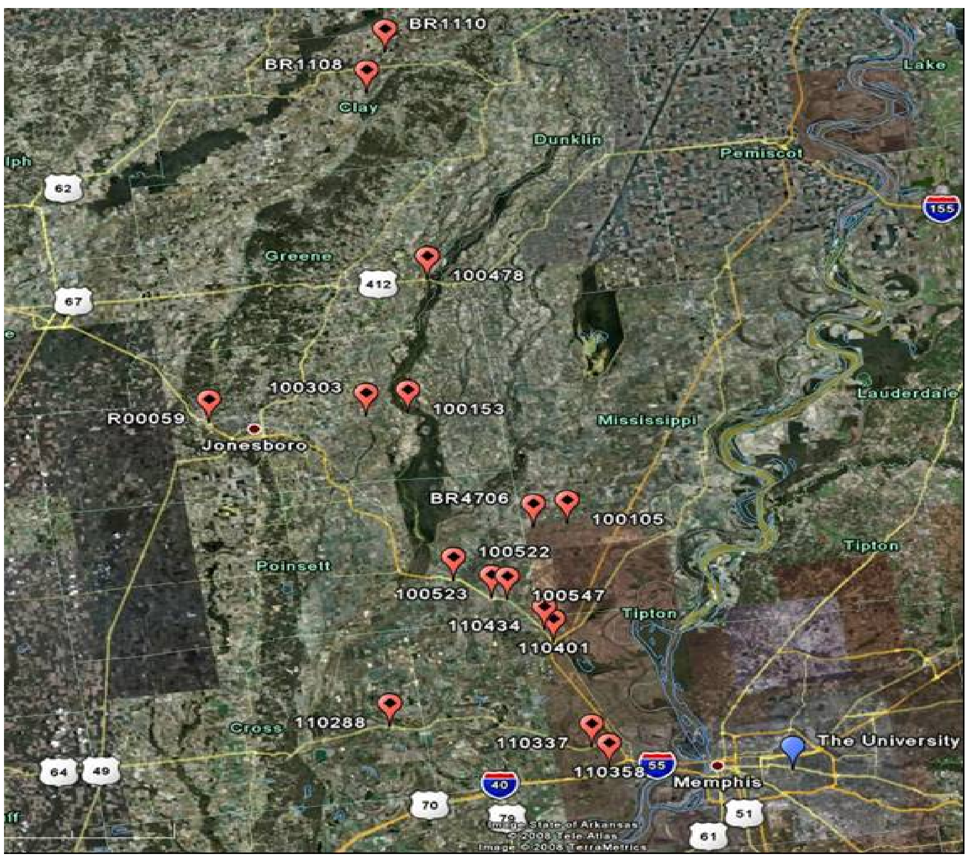

Fig. (1). Test Site Locations.

Two of common noninvasive methods used today are Multi-Channel Spectral Analysis of Surface Waves (MSASW) and Refraction Microtemors (ReMi). MSASW utilizes an active harmonic source for generating Raleigh waves over a range of frequencies. ReMi utilizes the ambient vibrations (noise) already present in the earth as the source for Raleigh waves.

Both methods are used to produce a dispersion curve that describes the Raleigh phase velocity with respects to frequency. The two methods were combined for the construction of a composite dispersion curve that was used in the inversion process. The dispersion curve is then used in the inversion process for the determination of soil velocity with depth. The inversion process used for the determination of soil velocity profiles is that outlined in Pezeshk and Zarrabi [8]. The process utilizes a genetic algorithm (GA) to adjust theoretical dispersion curves obtained from the forward method (Rix and Lai [9]) to fit the experimental dispersion curves obtained from the methods described above. The GA is an optimization process that simulates the natural

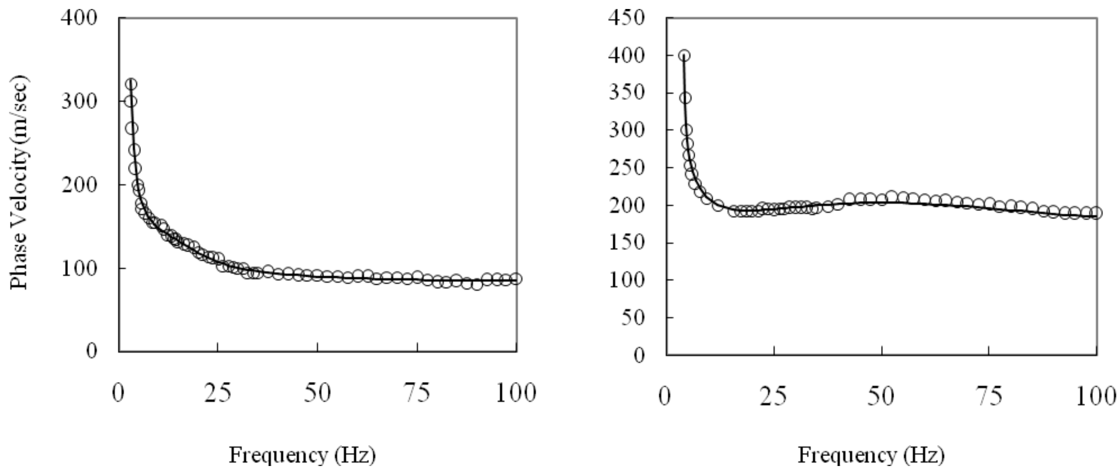

Fig. (2). Dispersion Curves for 2 sites. Open circles denote experimental data; solid lines denote theoretical relationship

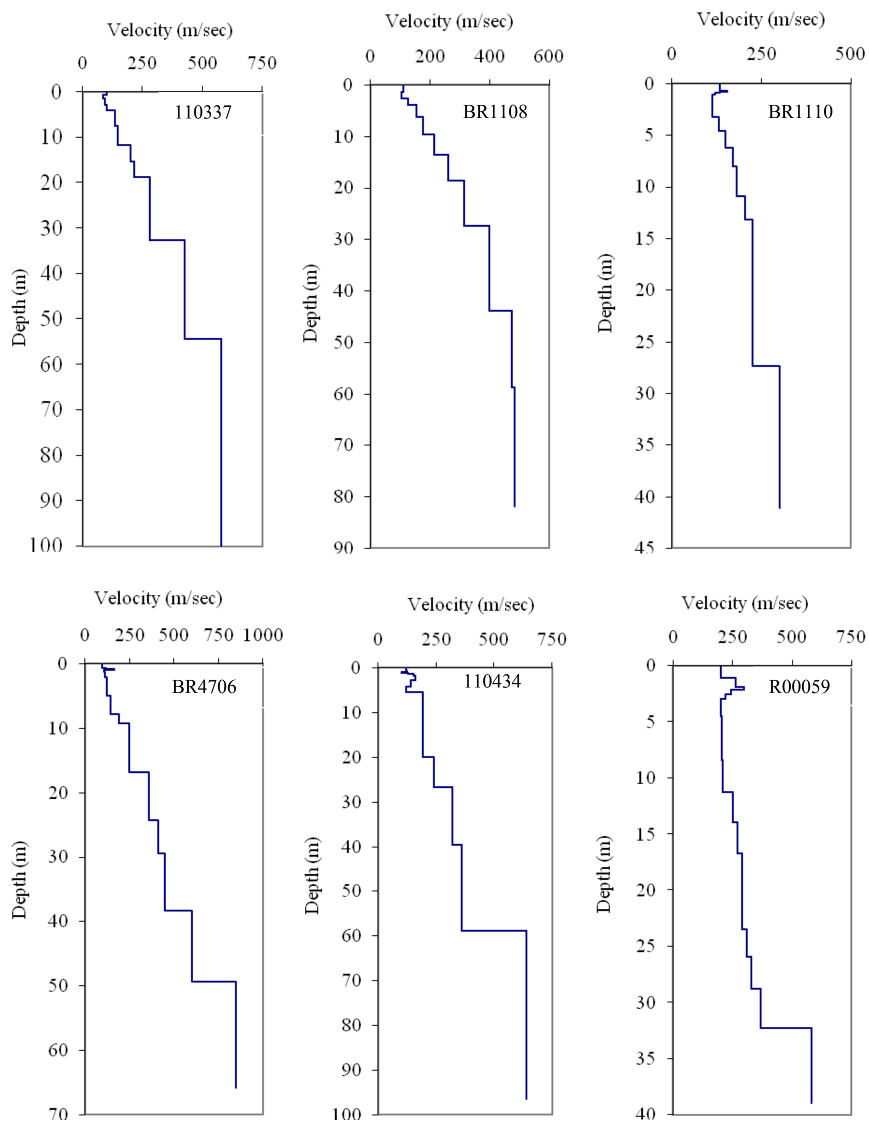

Fig. (3). Shear-wave velocity profiles for sites tested.

evolution process using global search methods based on a stochastic approach, which relies on survival of the best fit (Holland [10]). An example of the experimental dispersion curves along with their theoretical convergence is displayed in Fig. (**2**). The resulting soil velocity profiles are displayed in Figs. (**3**, **4**, and **5**).

