Site Specific Ground Response and Liquefaction Analyses for a Project Site in New Madrid Seismic Zone

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ABSTRACT: With the rapid pace of industrialization, structures are being designed and constructed in the flood plains of major rivers. In high seismicity areas, such as the New Madrid seismic zone (NMSZ), International Building Code (IBC) recommends that a site specific ground response analysis be performed if the site soils have potential for liquefaction. For projects in the NMSZ one of the major challenges for performing site specific seismic analysis is the lack of recorded ground motion data. Therefore, synthetic time histories need to be generated using the attenuation models applicable to the region. This paper provides detailed information about site specific shear wave velocity measurements using downhole method and development of site specific seismic parameters to be used for the design of structures at a project site in the bootheel area of Missouri, USA which is located in the NMSZ.

INTRODUCTION, GENERAL GEOLOGY, AND SEISMICITY

Damaging earthquakes occur infrequently in the Central United States (CUS). The earthquakes of 1811-1812 caused damage in the St. Louis area, at least 175 miles from the main-shock epicenters. However, because of the sparse population and simple, log cabin structures in the region during this era, a relatively small number of deaths and minimum property loss was observed. The earthquakes of 1811-1812 caused liquefaction and landslides in an area of 6,000 square miles in southeast Missouri, western Tennessee, and northeastern Arkansas. Although, surface indications of liquefaction during these earthquakes are rare in the St. Louis metropolitan area, any liquefaction below the ground surface today is likely to cause significant loss of life and property (Kumar 2001).

The site is located in the boot heel area of Missouri which lies near the northern edge of Mississippi embayment. The Mississippi embayment is a physiographic feature in the south-central United States which is essentially a northward continuation of the Mississippi River delta. The embayment is a topographically low lying basin that is filled with tertiary to recent sediments. The New Madrid seismic zone (NMSZ), also known as the Reelfoot Rift or New Madrid Fault Line, lies at the northern end of the embayment. The NMSZ extends southward from Southern Illinois, through the Missouri boot heel and western Kentucky, into northwestern Arkansas. The fault zone in this area is predominantly characterized by high-angle normal faults. Figure 1 shows the epicenters of various earthquakes recorded in the vicinity of the site. The size of the circle is related to the magnitude of the earthquake as shown in the legend for Figure 1.

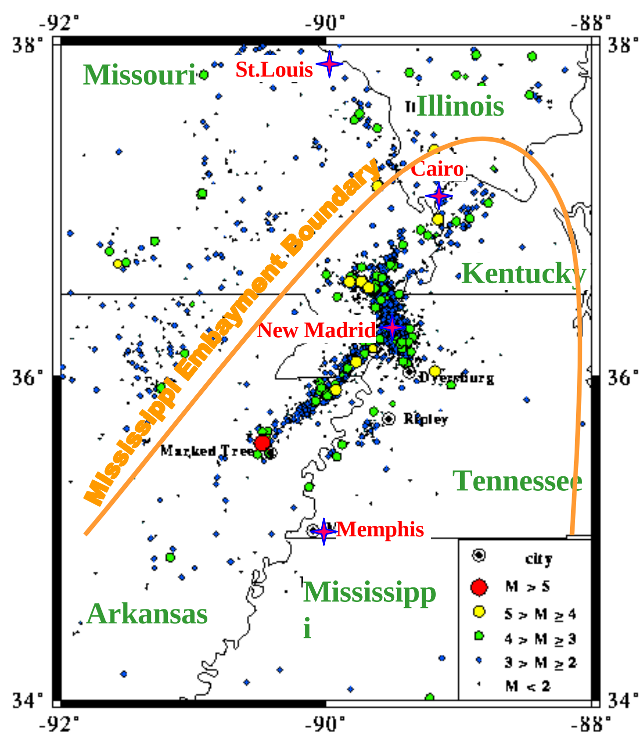

Figure 1. Seismicity in General Vicinity of the Site

Three earthquakes of magnitude 7 to 8 occurred within this seismic zone during the winter of 1811-1812. Presently, the New Madrid area contains the highest level of seismicity in the central and eastern parts of the United States. Paleo-seismic studies suggest that the region has experienced several major prehistoric earthquakes with an approximate recurrence interval of 500+ years. However, it is important to note that three of the largest earthquakes in the central United States during the 20th century were not on the New Madrid fault. Two were on the Wabash Valley fault, which runs approximately north-south from the Ohio River along the Illinois-Indiana state line and the third occurred on the Cincinnati Arch near Sharpsburg, Kentucky. The largest earthquake from the New Madrid fault in the 20th century was in 1976 near Marked Tree, Arkansas (CUSES 1994).

