Interpretation of Ground Shaking from Paleoliquefaction Features

Grant Award 01HQGR0030

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NEHRP Element: II (Memphis Metropolitan Area)
Keywords: Paleoliquefaction, Strong Ground Motion, Earthquake Scenarios

Non-technical Summary

Evidence in the geologic record can be used to determine the magnitude of earthquakes that occurred before the advent of seismic recording devices. Marginal liquefaction represents the threshold where the driving forces caused by earthquake shaking are essentially equal to the resisting strength of the soil. If the resisting strength of the soil at a site of marginal liquefaction is known, the driving force, i.e., the maximum acceleration and magnitude of the earthquake, can be back-calculated. The main objectives of this study are to develop a procedure to back-calculate the magnitude and acceleration of an historic earthquake using paleoliquefaction features and to use the procedure to evaluate the creation of the paleoliquefaction features found near Memphis, Tennessee and at various locations in the New Madrid Seismic Zone. The results of this procedure are used to estimate the magnitude and acceleration of the New Madrid earthquakes of 1811 –1812.

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Introduction

Evidence in the geologic record, e.g. buried liquefaction features, can be used to determine the magnitude of earthquakes that occurred before the advent of seismic recording devices. Marginal liquefaction represents the threshold where the driving forces caused by earthquake shaking are essentially equal to the resisting strength of the soil. If the resisting strength of the soil at a site of marginal liquefaction is known, the driving force, and thus the maximum acceleration and magnitude of the earthquake, can be back-calculated. Sites of no liquefaction and extensive liquefaction can be used to provide a lower bound and upper bound, respectively, of the earthquake magnitude and acceleration.

Field Testing

The main objective of the study is to develop a procedure to back-calculate the magnitude and acceleration of the New Madrid earthquakes of 1811-1812 using marginal paleoliquefaction features found near Memphis, Tennessee and sites of extensive, marginal, and no paleoliquefaction features found at various locations in the New Madrid Seismic Zone (NMSZ). In the vicinity of Memphis, Dr. Stephen Obermeier conducted a field search to locate sites exhibiting features of marginal liquefaction during the first year of this project. The selected sites are along the Wolf River, in the eastern suburbs of Memphis, between the towns of Germantown and Collierville. The sites contain a number of small dikes extending upward into the overlying clay cap, such as the one shown in Figure 1, and other small features that are deemed to be manifestations of marginal liquefaction.

Figure 1. Small sand dike cutting into clay cap is evidence of marginal liquefaction at Wolf River test site AA3

In the NMSZ, nine separate sites are used in this study. Eight of these sites have been identified by prior researchers (Tuttle 1999) as exhibiting either marginal or extensive paleoliquefaction features. The ninth site is a site of no paleoliquefaction.

Field cone penetration testing was conducted at the Wolf River and NMSZ study sites to obtain information about the current strength of the soil. In particular, data from forty-three truck-mounted continuous cone penetration tests (CPT) taken near the sites were obtained for this study. At the Wolf River test site near Memphis, a portable dynamic cone penetrometer (DCP) was also used to obtain readings next to or in the liquefaction features that were located in the river banks and thus were inaccessible to the truck-mounted CPT rig. In order to correlate the DCP test data to equivalent CPT values, DCP testing was conducted adjacent to two of the CPT test sites. A correlation that converts DCP (blow count or N) values to equivalent CPT (q_c) penetration resistance was developed during this study that allows the portable and more cost-effective DCP to be used in the Memphis area. As this research continues, a similar correlation will be developed for the NMSZ so the DCP can be used to evaluate liquefaction potential and perform back-calculations using paleoliquefaction features in the NMSZ.

At the Wolf River test site near Memphis, samples of the sand in the liquefaction features and in the source beds for the features were also collected during the DCP tests and were used to conduct grain size analyses. The range of grain size distributions developed for the soil samples falls completely within the boundaries for most liquefiable soils as defined by Ishihara et al. (1989). This supports the field observations and confirms that the chosen test sites represent locations of previous marginal liquefaction.