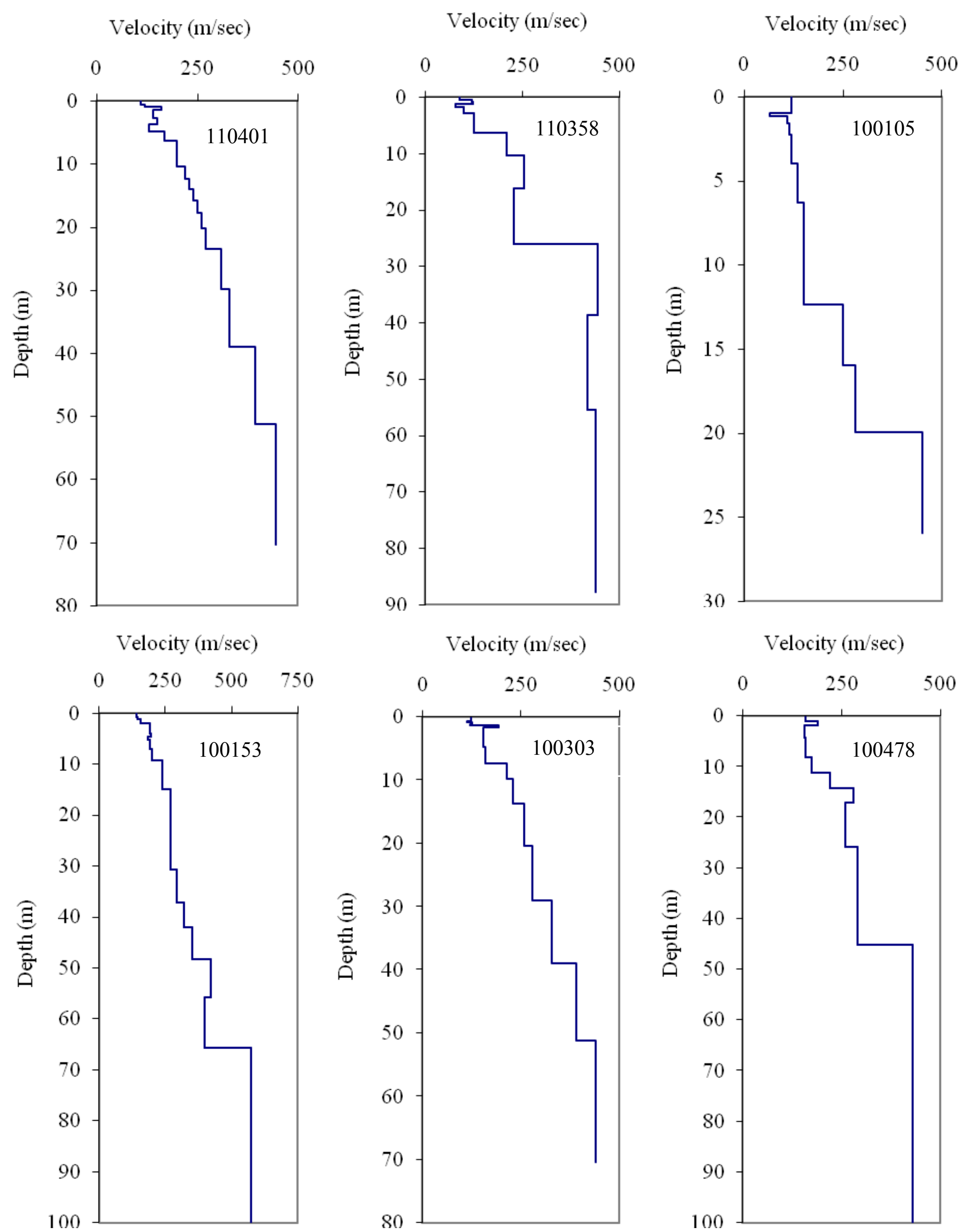

Fig. (4). Shear-wave velocity profiles for sites tested.

The Site Class was determined for each test site based on the definitions introduced in the AASHTO Guideline Specifications for LRFD Seismic Bridge Deign. The determinations were based on the V_s values and a SPT values from a representative boring. The results are presented in Table **2**.

The data in Table **2** indicates consistent results, except for sites 110337 and 110401. The results from these two sites were considered "borderline" between site classes D and E. It is imperative to recognize the difference between the two methods, as each of the 2 methods has its advantages, disadvantages, and sources of errors.

3. COMPUTATIONAL METHODS

The liquefaction analyses were performed using both V_s profiles and SPT blow counts as discussed below.

Method of Analysis

Seed and Idriss [6] developed a "simplified procedure" that can determine the safety factor against liquefaction. The simplified procedure was developed from evaluation of field observation and field and laboratory test data. The procedure uses two variables to evaluate for liquefaction of soils. These variables are the seismic demand induced by the design earthquake and expressed in terms of cyclic stress ratio (*CSR*), and the capacity of soil to resist liquefaction, expressed in terms of cyclic resistance ratio (*CRR*). The CSR is calculated as follows:

$$CSR = \frac{\tau_{av}}{\sigma'_{VD}} = 0.65\left(\frac{a_{\max}}{g}\right)\left(\frac{\sigma_{VD}}{\sigma'_{VD}}\right)r_d \tag{1}$$

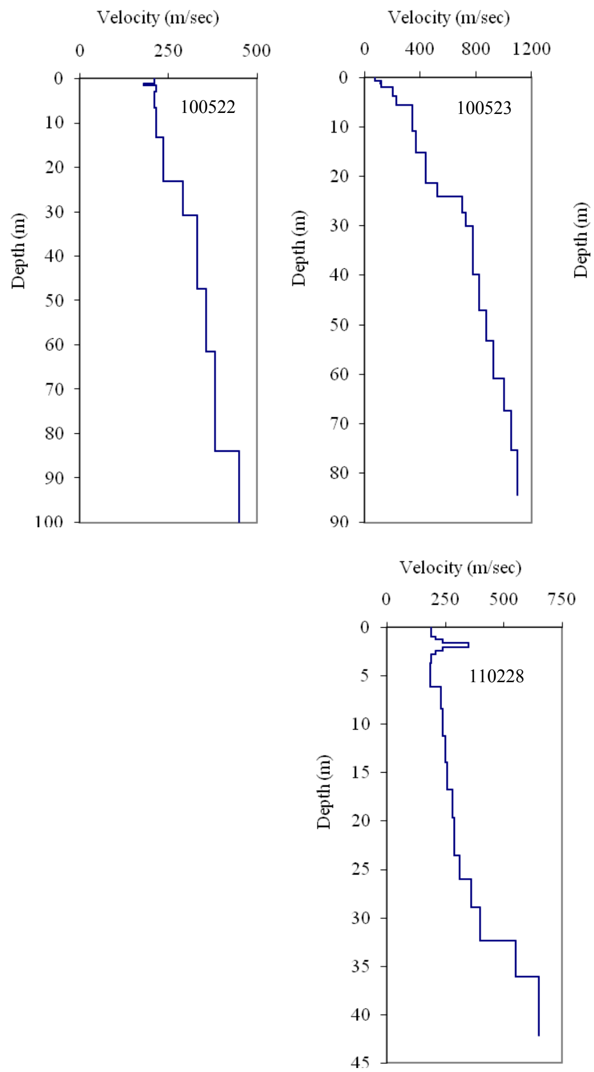

Fig. (5). Shear-wave velocity profiles for sites tested.

where,

0.65: a weighing factor introduced by Seed to account for the average equivalent cyclic shear stress caused by the earthquake (assumed to be 0.65 of the maximum induced stress).

a_{max}: peak horizontal ground acceleration.

σ_{VD}: total vertical overburden pressure.

σ'_{VD}: effective vertical overburden pressure.

r_d: shear stress reduction coefficient to adjust for the flexibility of the soil profile.

Values of r_d are commonly estimated from a chart introduced by Seed and Idriss [6]. The participants of the 1996 National Center for Earthquake Engineering Research, known as NCEER [Youd., Idriss, Andrus, *et al.* [7]) recommended the following equations to estimate the average r_d values for noncritical projects (z is the depth below ground surface):

$$r_d = 1.0 - 0.000765 \cdot z \qquad z \leq 9.15m \tag{2}$$

$$r_d = 1.174 - 0.0267 \cdot z \qquad 9.15m \leq z \leq 23m \tag{3}$$

$$r_d = 0.744 - 0.008 \cdot z \qquad 23m \leq z \leq 30m \tag{4}$$

The first 2 equations were proposed by Liao and Whitman [11], and the third equation was proposed Robertson and Wride [7]. Revised average values were proposed by Idriss [12] based on analytical work by Golesorkhi [13]. The revised values were magnitude dependent (Andrus and Stokoe [14]). It is important to note that the workshop participants indicated that the certainty with which CSR can be calculated decreases with depth when the mean r_d values are used to simplify the calculations. Moreover, the simplified procedure is not well verified with case history data for depths greater than 15 m. However, they agreed that for convenience in programming, r_d values presented by Equations (2) through (4) above are suitable for routine engineering practice. Therefore, the method of calculating CSR intro-

Table 2. Site Class Based on V_s and SPT values

PI Site No.	AHTD Site No.	Vs-Based Classification		SPT-Based Classification	
		Average V_s (m/sec) top 30 m	Site Class	Average N Value in Top 30 m., blows/0.30 m	Site Class
1	110288	259	D	15.5	D
2	110358	200	D	19.7	D
3	110401	217	D	15.0	E
4	100547	219	D	23.3	D
5	110337	175	E	15.3	D
6	110434	203	D	15.5	D
7	100523	330	D	22.5	D
8	100522	224	D	20.3	D
9	BR4706	223	D	20.9	D
10	100105	211	D	15.6	D
11	R00059	220	D	16.1	D
12	100303	220	D	20.6	D
13	100153	230	D	16.7	D
14	100478	215	D	16.0	D
15	BR1108	211	D	19.4	D
16	BR1110	202	D	18.0	D

duced in this report represents the best available approach at this stage.