For structural design purposes, the loads imparted to the structure are derived through elastic dynamic structural analysis procedures such as the equivalent lateral force or modal analysis, or if a more advanced dynamic structural analysis is required, by using a procedure such as an inelastic response history analysis. The equivalent lateral force and modal analysis procedures use the response spectrum derived from either code based or site specific methods, to evaluate the base shear force. The inelastic response history method uses time histories, either modified recorded time histories or synthetic time histories, to evaluate the seismic load demand. For this project, a site specific seismic study was performed to produce a smooth, uniform hazard, response spectrum based on the seismic parameters used in the International Building Code (IBC, 2006) which include: seismic hazard related to 2 percent probability of exceedance in 50 years (i.e., 2500-year return period) and 5 percent damping for a single degree of freedom structure.

SUBSURFACE CONDITIONS

In general, the soil stratigraphy at the site consists of intervening layers of brown and gray, clay, sandy clay, sandy silt, and silty sand to depths of 18 to 20 feet. Below this stratum, the soil layer consists of gray, loose to medium dense, fine to medium sand with intervening clay layers down to the maximum depth explored, i.e. 100 feet. The groundwater was encountered at depths between 8.5 and 11 feet during drilling.

SHEAR WAVE VELOCITY MEASUREMENTS

Measurement of shear wave velocity at the site was accomplished by the Downhole testing method complemented by ReMi and SASW methods. However, information from only Downhole testing is presented here.

Downhole Testing

The initial step in performing the site specific seismic study was to perform downhole tests to determine the shear wave velocity profile of the upper soil strata. The downhole tests were performed using a borehole which was cased to a depth of 100 feet below the ground surface.

Downhole testing is the most widely used method for measurement of shear wave velocities. The vertical path of wave propagation is orthogonal to the wave direction in a crosshole survey; however, it is the correct direction for earthquake waves propagating upward from bedrock (Woods, 1978). Using the downhole survey, low velocity layers can be detected even if trapped between higher velocity layers (provided the receivers or geophones spacing is close enough). High-resolution digital recorders are usually used to produce an overall average velocity-depth profile as well as it can be used with interval measurements to produce detailed profiles (EPRI, 1993).

The downhole test is performed using an impulsive energy source located at the ground surface near the borehole. This source generates shear waves which propagate in a radial pattern from the source. The impulse can be produced by a hammer blow or by an automated source. Multiple receivers (geophones) are lowered into the borehole and positioned at pre-selected depths. The recorded data are then plotted on travel time plots. Figure 2 presents a typical seismic downhole test setup.

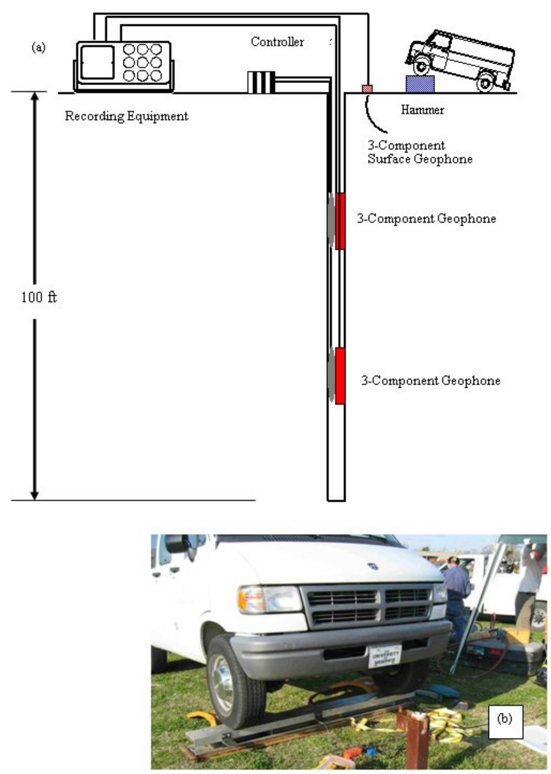

Figure 2. (a) Seismic Downhole Velocity Measurement Test Setup; and (b) Actual Shear Hammer

