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ABSTRACT

In this paper, we present an evaluation of the potential risk of earthquake-induced damage to pile foundations in Indiana. Piles are commonly used in foundations of bridge piers in the southern part of Indiana; the potential seismic sources in this region are the Wabash Valley Fault system and the New Madrid Seismic Zone. Based on in-situ test data for specific sites in southern Indiana, one-dimensional wave propagation analyses are performed. Additionally, the liquefaction potential is estimated based on the calculated acceleration profile. Data on real cases of pile damage due to seismic events are collected after an extensive literature survey. Using this information, the main causes and characteristics of earthquake-induced pile failure are identified, and the conclusions obtained are applied to southern Indiana to make a preliminary assessment of pile damage potential. Simple 3-D numerical analyses for the pile foundations of an existing bridge structure are performed using the finite element method. The results show that, for typical pile foundations and soil profiles in southern Indiana, a credibly large earthquake is capable of producing significant damage to the piles.

INTRODUCTION

The southwestern part of Indiana, bounded by the Ohio and Wabash rivers, is located about 300km from the New Madrid seismic zone. The last significant seismic events produced by the New Madrid seismic zone were the 1811 and 1812 earthquakes of magnitude M_s greater than 8.0, causing significant structural damage (Nuttli, 1982). Several small-to-moderate seismic events have occurred in the vicinity of the Wabash Valley fault system since the 19th century. Extensive evidence of paleoliquefaction found in the alluvial deposits of the southern Indiana and Illinois suggests that earthquakes originating from the Wabash Valley fault system of moment magnitude up to M=7.5 took place in prehistoric times (Obermeier, 1998).

Observations made after recent earthquakes around the world suggest that pile foundations are highly susceptible to damage when subjected to loading induced by major seismic events (Ishihara, 1997; Matsui 1996; Mizuno, 1987; Tazoh 1987; Tokimatsu 1996). Numerous single and multiple-span bridge structures in the southern part of Indiana are founded on piles. Since the occurrence of a large earthquake near this area is possible and given that bridge pile foundations are embedded in loose deposits near the waterfront, there is an issue about the potential of pile damage due to ground shaking and liquefaction.

In the present paper, the possibility of damage to piles in Southern Indiana due to a seismic event is assessed based on the combination of actual data for Indiana's seismicity, geology, and bridge foundation design, as well as on the data obtained from the literature concerning pile damage during earthquakes. First, the ground acceleration and the liquefaction susceptibility are estimated for nine sites selected in the southwestern part of Indiana. Second, a classification of the causes and mechanisms of pile damage is proposed based on an extensive literature review on pile failures. Finally, a finite element model is used to analyze a bridge foundation in one of the selected sites.

GROUND ACCELERATIONS AND LIQUEFACTION POTENTIAL

Seismicity

Two potential seismic sources were considered: (1) the Wabash Valley Fault System (WVFS); and (2) the New Madrid Seismic Zone (NMSZ) (Fig.1). Magnitude-recurrence relationships developed by Kayabali, (1993) yield values of the magnitude m_b for 1000yr earthquakes of 6.9 for the Wabash Valley and 7.4 for the New Madrid seismic zone. The earthquake recurrence model of Green et al. (1988) gives to m_b=6.25 and m_b=6.8, for WVFZ and NMSZ, respectively. Two scenarios are used in this study: (1) WVFZ earthquake with m_b=6.5; and (2) NMSZ earthquake with m_b=7.2, corresponding to a 10% probability of exceedance in 100 years.

Nine sites in southern Indiana are selected for this study (Fig. 2). Seven of them are road bridge sites crossing rivers and ditches, and the other two are located inside Evansville, next to the Ohio River (WH and HP sites in Fig.2). The Nuttli and Herrmann (1984) attenuation relationships, which are the most representative for an earthquake occurring in Central North

America, are used to determine the amplitude of the input bedrock acceleration for the ground response analysis. The distance between the potential sources and the selected sites ranges from 19 to 61km for a WVFS earthquake and from 300 to 400km for a NMSZ earthquake. Although the considered magnitude for a NMSZ earthquake is larger than that of a WVFS earthquake, the peak acceleration produced by a WVFS earthquake is significantly larger due to the proximity of that seismic source to the selected sites (Table 1).