The CRR for a moment magnitude, $M_w = 7.5$ earthquake, $CRR_{7.5}$, can be determined using several field testing methods such as SPT, Cone Penetration Testing (CPT), Becker Penetration Testing (BPT), and Shear Wave Velocity, V_s.

According to the 2008 addendum to the AASHTO Guide Specifications for LRFD Seismic Bridge Design, the 1000-year return period (approximately equivalent to a 7% probability of exceedance in 75 years) was recommended for the seismic design of highway bridges. The United States Geological Survey (USGS) prepared a ground motion software tool to simplify the determination of the seismic design parameters. The software provides values for the design peak ground acceleration based on the site class and the location as documented by the Latitude / Longitude. The method is based on seismic hazard curves previously published by the USGS, which takes into account the nature of the hazard (fault system and focal depth), the source to site distance, and the attenuation characteristics of the NMSZ. These data are presented in Table **3**.

As previously mentioned, several researchers estimated the moment magnitudes of the 1811-1812 events to be in the range of 7.5 to 7.8 (Bakun and Hopper [3], and Rix and Romero [2]). After evaluating the available data, and based on the previous experience of the PIs in the study area, a decision was a made to utilize a moment magnitude (typically used in engineering practice) of 7.5 as the AASHTO-based event (probability of exceedance of 7% in 75 years).

SPT-Based Analysis

To calculate the CRR using the SPT data, the equivalent clean sand standard penetration resistance defined as $(N_1)_{60cs}$, should first be determined. This can be done as follows:

$$(N_1)_{60cs} = N_f C_n C_a C_b C_r C_s \tag{5}$$

where,

$(N_1)_{60cs}$: corrected normalized standard Penetration N-value,

N_f: standard penetration value measured in the field,

C_n: depth (overburden stress) correction factor for an effective overburden pressure of 100 kPa,

C_e: hammer energy ratio correction factor for 60% hammer efficiency,

C_b: borehole diameter correction factor,

C_r: rod length correction factor, and

C_s: sampler correction factor (with or without a liner).

Fine content (% pass No. 200 sieve) factors were then applied to $(N_1)_{60cs}$ in order to account for the effect of the fines in the soil, thus calculate $(N_1)_{60f}$. Two approaches were adopted in this study. The first one (ALT.A) was to apply the fines correction using the method introduced by Stark and

Table 3. Site Geographic Locations and Design Peak Ground Accelerations (A_s), g

AHTD Site No.	Geographic Location		Design Peak Ground Acceleration, A_s (g)
	Latitude	Longitude	
110288	35.27326	-90.55888	0.833
110358	35.16262	-90.22425	0.522
110401	39.39146	-90.27344	0.800
100547	35.4748	-90.33327	0.951
110337	35.19971	-90.24574	0.592
110434	35.41504	-90.28398	0.839
100523	35.48047	-90.35757	0.976
100522	35.51862	-90. 41266	1.035
BR4706	35.59755	-90.26931	1.038
100105	35.59754	-90.21449	1.030
R00059	35.84046	-90.75316	0.620
100303	35.82309	-90.5006	0.830
100153	35.82073	-90.43327	0.897
100478	36.05171	-90.36039	0.667
BR1108	36.39091	-90.39922	0.470
BR1110	36.45966	-90.35708	0.462

Olsen [15]. This method applies the correction for fines contents in the range of 0 to 35%. When this approach was taken, fine-grained soils with more 60 percent fines were assumed to be non-liquefiable. The second approach (ALT. B) was to use the Modified Stark and Olsen, where the fines content correction line is extended to fines of 100% instead of having a maximum correction factor (keeping the correction line flat) at 35%. For this approach, any soil layer with less than 100% fines was assumed to be liquefiable. For the purpose of comparing the two methods of analysis (V_s and SPT) in this study, only the first approach was used in this study.

The following equation was used to calculate $CRR_{7.5}$ (Blake [16]):

$$CRR_{7.5} = \frac{a + cx + ex^2 + gx^3}{1 + bx + dx^2 + fx^3 + hx^4} \tag{6}$$

where,

x = $(N_1)_{60f}$

a = 0.048

b = -0.1248

c = -0.004721

d = 0.009578

e = 0.0006136

f = -0.0003285

g = $-1.673\text{x}10^{-5}$

h = $3.714\text{x}10^{-6}$

The safety factor against liquefaction is defined according to Youd *et al.* [7] as:

$$FS = \frac{CRR_{7.5}}{CSR} MSF \tag{7}$$

where *MSF* is the magnitude scaling factor to adjust the simplified curve to magnitudes smaller or larger than 7.5 (Seed and Idriss [17]). Since the design earthquake was assumed to have a moment magnitude of 7.5, the *MSF* had no impact on this analysis.

Correction factors for Equation (7) were introduced by Seed and Idriss [18] to account for high overburden stress (K_σ) and static shear due to sloping ground (K_α). The NCEER workshop participants, however, agreed that the effect of sloping ground is not well understood, although correction curves were available (Harder and Boulanger [19]). They recommended that the evaluation is beyond routine application of the simplified procedure. The high overburden stress correction factor was also ignored as a partial compensation for the unquantified but substantial increase in the liquefaction resistance due to aging.

Figs. **6** through **9** present representative boring logs from the 16 sites. A commercial software (*Liquefy Pro, by*

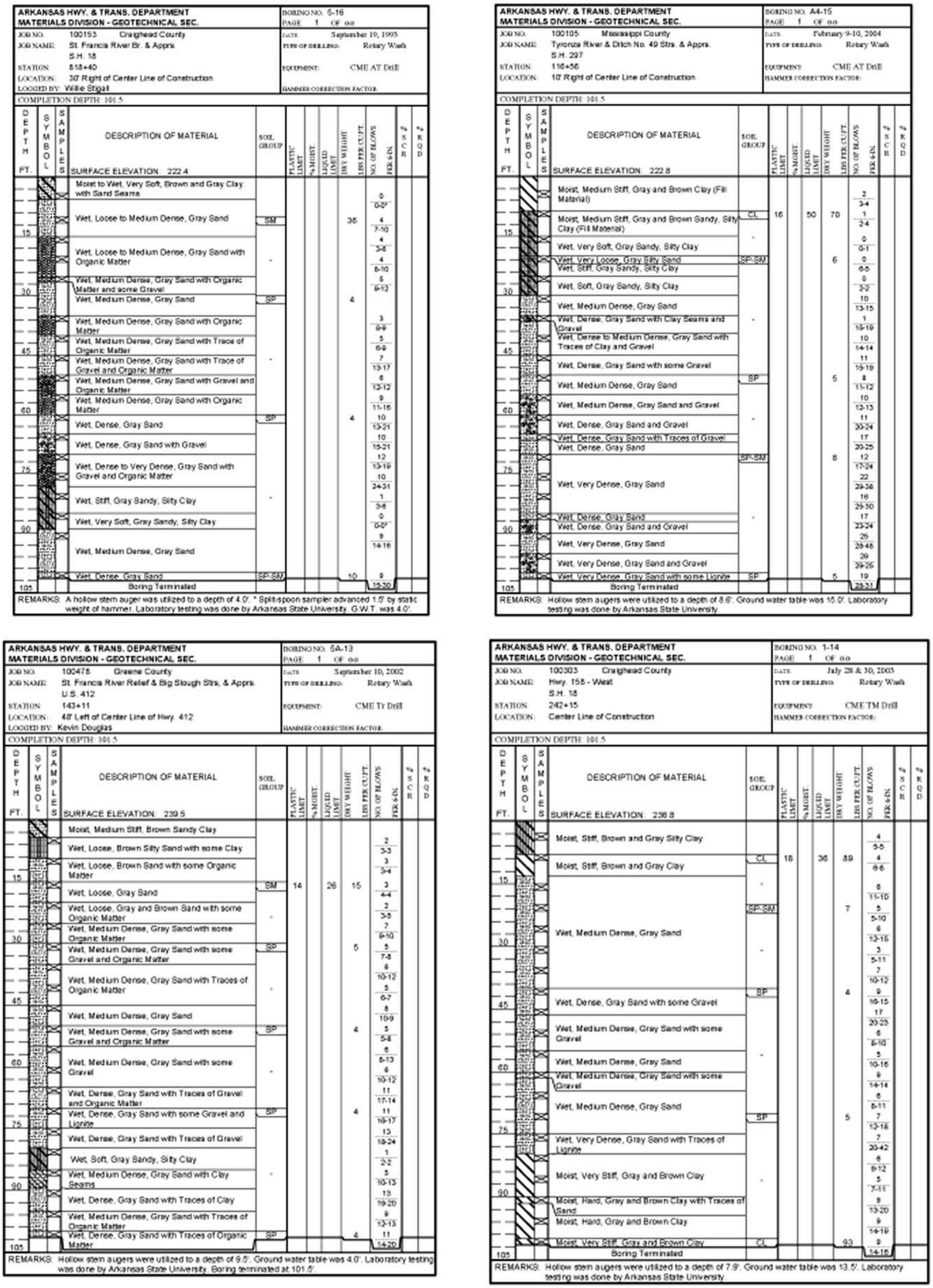

Fig. (6). Representative boring logs from Sites 100153, 100105, 100478 and 100303.