The simple method of striking a plate with a hammer is quite effective; however, the wave that is generated is not repeatable. For this study, a shear hammer was used. The principle of the shear hammer is to create the desired shear wave with the use of pneumatic cylinder to drive a steel block into an anvil. The driving force comes from a double acting air cylinder mounted on an anvil. A hammer is attached to the end of the air cylinder piston rod and slides on linear bearings and low-friction tracks (Liu, et al., 1997, Pezeshk, et al., 1998). In the forward stroke, the hammer impacts the end anvil and when retracting it impacts the central anvil. Two aluminum channels bolted to the top and the bottom of the anvils serve as the base and the cover of the shear wave source. The motion of the hammer is initiated by letting compressed air into the chamber on either side of the air cylinder piston. The impact of the hammer on the anvil results in a traction exerted on the ground surface by the base channel. The resulting wave which is generated is highly repeatable (Liu, et al., 1997).

The downhole geophones (seismometers) used for this project were Geostuff Model BHG-2. These geophones have three channels, one vertical and two horizontal components. Each geophone has a resonant frequency of 10Hz. These geophones use a clamping mechanism consisting of a steel spring compressed by a DC electric motor. One three-component surface geophone was placed on the ground surface between the shear wave source and the borehole.

Seismic signals were collected using the downhole geophones at 5-foot spacing intervals throughout the entire depth of each borehole. Four tests were conducted at each depth for redundancy. The raw data collected from the field were interpolated to achieve much higher resolution using the Fast Fourier Transform (FFT) technique (Liu, et al., 1997) and the wave travel times were corrected based on the geometry of the test setup. Table 1 shows the shear wave velocity measured at various depths.

Low Strain Soil Shear Modulus

A key parameter necessary to evaluate dynamic response of soils is the dynamic shear modulus, G_s or shear wave velocity which is also related to dynamic shear modulus. Shear modulus is not a constant property of soil but decreases nonlinearly with increasing strain. For initial design purposes, shear modulus measured at small shear strain amplitudes (less than 10^{-4} percent), referred to as G_{max}, is a desired design parameter. The shear modulus, G_{max}, corresponding to small shear strain was calculated using the shear wave velocities measured at the site.

Damping

The inelastic behavior of soil also gives rise to energy absorption characteristics of soil which is known as material damping. Damping is generally expressed as percentage of the critical damping. Low strain damping of approximately 5 to 10 percent of the critical damping is commonly used for soils. Damping of 5 percent of critical was used for the analysis. However, this damping was modified in the analysis based on the strain levels in the soil.

Table 1. Shear-Wave Velocities Measured at the Boring B-3 Location.

Depth *(ft)*	**Velocity** *(ft/sec)*
2.5	98
7.5	169
12.5	338
17.5	462
22.5	544
27.5	659
32.5	578
37.5	566
42.5	878
47.5	485
52.5	841
57.5	768
62.5	580
67.5	806
72.5	688
77.5	577
82.5	497
87.5	626
92.5	760

Effect of Strain on Shear Modulus and Damping

It is well understood that the stress-strain relationship of soils is nonlinear. This means that the soil shear modulus and damping are not constant values but degrade nonlinearly with increasing strain in the soil. Dynamic analyses considering true nonlinear behavior of soil are very complicated and are still not fully developed. Therefore, equivalent nonlinear analysis is most commonly used in practice. Equivalent nonlinear analyses consists of performing a series of linear analyses, in an iterative way, using, for each analysis, soil properties consistent with the strains resulting from the previous one. Equivalent nonlinear analysis was used in the present study. Many studies have been performed in the past to establish a relationship between modulus degradation with strain. The shear modulus degradation curves and damping ratio curves used were taken from Pezeshk et al. (1996) and Chang et al. (1989).

ANALYSES FOR EXISTING SUBSURFACE CONDITIONS

Figure 3 presents the measured N-values (N_{msd}) at the site. The N-values were corrected for the overburden and hammer energy, assuming the efficiency of the automatic hammer used to be 75 percent. The average N-value for this site ($\overline{N}$) as per the recommendations of IBC 2006 was calculated to be 12. The average shear wave velocity for this site ($\overline{V}_s$) as per the recommendations of IBC 2006 was calculated to be 466 ft/sec. As the IBC 2006 bases the site classification on the average properties in the top 100 feet, the site class for the site was identified to be a Site Class "E" according to the $\overline{V}_s$-value (Table 1613.5.2). According to Tables 1613.5.3(1) and 1613.5.3(2) and the mapped spectral acceleration, the site coefficients F_a and F_v for Site Class "E" were taken as 0.9 and 2.4, respectively.