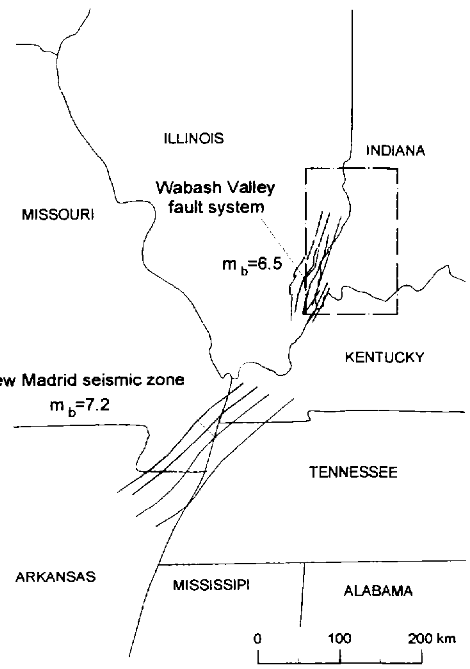

Fig.1. Map showing seismic sources and earthquake magnitudes considered in the study.

Geology and Soil Properties

The bedrock formations in the area consist mainly of limestone, shale and sandstone and are covered by thick soil deposits of alluvial and lacustrine origin. Near the Wabash and Ohio rivers, soils are composed by alluvial sands and silts, as well as by outwash deposits of sand and gravel. In other areas, windblown silt and lacustrine deposits of clays predominate. The thickness of the soil deposits for the nine sites ranges from 9 to 43m (Fig. 2). The soil properties required to perform the site response analyses are assessed from SPT data and other experimental data obtained from the Indiana Department of Transportation (INDOT). In many cases, N_{SPT} blowcounts indicate very loose deposits. The Imai and Tonouchi (1982) relationship between shear wave velocity V_s and blowcounts N_{SPT} is used for all soil types, according to which

$$V_s = 97 \cdot N_{SPT}^{\ 0.314} \quad \text{(m/sec)} \tag{1}$$

In addition, the shear modulus reduction and damping ratio curves by Ishibashi and Zhang (1993) are used.

Site Response Analysis.

The input bedrock ground motion is taken from acceleration records from San Fernando, 1971, for WVFS, and Landers, 1992, for NMSZ earthquakes after scaling both amplitude and time. This scaling is done to be consistent with the PGA values coming from the attenuation relationships and the low predominant period of the Central and Eastern North American earthquakes compared with California earthquakes (Kayabali, 1993).

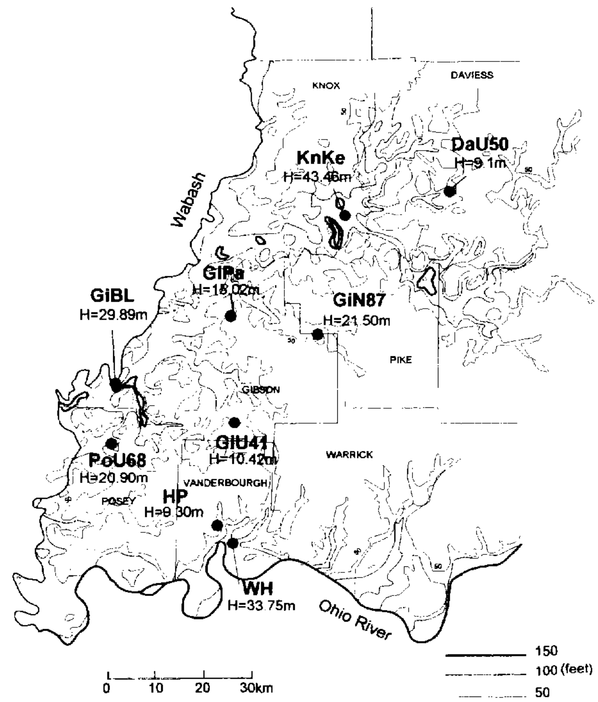

Fig.2. Location of selected sites and thickness of soil deposits.

Table 1: Bedrock and surface peak acceleration and amplification factor for each site, for the WVFS and NMSZ earthquakes. (liq. indicates liquefaction initiation according to Seed et al. (1985))

Site	Event from Wabash Valley fault system			Event from New Madrid seismic zone		
	Bedr. peak acc. (g)	Surf. peak acc. (g)	Amplif. factor	Bedr. peak acc. (g)	Surf. peak acc. (g)	Amplif. factor
DaU50	0.18	0.39 liq.	2.17	0.06	0.13 liq.	2.17
KnoxKe	0.20	0.32	1.60	0.06	0.18	3.00
GiPa	0.25	0.49	1.96	0.07	0.15	2.14
GiN87	0.20	0.25	1.25	0.07	0.17	2.43
GiBL	0.31	0.39	1.26	0.08	0.20	2.50
GiU41	0.20	0.37	1.85	0.08	0.15	1.88
PoUS68	0.30	0.49 liq.	1.63	0.08	0.16	2.00
EvanHP	0.20	0.44	2.20	0.09	0.20	2.22
EvanWH	0.19	0.16 liq.	0.84	0.09	0.08 liq.	0.89