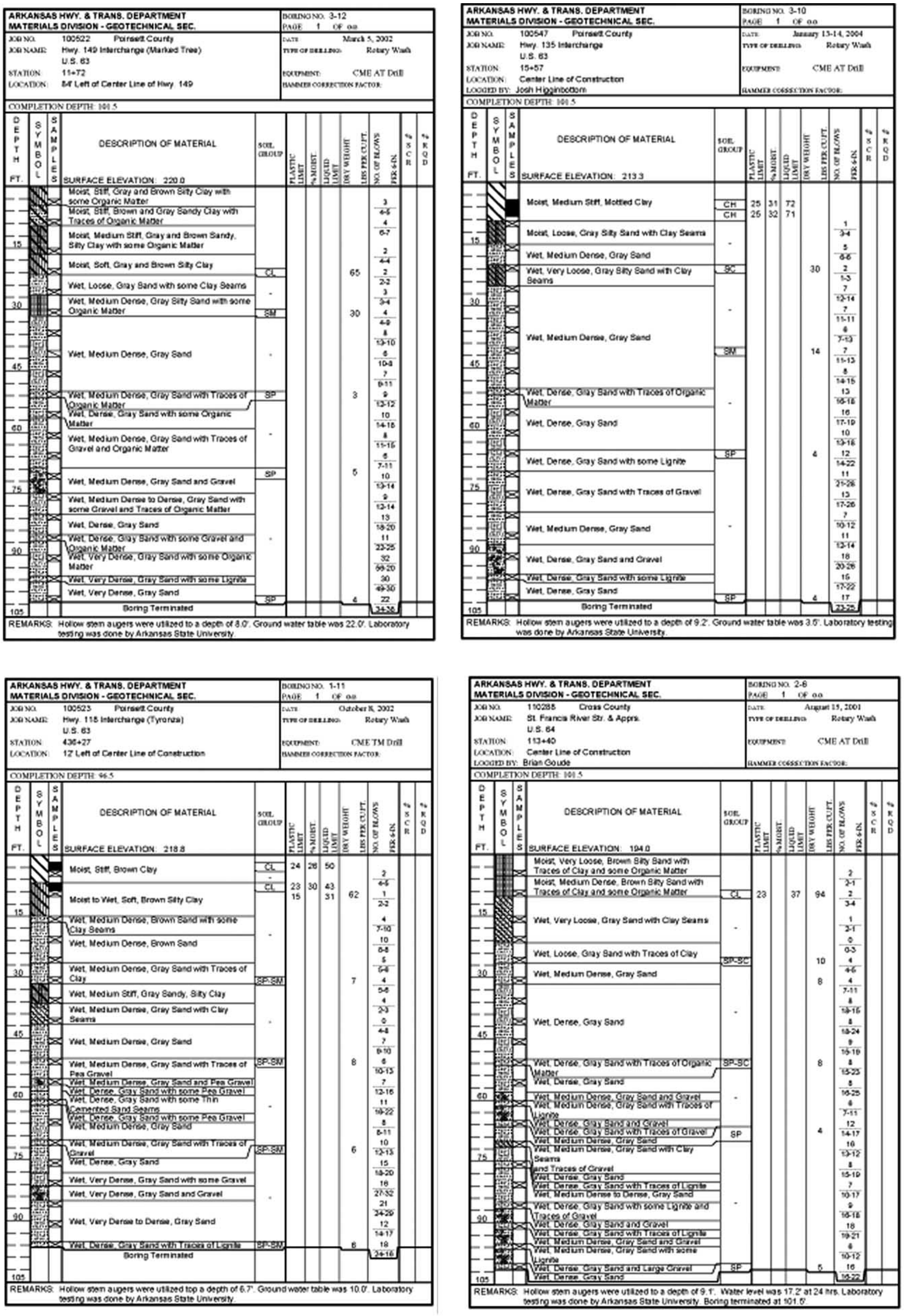

Fig. (7). Representative boring logs from Sites 100522, 100523, 100547 and 110288.

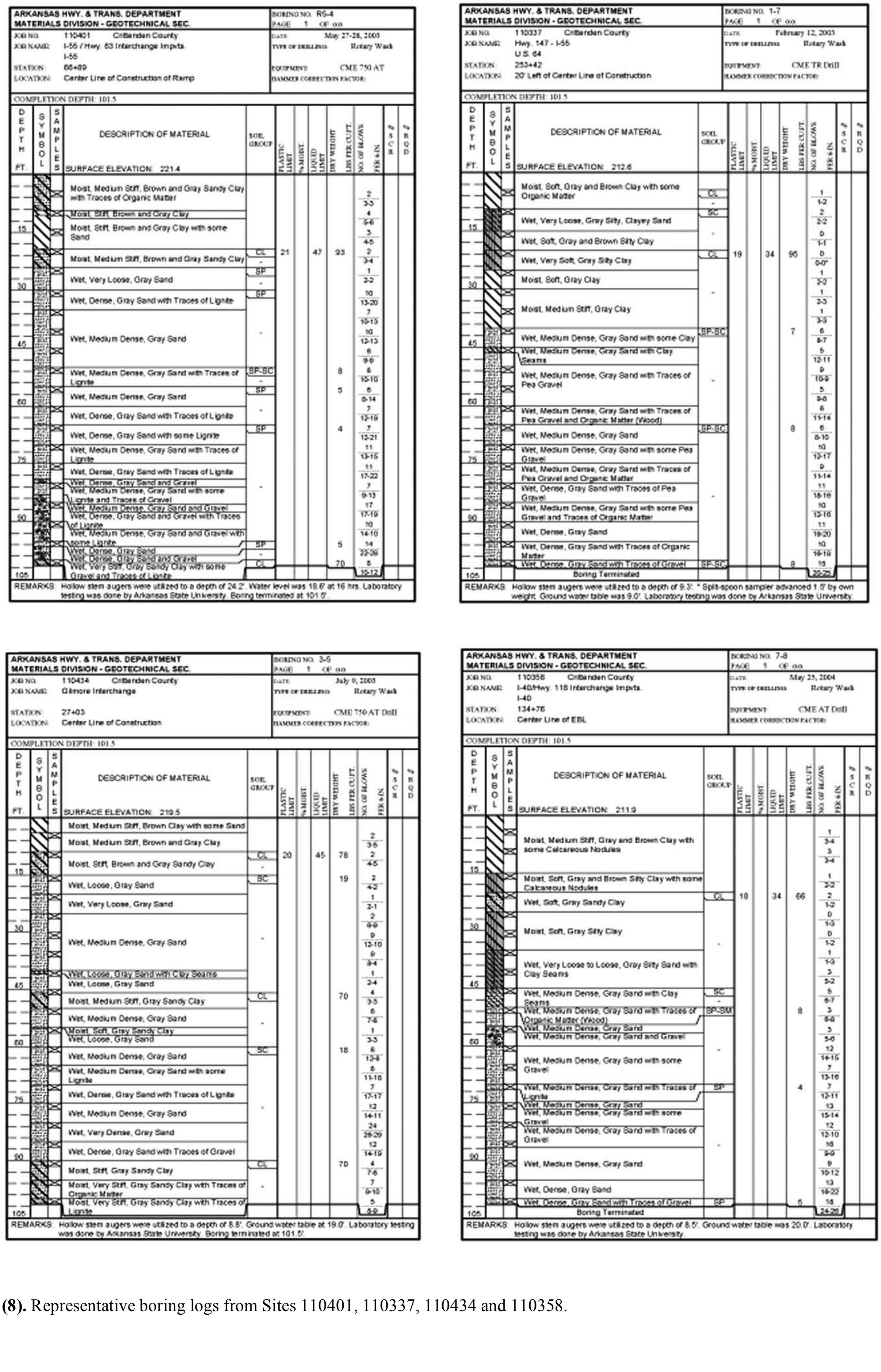

Fig. (8). Representative boring logs from Sites 110401, 110337, 110434 and 110358.