GROUND RESPONSE ANALYSIS

Ground response analysis was performed to obtain representative response spectra at the ground surface based on the time histories at B-C boundary propagated through the site soils. According to the United States Geological Survey (USGS) Hazard Maps, the project location has a mapped 0.2 second spectral response acceleration (S_s) of approximately 3.259g and a mapped 1.0-second spectral response acceleration (S_1) of approximately 1.101g. The maximum considered earthquake spectral response acceleration for short period, S_{MS}, and at 1-second period, S_{M1}, adjusted for site class effect are determined to be:

$$S_{MS} = F_a S_S = 0.9 \times 3.259 = 2.933$$
$$S_{M1} = F_v S_1 = 2.4 \times 1.101 = 2.641$$

Five percent damped design spectral response acceleration at short period, S_{DS}, and at 1-second period, S_{D1}, are determined to be:

$$S_{DS} = \frac{2}{3} S_{MS} = 1.955$$
$$S_{D1} = \frac{2}{3} S_{M1} = 1.761$$

Horizontal bedrock time histories were generated at the site from a seismologically-based model mainly due to shear waves generated from a seismic source. The seismologically-based model used included effects of attenuation, characteristics of the source zone, recurrence interval, and the seismotectonic setting of the New Madrid Seismic zone, Wabash zone and other potential seismic sources in the region.

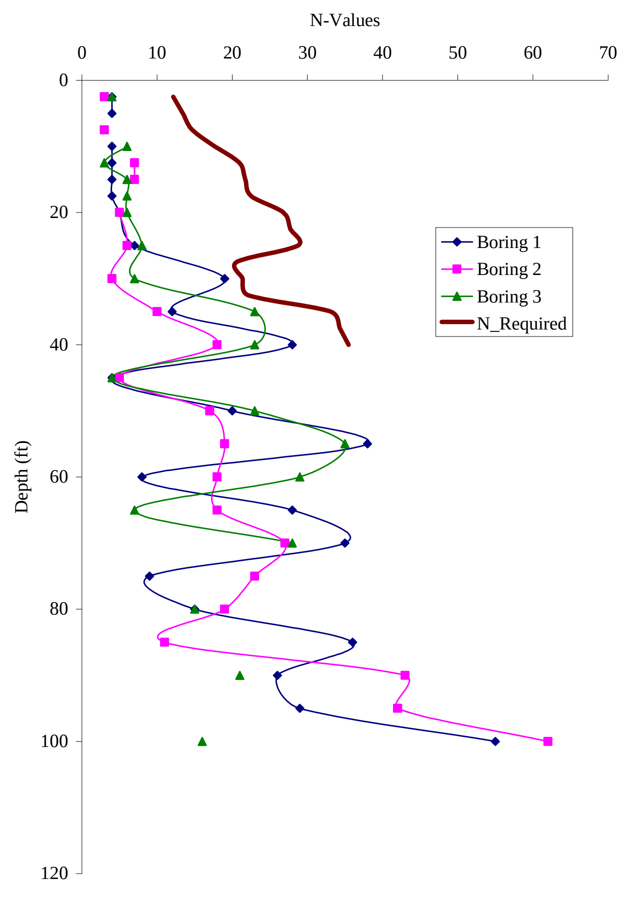

Figure 3. N-Values Measured and those Required to Reduce Liquefaction Potential

Site Specific Analysis Results

According to the results of the probabilistic hazard study, the design spectral accelerations, S_{DS} and S_{D1}, for the existing soil conditions are determined to be 1.247 g and 2.185g, respectively for the 2% probability of exceedance in 50 years. However, IBC 2006 recommended that the site specific acceleration coefficients not to be lower than 80% of the code acceleration coefficients. Furthermore, S_{DS} obtained from site specific at a period of 0.2 s, shall not be taken less than 90% of the peak spectral acceleration at any period larger than 0.2 s. Therefore, the site design spectral accelerations, S_{DS} = 1.966 and S_{D1} = 2.185g, were recommended for the existing soil conditions. The design response spectrum using these values and the design response spectrum for Site Class "E," developed as per IBC 2006 are shown in Figure 4.