SHAKE analysis has been performed at each site to obtain the soil response at each location. Table 1 shows the results from the SHAKE analyses. Peak ground horizontal acceleration can be up to 0.49g and 0.20g for a Wabash Valley Fault System earthquake and a New Madrid Seismic Zone earthquake, respectively. Generally, the amplification factor is greater for a NMSZ event since the effects of non-linearity and damping are less significant for relatively low amplitude ground motion. From the profile of peak acceleration with depth, the potential of liquefaction initiation is assessed from Seed et al. (1985). As an example, the acceleration profile and the corresponding cyclic stress ratios, critical and computed, for the site DaU50 and for the Wabash Valley Fault System earthquake scenario are presented in Fig. 3. The cases where computations yielded a factor of safety against liquefaction less than 1.0 (i.e. actual cyclic stress ratio larger than the critical stress ratio) are also noted in Table 1. These sites correspond to soil profiles including loose to medium dense sands.

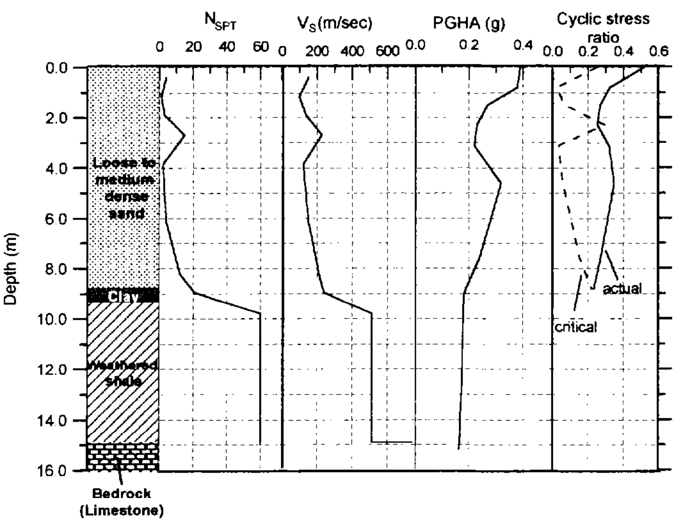

Fig. 3. Soil profile, resulting peak horizontal accelerations and cyclic stress ratios (critical and actual) for the site DaU50 and for WVFS earthquake scenario.

EARTHQUAKE INDUCED DAMAGE TO PILES

Mizuno (1987) recorded cases of pile damage due to earthquakes observed in Japan between 1923 and 1983. Numerous additional cases of pile foundations that suffered damage during major seismic events can be found in the literature, especially after the Hyogoken-Nambu, 1995, earthquake.

According to Matsui and Kazuhiro (1996) and Okahara et al. (1996), pile damage can be categorized with respect to severity as follows:

a) Severe: Dense cracking all over the pile, concrete separation, buckling of rebars, discontinuity of pile shaft; these types of failure are usually accompanied by residual horizontal displacement or settlement of the superstructure.

b) Heavy: Dense cracking and concrete separation near the pile head and several bending cracks at other locations at depth.

c) Light: Some bending cracks near the pile head and possibly at other locations.

d) No damage: Almost no cracking.

Testing of damaged foundations in Kobe showed that lightly damaged piles maintain sufficient vertical and lateral load capacity. If the cracking becomes denser and shear failure appears near the pile head, the lateral stiffness of the pile is reduced significantly, but it can still maintain adequate vertical capacity. If the pile is crushed, then the pile looses both vertical and horizontal capacity.

Our review and interpretation shows that, generally, pile failures take place first near the pile head, where bending moments and shear forces are maximum. However many cases show that large cracks may occur at pile locations near an interface between layers with large differences in stiffness, as well as between liquefied and non-liquefied layers (Fig. 4). This has been extensively observed in Kobe, where structures were founded on liquefiable reclaimed land (Matsui and Kazuhiro, 1996; Tokimatsu et al., 1996; Fujii et al., 1998; Nakayama et al., 1998) and after the Niigata, 1964 earthquake (Tazoh et al., 1987). Other locations where damage may develop are sections where the density of steel reinforcement is reduced or the location of the second largest moment (Matsui and Kazuhiro, 1996), especially after redistribution of loads following pile head failure.