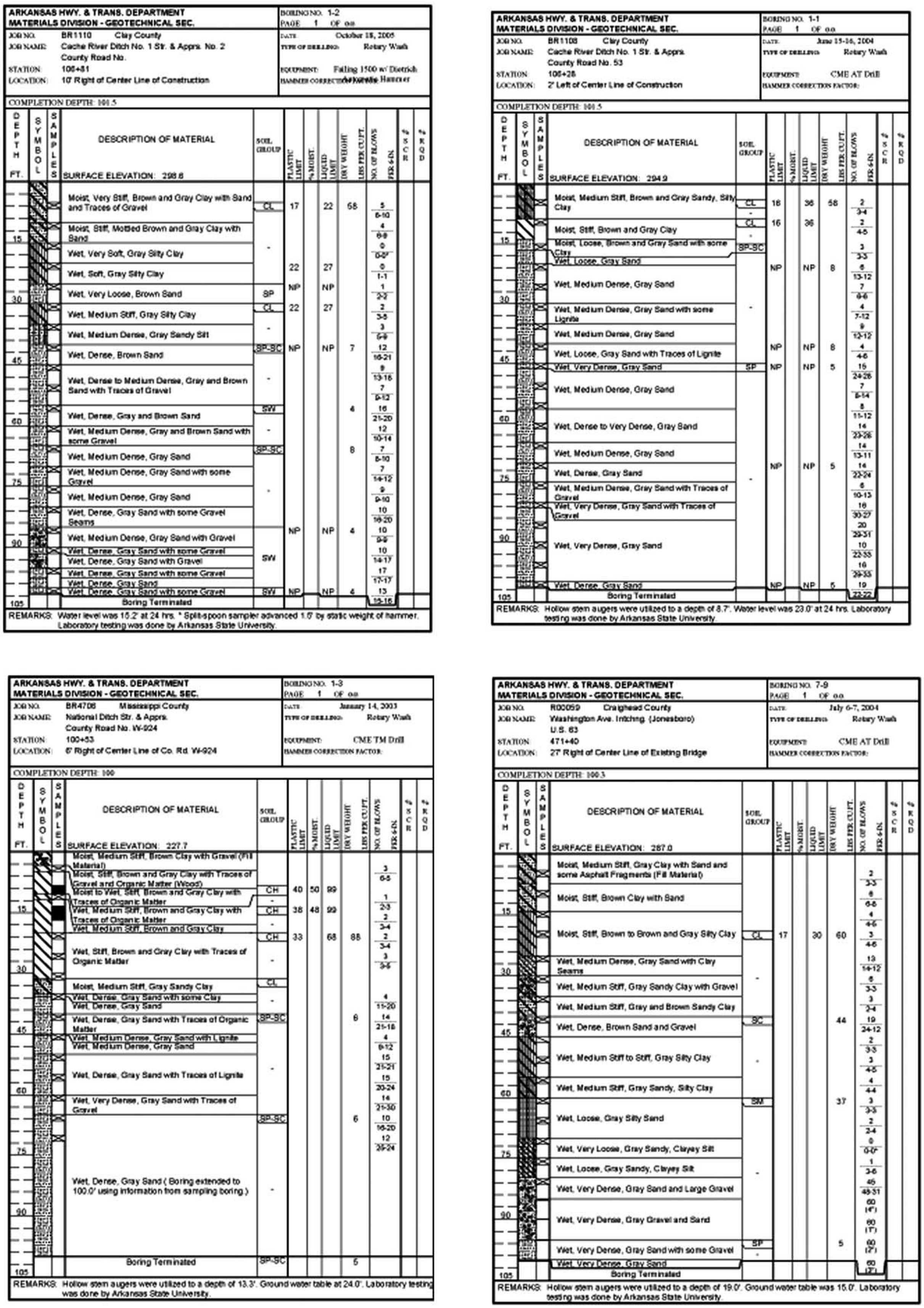

Fig. (9). Representative boring logs from Sites BR1110, BR1108, BR4706, and R00059.

Civiltech Software, 2007) was used to perform the SPT-based liquefaction analysis using the available boring logs and soil data for each site. Table **4** presents the results from 2 sites.

Shear Wave Velocity-Based Analysis

The shear wave velocity-based liquefaction analysis, referred to herein as the V_s analysis, required the following information:

Peak ground acceleration and earthquake magnitude.

Depth to groundwater.

Soil unit weights, soil classifications, fine contents, and Plasticity Indices.

Shear wave velocity profiles, which were determined using the field testing.

In addition, the following assumptions were made to perform the analysis:

The slope of the ground surface is 6% or less. This assumption is accurate, since the area tested at each site were basically flat.

The approximate age of the soil deposits at the test sites is 2000 years. This is considered a conservative assumption to some extent. The geologic maps of the sites indicated that the surficial soils are of Holocene age, which is measured in terms of thousands of years (< 10,000 years).

The method presented by Andrus and Stokoe [20] was used to perform the analysis. First, the stress-based or normalized shear wave velocity was calculated as follows:

$$V_{s1} = V_s \left(\frac{P_a}{\sigma'_{vo}} \right)^{0.25} \tag{8}$$

where P_a is a reference stress of 100 kPa (2000 psf), approximately the atmospheric pressure, and σ'_{vo} is effective overburden pressure in the same units. In applying the above equation, two assumptions were made (Andrus, Stokoe, and Juang [21]). These assumptions were that the initial effective horizontal stress is a constant factor of the effective vertical stress. The second assumption is the factor mention above is equal to 0.50. These assumptions were considered accurate for the level, normally consolidation deposits of the study area.

The *CRR* is calculated as follows (Andrus and Stokoe [20]):

$$\text{CRR} = (a\ (K_{a1} V_{s1}/100)^2 + b\ \{1/(V^*_{s1} - V_{s1}) - 1/V^*_{s1}\}\ K_{a2})\ MSF \tag{9}$$

where,

V^*_{s1}: limiting upper value of V_{s1} for liquefaction occurrence.

= 215 m/sec for sands and gravels with fines contents ≤ 5%

=215 – 0.5 (FC – 5) for sands with 5% < FC <35%

= 200 m/sec for sands and gravels with fines contents ≥35%

a,b: curve fitting parameters taken as 0.022 and 2.80, respectively

MSF: magnitude scaling factor as previously described (equal to 1.0 in this study)

Andrus, Stokoe, and Juang [21] introduced two factors (K_{a1} and K_{a2}) to account for the effect of aging. The first factor is to correct for high V_{s1} caused by aging, and the second factor is to correct for the influence of aging on CRR. They

Table 4. SPT-Based Liquefaction Analysis Results – Sites 110288 / 100547

Site No.	Borehole No./Depth, m	Approach	Zones (ft. below Ground surface) that Indicated SF < 1.0
110288	B-1/30	ALT. A	4.6-20.0/22.6-30.0
	B-2 / 30		5.2-10.0/17.7- >30.0
	B-3 / 21		3.0-5.5/10.0-15.2/16.8
	B-4 / 20		3.0-6.0/6.4-8.5/9.8-12.8/16.8
	B-5 / 30		35-45 16.8-20.0/24.4-25.0
	B-6 / 23		8.5-11.6/13.1-17.4
100547	B-1 / 30	ALT. A	6.1-48/19.2-21.0/21.9-22.9/26.8-29.0
	B-2 / 30		6.0-15.8/17.7-22.3/24.4-29.0
	B-3 / 30		7.0-20.1/23.2-27.4/27.7/30.0
	B-4 / 30		7.0-9.8/11.9-13.4/15.5-24.4/24.7-30.0
	B-5 / 30		7.0-7.6/9.8-14.9/15.5-18.9/22.3-23.2
	B-6 / 30		7.6-8.8
	B-7 / 30		7.6-7.9/13.7-14.4

stated that both factors are equal to 1.0 for un-cemented deposits of Holocene age. Therefore, a value of 1.0 was assigned to both factors in this study.

It is of particular importance to recognize that the CRR – V_{s1} relationship was developed using observation data from 26 earthquakes in California, Japan, China, Taiwan, and Idaho. Only seven of the 26 earthquakes had a moment magnitude (M_w) larger than 7.0. It is not known how well this correlation would fit with an earthquake of magnitude of 7.5 in the NMSZ.

The CSR values were calculated in the same manner as in the previous analysis. The safety factor against liquefaction was calculated as CRR/CSR. No additional corrections factor due to sloping ground or high overburden stress were applied due to the reasons previously mentioned. Soils classified as CL, CH, and ML were assumed to be nonliquefiable. A spreadsheet was used to perform this analysis. The results are summarized in Table **5**.

4. LIQUEFACTION POTENTIAL INDEX

The liquefaction potential for the test sites was evaluated using the Liquefaction Potential Index (LPI) as proposed by Iwasaki *et al*. [22, 23], which can be calculated as follows:

$$LPI = \sum_{i=1}^{n} w_i S_i H_i \tag{10}$$

where

n: number of layers in the upper 20 m,

w_i: depth-dependent weighting function for layer I,

$wi(z) = 10\text{-}0.5z$ (z = depth below ground surface, m)

S_i: degree of severity for layer i defined as:

$S = 0$ for $FS > 1.0$

$S = 1 - FS$ for $FS < 1.0$

FS: factor of safety against liquefaction for layer i as previously defined.

H_i: thickness of layer i, m.

Iwasaki *et al.* [22] identified LPI values of 5 and 15 as the lower bounds of "moderate" and "major" liquefaction, respectively. The parameter was used by Rix and Romero-Hudock [2] to map the liquefaction potential for Shelby County, Tennessee. Moreover, Toprak and Holzer [24] found that median values of LPI of 5 and 12 corresponded to occurrence of sand boils and lateral spreading, respectively. They also found that LPI correlated well with liquefaction effects. The results are summarized in Table **6**.