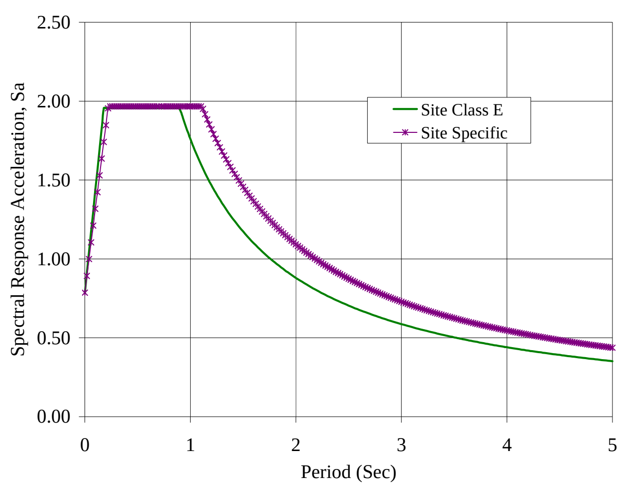

Figure 4. Design Response Spectra for Existing Soil Conditions

LIQUEFACTION ANALYSIS

Liquefaction is a phenomenon of loss of shear strength of saturated soils due to the sudden increase in pore pressures. Generally, loose cohesionless soils are susceptible to liquefaction. However, studies have shown that certain low plastic clayey soils may also suffer strength loss during and immediately after an earthquake.

Subsurface exploration at the site indicated that the existing soils are primarily loose to medium dense sands except the surface stratum which consists of intervening layers of brown and gray, silty clay, sandy clay, sandy silt, and silty sand. Groundwater was encountered at depths between 8.5 and 11 feet at the time of exploration which fluctuates depending on the water levels in the Mississippi River.

According to IBC 2006, Peak ground Acceleration (PGA) at the B/C boundary and the ground surface may be taken as 1.697g and 1.527g, respectively. However, based on the site specific analysis PGA at the B/C boundary and the ground surface was calculated to be 1.488g and 1.222g respectively. Because of the presence of low density, saturated sands having relatively uniform grain size distribution, and the level of ground shaking expected at the site from an earthquake, the site has significant potential for liquefaction. Analysis was performed to determine the density of sands required to reduce the potential of liquefaction. These densities were then compared with the densities of the existing soil to determine the liquefaction potential of the site.

Liquefaction analysis was performed using the simplified method originally proposed by Seed and Idriss (1971, 1982) and Seed et al., (1983) which is based on in-place evaluation of resistance of soils. Simplifications and modifications proposed by Youd et al. (2001) were used to perform the liquefaction analysis. This method is based on the extensive analysis of field data from sites which liquefied or did not liquefy in various earthquakes in the past. The procedure consists of comparing the shear resistance of the soil (in terms of corrected blow count, $(N_1)_{60}$) to the cyclic shear stresses expected from the design level earthquake.

To determine the liquefaction at the site, the corrected number of blows $[(N_1)_{60}]$ required at any depth to reduce the liquefaction were estimated using the simplified procedure. Table 2 and Figure 3 show the corrected $(N_1)_{60}$ measured during the subsurface exploration and corrected $(N_1)_{60}$ required to reduce the liquefaction potential at the site.

Based on the results of the liquefaction analysis performed, it was concluded that the site has significant potential for liquefaction. Therefore, improvement of the site to reduce liquefaction potential was recommended.

CONCLUSIONS

Results of shear wave velocity measurements using downhole method, a site-specific ground response using site-specific seismic parameters, and liquefaction analyses performed for a site in the boot heel area of Missouri are presented. In addition, the recommended values of design spectral accelerations, S_{DS} and S_{D1} for the site and a site-specific response spectra for existing soil conditions are presented in this paper. The liquefaction analysis showed that the existing soils at the site had significant liquefaction potential.

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Table 2. Results of Liquefaction Analysis

Depth (ft)	Measured N-values		Required N-Values	
	Uncorrected	Corrected	Uncorrected	Corrected
2.5	4	5	10	20
5	4	5	11	20
7.5	4	5	12	20
10	4	5	14	22
12.5	4	5	17	25
15	4	5	17	25
17.5	4	5	18	25
20	5	6	21	29
22.5	6	8	22	29
25	7	9	23	29
27.5	19	24	16	20
30	19	24	17	20
32.5	12	15	18	20
35	12	15	26	29
37.5	28	35	27	29
40	28	35	28	29
42.5	4	5	20	20
45	4	5	21	20
47.5	20	25	31	29
50	20	25	32	29

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