<u>Determination of 1811-1812 In-Situ Condition</u>

In order to determine the driving force that would have caused liquefaction at a given site, it is necessary to make a determination of what the resisting strength of the soil was just prior to the earthquake shaking. To relate the current penetration resistance values to values of penetration resistance prior to the earthquake, it is necessary to account for the processes that the soil has been subjected to since the earthquake. These processes include aging of the soil over time, densification due to the liquefaction event, and effects caused by lowering of the groundwater table, e.g. due to drainage and/or flood control measures.

The phenomenon of soil aging results in soils gaining strength over time. This process is well recognized but attempts to quantify this strength gain are preliminary and ongoing. Three previous studies developed expressions to predict the increase in penetration resistance for sands over time. These three expressions were applied for the 189-year time period since the New Madrid earthquakes of 1811-1812 and yielded expected increases in cone penetration resistance, i.e. soil strength, of 176% to 392%. It was decided to use the expression by Mesri et al. (1990) for this study because it yields the most conservative value of increase in penetration resistance to account for aging effects.

The process of liquefaction also leads to densification of the soil after earthquake shaking ceases and thus increased penetration values. Review and analysis of Standard Penetration Test (SPT) N values measured before and immediately after the occurrence of liquefaction (so little if any aging had occurred) for three Japanese earthquakes shows that the N values increased by about 25%. Relating a 25% increase in N values to a corresponding decrease in void ratio through the use of relative density relationships (USACE 1993) shows an expected decrease in void ratio of 4-10%, depending on the initial value of N and the level of earthquake shaking. To corroborate this 4-10% decrease in void ratio, the results of three studies that measured the change in void ratio due to liquefaction were utilized. Laboratory testing as well as field testing conducted for these studies show a decrease in void ratio of 10-13% after the occurrence of liquefaction. As a result, there is agreement between the field and laboratory testing that shows densification by liquefaction causes a decrease in void ratio of approximately 10%, which corresponds to a 25% increase in the N value.

Another factor that has a marked influence on liquefaction susceptibility is the depth of the water table. Because liquefaction can only occur in saturated soils, it is important to determine the depth of the water table at the time of the earthquake in question. At the Wolf River test area east of Memphis, a recent downstream channelization project has caused downcutting in this portion of the river. Field observation of the resulting exposed banks led to the determination that the depth of the water table at the time of the 1811 – 1812 earthquakes was about 3 m. This is shallower than the water table depth of 6 m measured during CPT testing in the summer of 2000. Lowering of the water table can also result in increased penetration resistance values due to mobilization of total instead of buoyant unit weight, negative pore pressures and capillary tensions, which increase effective confining pressures, as described by Hryciw and Dowding (1987). This effect may account for the increased penetration resistance values observed in the CPT soundings for the sands located in the 3 to 6 m depth range at the Wolf River test site. To reverse this increased penetration resistance due to groundwater lowering and return the soil strength to its values prior to the 1811-1812 earthquakes, the CPT values in the 3 to 6 m depth range were conservatively assumed to be the same as the lesser CPT values measured in the 6 to 9 m depth range.

Simplified Procedure to Back-calculate a_{max}

A methodology known as the Simplified Procedure (Seed and Idriss 1971) has been a standard for 30 years for evaluating the liquefaction resistance of soils. The method compares two quantities, the seismic demand on a soil layer, known as seismic stress ratio (SSR), and the capacity of the soil to resist liquefaction, expressed as the seismic resistance ratio (SRR), to determine if liquefaction will occur at a given location. A site of previous marginal liquefaction represents a state where the SSR driving force of the earthquake is approximately equal to the SRR resisting strength of the soil, i.e. the threshold for liquefaction occurrence had just been reached. SSR is a function of the maximum acceleration, a_{max}, and magnitude generated by the earthquake. If the SRR resisting strength of the soil just prior to the earthquake is known at a site of marginal liquefaction, the SSR and thus the maximum acceleration and magnitude of the earthquake can be back-calculated.