General Discussion Regarding the Shear Wave Velocity Approach

According to Youd, Idriss, Andrus *et al.* [7], the shear wave velocity approach has several advantages. It can be accurately measured in situ using a number of techniques such as downhole seismic tests, the seismic cone penetration tests, spectral analysis of surface waves (SASW), or refraction microtremor (ReMi). It is also directly related to small-strain shear modulus, which a parameter required in analytical procedures for estimating dynamic soil response at small

Table 5. Shear Wave Velocity-Based Liquefaction Analysis Results

Site No.	Average Shear Wave Velocity in the Upper 30 m, m/sec.	Source of soil Information (Boring No.)	Zones (m below Ground surface) that Indicated SF < 1.0
110288	259	B-1	6.1 – more than 30.0
110358	200	B-1	16.8 – 27.4
110401	217	B-2	10.7 – 24.4
100547	219	B-7	10.7 – 22.9
110337	175	B-2	15.2 – 19.8
110434	203	B-9	6.1 – 27.4
100523	330	B-1NEW	No Liquefaction
100522	224	B-3	7.6 – 24.4 / 30.0 -
BR4706	223	B-1	13.7 – 18.3
100105	211	B-4	10.0 – 16.8
R00059	220	B-6	No Liquefaction
100303	220	B-1	4.6 – 13.7 / 18.3 – 21.3
100153	230	B-9	7.6 – 10.7 / 25.9 – more than 30.0
100478	215	B-5	15 -13.7/ 16.8- 25.9
BR1108	211	B-1	6.1-12.2 / 15.2 – 19.8
BR1110	202	B-1	12.2 – 27.4

Table 6. LPI Evaluation Results

Site No.	SPT Analysis		V_s Analysis
	Boring No.	LPI Value	
110288	B-2	47.1	14.7
110358	B-7	15.5	2.3
110401	B-5	24.8	14.4
100547	B-3	22.3	19.5
110337	B-1	16.1	4.7
110434	B-3	31.1	39.9
100523	B-1	43.8	0
100522	B-3	29.3	28.0
BR4706	B-1	5.7	1.8
100105	B-4A	25.5	21.9
R00059	B-7	10.0	0
100303	B-1	27.5	29.4
100153	B-5	48.7	10.5
100478	B-5A	48.4	30.5
BR1108	B-1	11.8	23.9
BR1110	B-1	19.7	7.8

and intermediate shear strains. In addition, shear wave velocity measurements are possible in soils that are difficult to penetrate and in sites where drilling is not permitted. Youd *et al*. [7] pointed several limitations to the approach. Seismic wave velocity measurements are made in small strain, where as liquefaction is a large strain phenomenon. The testing does not provide samples. The V_s analysis may indicate liquefaction potential in soft, non-liquefiable clay-rich deposits. A V_s–based analysis may indicate high safety factors in weakly-cemented sands that may be liquefiable.

Andrus, Stokoe, and Juang [21] stated that site-specific liquefaction evaluation using only or mainly the V_s method should be limited to situations where:

Crosshole, downhole, suspension logger, or SASW tests are conducted such that high-quality V_s values are determined at intervals of at least ¼ of the critical layer (the layer most likely to liquefy).

The limitations they stated in their paper regarding these methods are considered.

Sufficient borings are conducted to identify materials type and to insure that thin, liquefiable strata are not present.

The critical layer is of Holocene age and contains no or little carbonate (considered as a cementing agent).

Andrus, Piratheepan, Ellis, Zhang, and Juang [25] compared the V_s-based CRR approach to the penetration-based one using data from 43 Holocene-age sand layers in California, South Carolina, Canada, and Japan. They stated that the V_s-based CRR curves are more conservative than the SPT-based curves.

5. EVALUATION OF THE RESULTS

It is of particular importance that the hammers utilized to perform SPT testing be calibrated in a regular basis to obtain accurate hammer efficiencies, thus reducing potential equipment-based variability. AHTD personnel provided hammer efficiencies for the hammers used during the site investigations of the tested bridge sites.

The safety factors against liquefaction using the V_s were plotted against three different sets of safety factor values using the SPT approach at all 16 sites. These sets are as follows:

The safety factors using a representative boring at each site. The representative boring was chosen as the closest boring to the center of the MSASW array.

The lower bound values of the safety factors using data from the available borings at each site. These plots were created by using the lowest safety factors from the analyzed borings at comparable depths.

The average values of the safety factors. These plots were created in a similar manner to the ones above, except the average values were used instead of the lowest ones.

The variability among the borings at each site was assessed by calculating the standard deviation of the safety factors at each depth then calculating the average standard deviation (STDEV) for the entire profile. The variability was considered significant when the STDEV value exceeded 1.0. The safety factors from both approaches below 20 m were also evaluated, since liquefaction below a depth of 20 m has not been previously documented. Figs. **10** through **13** show the plots mentioned above. Please note that the plots were truncated at a safety factor value of 2.0.

The comparison of these plots is summarized in Table **7** below.

Results Comparison

The liquefaction analyses and the LPI calculations indicated the following:

Both the SPT and V_s analyses indicated significant liquefaction may occur at the test sites during the design seismic event. This is due to the combined effect of the nature of the soil deposits and the relatively large peak horizontal ground acceleration required by the latest AASHTO LRFD design guide.

Significant variability was noted within each site using the SPT approach, as the STDEV values exceeded 1.0 in 11 of the 16 sites. This can be attributed to the nature of the soil deposits, as the alluvial materials are expected to vary in both the horizontal and vertical directions. Another source of variability is the test procedure. The SPT results are highly dependent on the equipment used, procedure, as well as the operator's experience and consistency, even when the test standard (ASTM D 1586) is closely followed. The effect of soil variability could not be verified in the V_s approach, as only one test was performed at each site.

Comparing the LPI values, the V_s approach in general results in lower a degree of liquefaction. The LPI values using SPT results indicated that 13 of the 16 sites, or 81%, have LPI values above 15. On the other hand, the V_s approach indicated that only 44% of the sites (7 of the 16 sites) have LPI values above 15.

Table **7** shows that only 3 out of 16 sites indicated lower V_s –based safety factors when compared to the safety factors indicated by the representative borings. When compared to the average SPT, 5 sites indicated lower V_s –based safety factors, 6 sites indicated lower average SPT safety factors, and 5 sites indicated similar results.

Uncertainty of the Approach

It is important to note the uncertainties regarding the depth of liquefiable zones. The analysis indicated soil that may liquefy at significant depths (30 m or deeper). The authors do not have any knowledge of sites in the NMSZ that indicated liquefaction at such a significant depth. The lack of recent large earthquakes in the NMSZ adds to the uncertainty.

It is of particular importance to recognize that the CRR – V_{s1} relationship was developed using case history data from 26 earthquakes in California, Japan, China, Taiwan, and Idaho. Only seven of the 26 earthquakes had a moment magnitude (M_w) larger than 7.0 (Andrus and Stokoe [20]). The data were limited to average depths of less than 10 m. It is not known how well this correlation would fit with an earthquake of magnitude of 7.5 in the NMSZ. Andrus and Stokoe advised that care should be exercised when applying this approach where site conditions are different from the general characteristics of the case history data. This also should be applied upon performing the analysis using the SPT method. It is not known how suitable the upper (limiting) values for V_{s1} would be for the study area, as they were developed using data from shallower depths and mainly smaller earthquake magnitudes.

Safety Factor Discussion

Andrus, Stokoe, and Juang [21] stated that it is possible that liquefaction could occur when the safety factor is larger than 1.0 (outside the region of predicted liquefaction). Juang, Andrus, Jiang, and Chin, [26, 27] developed the concept of probability of liquefaction (P_L), which can be used to quantify and to establish an important link between the deterministic and probabilistic methods for determining the potential for liquefaction. According to Andrus, stoke, and Juang [21], the CRR- V_s curves (Andrus and Stokoe [20]) correspond to P_L of approximately 0.26, and the SPT-based procedure (Seed, Tokimatsu, Harder, and Chung [28]) corresponds to P_L of approximately 0.31. This means that if the safety factor against liquefaction is equal to 1.0, the probability of liquefaction will be 26% for the V_s method and 31% for the SPT-based method of analysis. Moreover, the Building Seismic Safety Council [29] has suggested a safety factor of 1.20 to 1.50 when the simplified procedure is applied in engineering practice. These safety factors are equivalent to P_L values of 0.16 and 0.08 when applying the V_s-based procedure (Andrus, Stokoe, and Juang [21]).

6. CONCLUSION

The results of the liquefaction analyses indicated that the tested bridge sites in northeast Arkansas may experience significant liquefaction problems during a seismic event that approaches the magnitude of the AASHTO LRFD design earthquake. Comparing the 2 methods of liquefaction potential analysis indicated that the V_s-based is less conservative. This is based on the higher safety factors; lower LPI values, and the lower evidence of liquefaction below a depth of 20 m. Evaluation of the SPT-based analyses indicated significant variability when several SPT profiles are considered. This was reflected by the STDEV parameter shown in Table **7**.

7. RECOMMENDATIONS

The following was recommended upon conducting a liquefaction hazard analysis for bridge sites in Northeast Arkansas. Great emphasis should be put on regularly calibrating the SPT hammers to obtain accurate efficiencies and minimize the equipment-based variability.

A. Noncritical or Nonessential Bridges:

Perform a geotechnical investigation at the bridge site. The investigation should include boreholes drilled to a minimum depth of 30 m. Perform in-situ testing (SPT or combination of SPT and CPT). Perform a liquefaction analysis using the SPT approach. Use a minimum safety factor of 1.20 to determine the potential liquefiable zones.

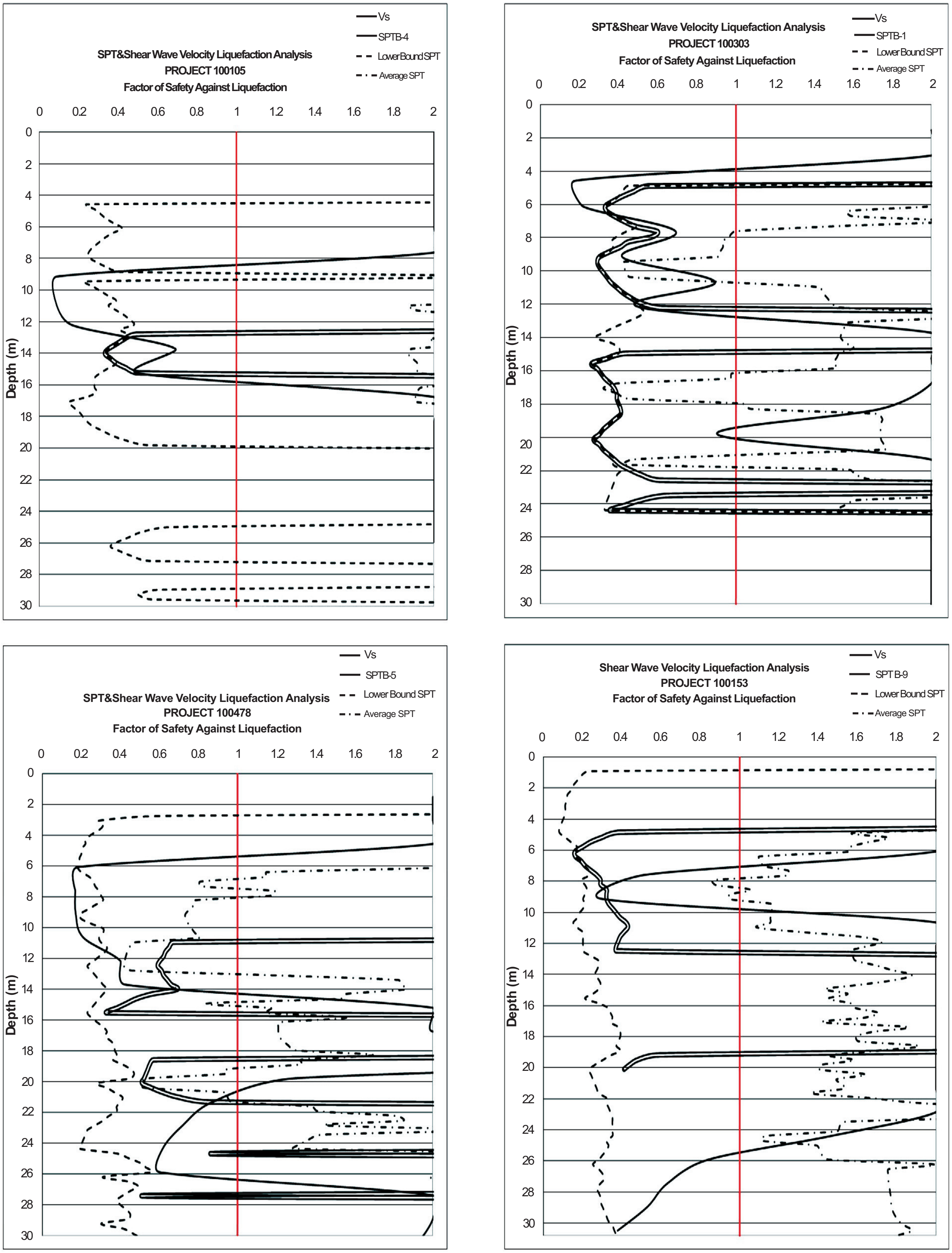

Fig. (10). Shear wave velocity *vs*. SPT plots from sites 1001105, 100478, 100303 and 1003.

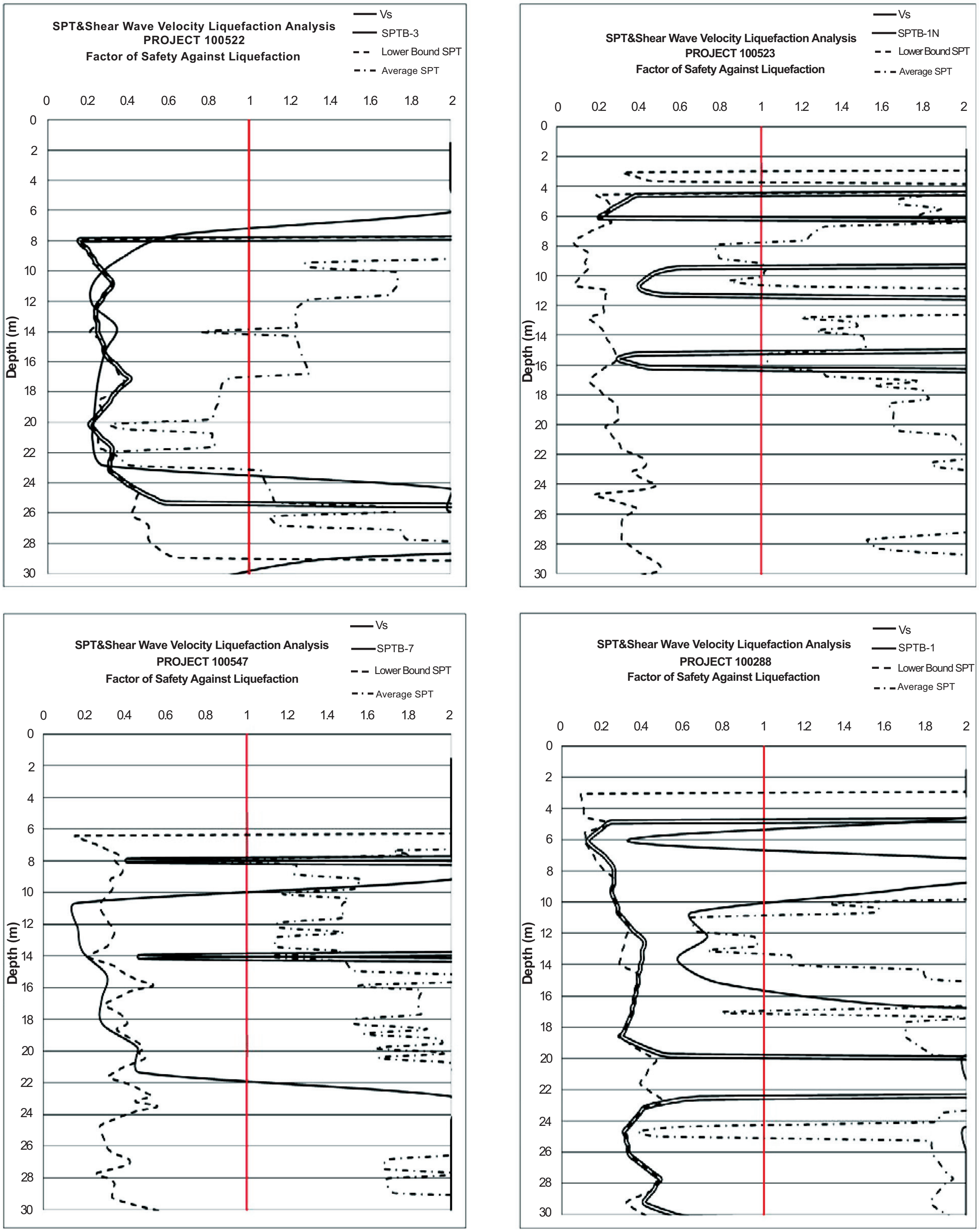

Fig. (11). Shear wave velocity *vs*. STP plots from sites 100522, 100523, 100547 and 110288.