During this study a Liquefaction Assessment Spreadsheet was developed that makes use of current cone penetration resistance values and the Simplified Procedure to back-calculate a_{max} for an historic earthquake. Digital soundings from CPT testing can be imported directly to the spreadsheet. Penetration resistance (q_c) values and corresponding depths can also be entered manually. The user is required to input the depth of the water table at the time of interest and trial values of a_{max} and magnitude. The spreadsheet normalizes the q_c values and corrects for unequal pore water pressures. The spreadsheet determines the soil classification from measured CPT quantities using the procedure of Robertson (1990). The spreadsheet modifies the values of penetration resistance to reflect the effects of aging and groundwater lowering, if applicable, since the time of the historic earthquake. (Because the Simplified Procedure was developed primarily from case histories of penetration resistance conducted AFTER earthquake/liquefaction events, a correction for densification due to liquefaction is not applicable with this methodology.) The spreadsheet then calculates a factor of safety against liquefaction at each depth using the Simplified Procedure and the relationships between SSR and CPT tip resistance developed by Stark and Olson (1995). If necessary, another trial value of a_{max} is used until the calculated factor of safety at a known depth of marginal liquefaction is approximately equal to unity. The spreadsheet can also be used with CPT soundings and a design value of a_{max} to evaluate current liquefaction potential at a given site.

Using the Liquefaction Assessment Spreadsheet, a preliminary analysis of the data gathered at the Wolf River test site near Memphis yielded values of a_{max} ranging from 0.09g (aging effects included) to 0.26g (no aging effects considered) for a $M_w = 7.5$ earthquake. As this study continues, the data from the NMSZ test sites will be used with the Liquefaction Assessment Spreadsheet and the results will be compared to the results obtained using the Wolf River test site data.

Energy Method to Back-calculate a_{max}

The Simplified Procedure for liquefaction assessment described above was developed based on post-earthquake field observations of liquefaction and no liquefaction at the ground surface at sites in California and the Far East. There is a fundamental difference between the strength of shaking that causes observable liquefaction features at the ground surface (full or extensive liquefaction) and that which causes marginal liquefaction features at depth like those observed at the Wolf River test site. For example, at a marginal liquefaction site small dikes are formed in response to hydraulic fracturing but are not extensive enough to reach the ground surface. Thus, the methodologies applied in developing the Simplified Procedure may not be the most appropriate means of analyzing the forces that led to development of a marginal paleoliquefaction feature at depth. This study will examine the use of an energy-based procedure to evaluate the creation of paleoliquefaction features and to back-calculate the maximum acceleration of the historic earthquake that caused them.

Energy-based techniques are used to quantify the total amount of seismic energy that passed through a soil. Green (2001) labels this seismic energy as the *Demand* placed on the soil and presents a relationship for *Capacity* of the soil (boundary where liquefaction begins) as a function of normalized *Demand* and corrected blow count value, N, as shown in Figure 2.