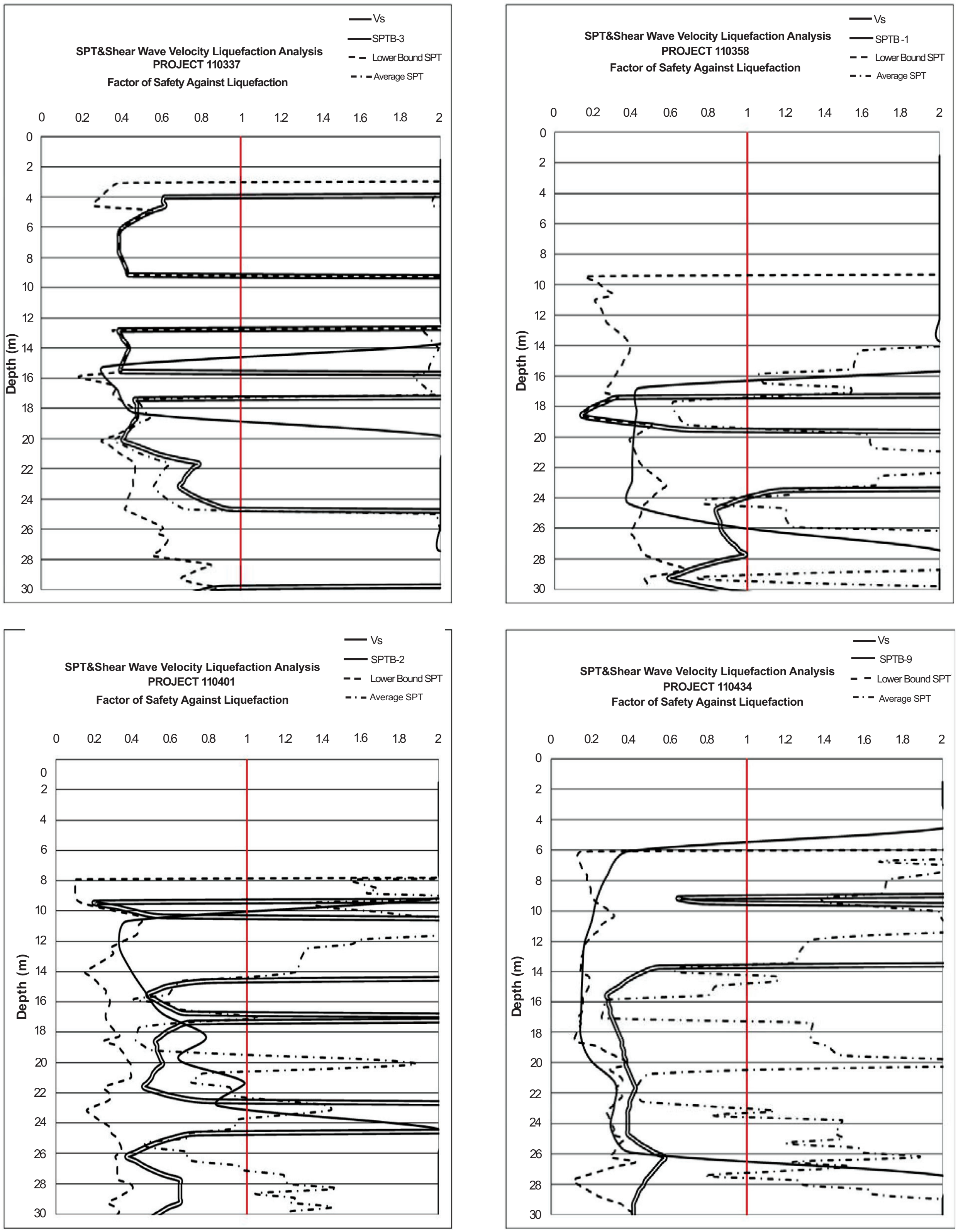

Fig. (12). Shear wave velocity *vs*. SPT plots from sites 110337, 110358, 110401 and 110434.

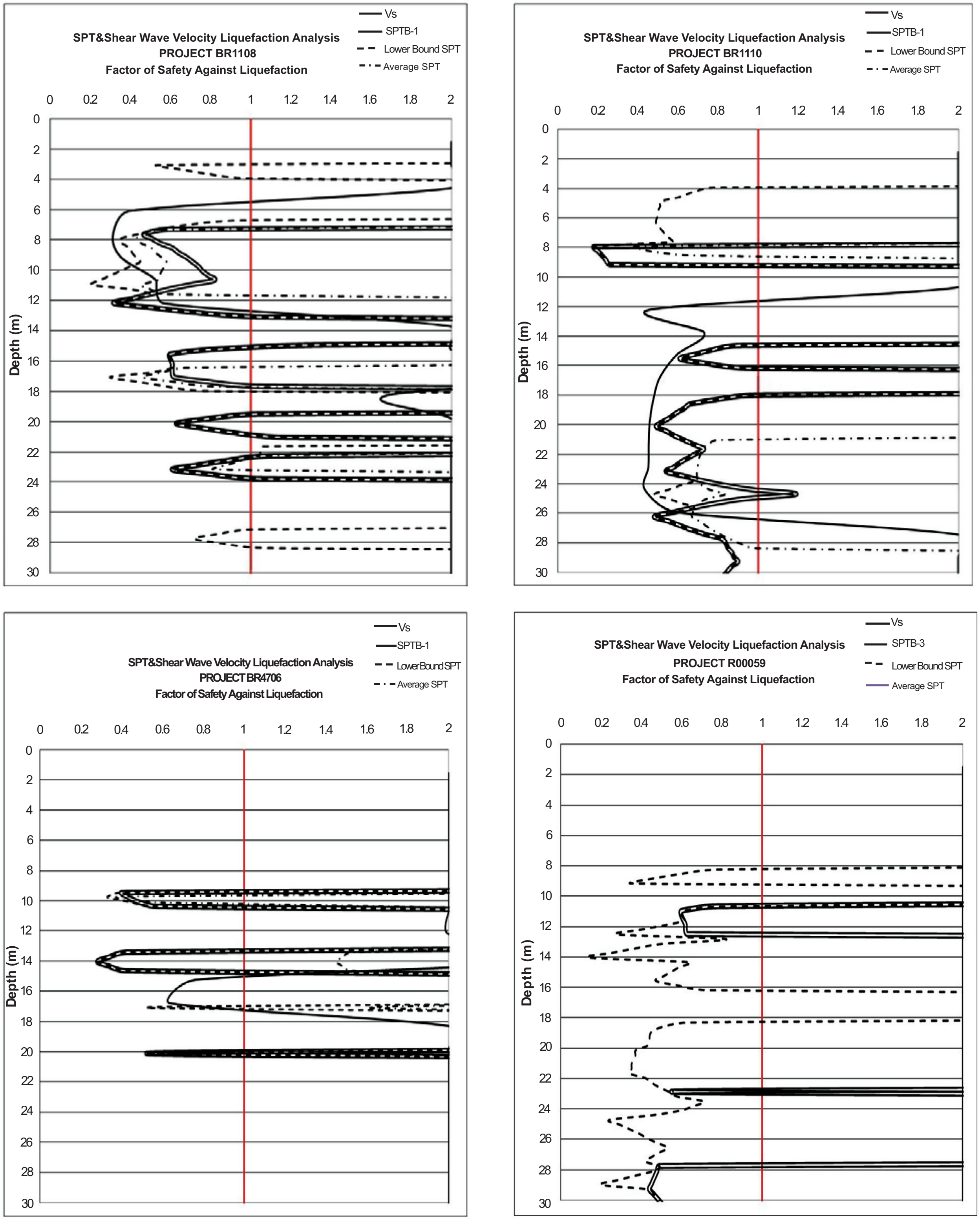

Fig. (13). Shear wave velocity *vs*. SPT plots from sites BR1108, BR1110, BR4706 and R00059.

Table 7. Comparison of Safety Factors – SPT vs. V_s

Site No.	No. of Borings	V_s versus SPT - Safety Factor Comparison (Source of Lower Value Noted)			Level of Variability Within SPT Analysis	Evidence of Liquefaction Below 20 m
		V_s versus Rep. Boring (RB)	V_s versus Lower Bound SPT (LB)	V_s versus Average SPT (AVG)		
110288	10	RB	LB	AVG	Significant STDEV=1.472	All SPT analyses indicated SF <1.0
110358	10	RB	LB	V_s	Significant STDEV=1.222	Both V_s and SPT analyses from all borings indicated SF <1.0
110401	14	RB	LB	Similar	Slight STDEV=0.911	Both V_s and SPT analyses from all borings indicated SF <1.0
100547	7	V_s	LB	V_s	Significant STDEV=1.256	Both V_s and SPT analyses from 5 borings indicated SF <1.0
110337	3	RB	LB	AVG	Significant STDEV=1.419	All SPT analyses indicated SF <1.0
110434	9	Similar	LB	Similar	Significant STDEV=1.172	Both V_s and SPT analyses from all borings indicated SF <1.0
100523	9	RB	LB	AVG	Significant STDEV=1.206	SPT analyses from 5 borings indicated SF <1.0
100522	5	Similar	LB	V_s	Slight STDEV=0.932	Both V_s and SPT analyses from all borings indicated SF <1.0
BR4706	2	RB	LB	Similar	Negligible STDEV=0.165	SPT analyses from one boring indicated SF <1.0 below 20 m
100105	8	V_s	LB	V_s	Slight STDEV=0.896	All SPT analyses indicated SF <1.0
R00059	8	RB	LB	AVG	Significant STDEV=1.369	SPT analyses from 4 borings indicated SF <1.0
100303	4	RB	LB	AVG	Slight STDEV=0.817	All SPT analyses indicated SF <1.0
100153	8	V_s	LB	V_s	Significant STDEV=1.337	Both V_s and SPT analyses from 4 borings indicated SF <1.0
100478	8	Similar	LB	Similar	Significant STDEV=1.420	Both V_s and SPT analyses from all borings indicated SF <1.0
BR1108	2	RB	LB	AVG	Significant STDEV=1.006	All SPT analyses indicated SF <1.0
BR1110	2	Similar	Similar	Similar	Significant STDEV=1.250	Both V_s and SPT analyses from all borings indicated SF <1.0

B. Critical or Essential Bridges:

Prior to performing the geotechnical investigation, perform a sufficient number (a minimum of 2, depending on the total length of the bridge) of non-invasive testing, such as MSASW, to determine the shear wave velocity profile along the route of the proposed bridge.

Obtain the design peak horizontal ground acceleration using the AASHTO design guide and the latitude/longitude coordinates of the midpoint of the array. Perform preliminary liquefaction analyses using V_s-based approach. Make reasonable assumptions regarding the soil unit weights, fine contents, and depth to groundwater table based on past experience. Calculate the corresponding LPI values using the safety factors produced by the analysis. If the LPI values exceed 15 on a consistent basis, performing invasive tests (such as crosshole or downhole testing) during the geotechnical investigation and verify the profiles already obtained from the noninvasive tests.

Perform a geotechnical investigation at the bridge site. The investigation should include boreholes at a minimum depth of 30 m. The SPT intervals should not exceed 1.5 m. Obtain representative samples and perform sufficient classification tests in the laboratory.

Once the laboratory test results are available, re-run the V_s-based analysis and perform SPT-based liquefaction analysis to confirm the findings. Use a minimum safety factor of 1.20. Calculate the corresponding LPI values. If the significantly high LPI previously calculated from the preliminary V_s-based analyses are confirmed, proceed with the invasive testing.

If the liquefiable layer extends to a great depth (24 m or more), perform a site-specific seismic study using the results of the invasive or the noninvasive tests. The invasive test results are preferred. Such a study may result in reducing the design seismic acceleration to 2/3 of their published values within a certain range of periods per the AASHTO design guide. Revise the liquefaction analyses using the results of the site-specific study.

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