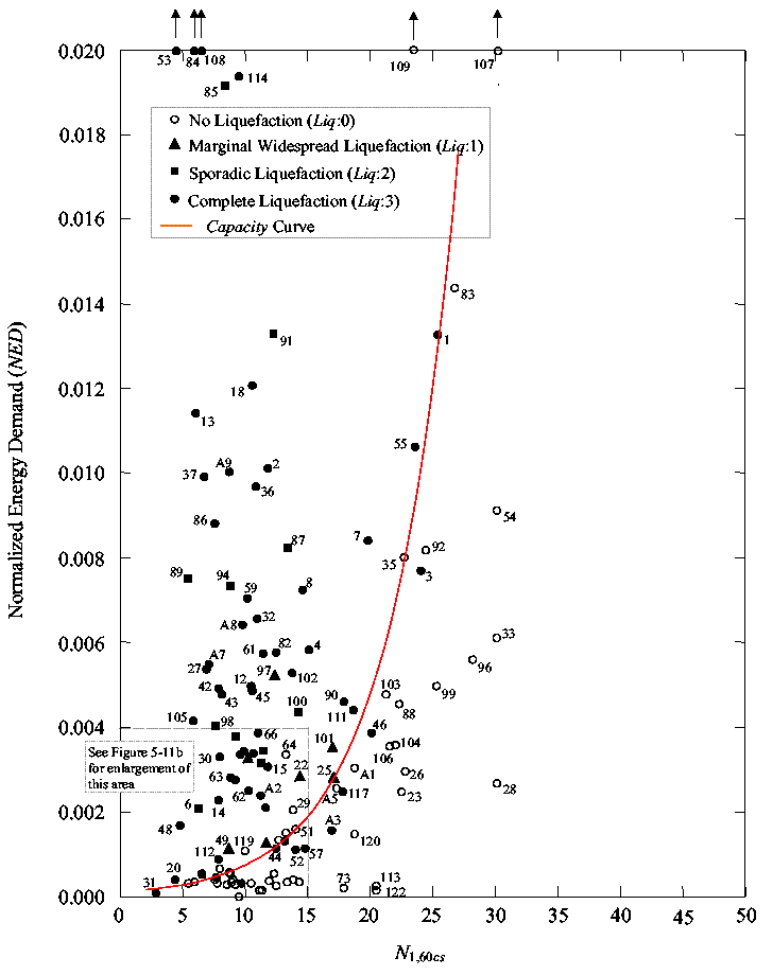

Figure 2. Energy-based *Capacity* curve developed from 126 liquefaction field case histories (Figure 5-11a from Green (2001))

At a site of marginal paleoliquefaction, i.e. a point on the *Capacity* curve, if the historic value of penetration resistance is known, a value of a_{max} for the historic earthquake, which is a function of the *Demand*, can be determined. This study is currently adapting the Green (2001) methodology for use with cone penetration resistance values, q_c. As this study continues, the values of q_c measured at the Wolf River and NMSZ test sites will be adjusted for historical effects and used in the adapted energy-based methodology to determine a range of maximum acceleration for the New Madrid earthquakes of 1811-1812.

Determination of M_w

Once a value (or range of values) of maximum acceleration has been back-calculated for a zone of paleoliquefaction, a corresponding value of maximum bedrock acceleration can be determined using a site response analysis. An attenuation relationship or ground motion model can then be used to trace this bedrock value back to its origin, i.e., the fault rupture, to yield a value of maximum magnitude for the earthquake that caused the liquefaction.

DEEPSOIL, a non-linear site response program described by Hashash and Park (2001), was used in a preliminary analysis with a_{max} = 0.09g-.020g in the liquefied layer at the Wolf River test site as calculated using the Liquefaction Assessment Spreadsheet. The resulting values of maximum bedrock acceleration were used with ground motion attenuation relationships in SMSIM (Boore 2000) to obtain values of M_w at the epicenters of the New Madrid earthquakes of 1811-1812. The preliminary results suggest the closest (to Memphis) New Madrid event (December 16, 1811) could have exhibited a M_w = 7.0 to 7.5 whereas the farthest event (February 7, 1812) could have exhibited a M_w = 8.1 to cause the marginal liquefaction feature at the Wolf River test site. These preliminary analyses illustrate the importance of determining, if possible, which of the three events caused each paleoliquefaction feature instead of grouping them as one event. Recently, Dr. Arch Johnston of the University of Memphis identified the December 16, 1811 event as the cause of the marginal paleoliquefaction features observed at the Wolf River test site (Johnston 2002). Thus, the preliminary analysis yields values of M_w = 7.0 to 7.5 for the December 16, 1811 New Madrid earthquake event.

As this study proceeds, the paleoliquefaction sites in the New Madrid Seismic Zone will be examined using the site response and ground motion analysis procedures described above for the Wolf River test site. Some of the NMSZ sites are locations of full or extensive liquefaction (features penetrating the ground surface). These will be analyzed to set an upper bound on the back-calculated values of magnitude and acceleration. Similarly, adjacent sites where no liquefaction features are observed will be analyzed to set lower bound values. In this way, a range of maximum magnitude will be determined for the New Madrid Earthquakes of 1811 – 1812.

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