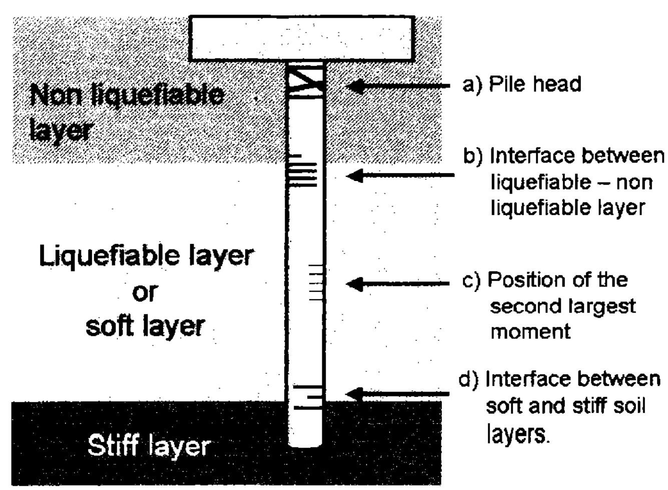

Fig. 4. Pile cross-sections most likely to sustain earthquake induced damage.

Reinforced concrete piles are highly susceptible to damage. However, it is often observed that concrete piles with steel casing (SC) have significantly improved performance even if the steel casing covers the pile only down to a certain depth, as reported by Tokimatsu et al. (1998) and Fujii et al. (1998). Steel pipe piles behave well even in cases of liquefaction-induced lateral spreading because of their ductility. Few cases of local buckling are reported (Mizuno, 1987; Tazoh et al., 1987). Finally, structures supported on friction piles in liquefiable layers may be subjected to tilting and significant settlement due to bearing capacity failure caused by soil liquefaction (Tokimatsu et al., 1996).

Figure 5 shows cases of earthquake induced damage to piles. The cases are sorted by the main cause of damage: ground shaking, liquefaction without significant lateral spreading, liquefaction with lateral spreading, and large inertial loads. Liquefaction can produce pile failure due to the degradation of soil stiffness and loss of lateral support. Lateral spreading is a factor that imposes additional loading to the pile and may produce heavier damage. Large axial and horizontal inertial loads coming from a tall superstructure can produce severe damage by crushing of the pile head (Mizuno, 1987; Tokimatsu et al., 1996; Kishida et al., 1980). Heavy damage in large diameter piles occurs mostly due to liquefaction. For piles with 0.5 m diameter or smaller, heavy or severe damage is caused by liquefaction with or without lateral spreading. However, if steel casing is used, the damage is light or there is no damage; in Figure 5, cases of steel piles and SC piles are circled to pinpoint their improved performance compared to that of reinforced concrete piles.

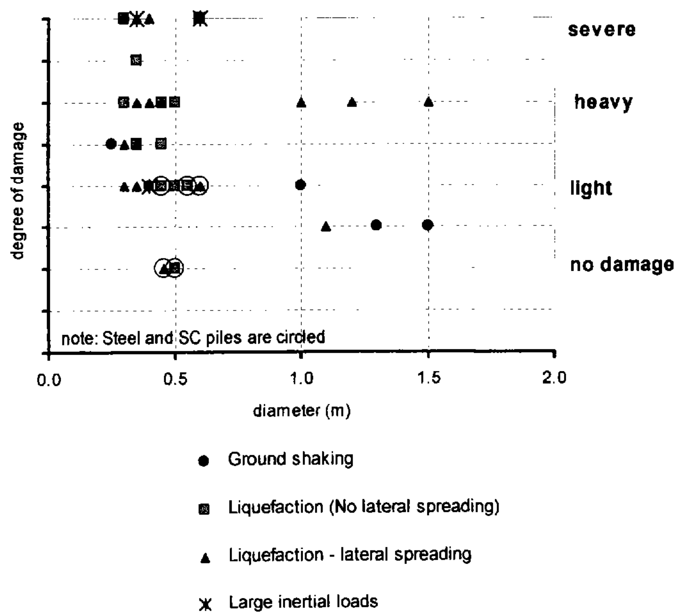

Fig. 5. Degree of damage vs. pile diameter sorted by cause of damage.

NUMERICAL ANALYSIS

Numerical simulations of pile subjected to seismic loading is performed to assess the damage susceptibility for typical soil profile condition in southwestern Indiana. A three-dimensional finite element model was set-up with ABAQUS to analyze a single pile subjected to ground shaking and inertial loads from the superstructure. The model, which is shown in Fig. 6, consists of a total number of 3040 second-order elements forming the soil and the embedded pile. At the lateral boundary, infinite elements are attached to the main model to allow outwards energy transmission during the dynamic analysis. A frictional interface is defined between pile and soil. The superstructure is modeled as a single degree of freedom oscillator (beam elements and a point mass) connected to the pile head through a layer of rigid elements. This allows for horizontal loads and moments due to the response of the superstructure to be transferred to the pile. Both soil and pile are modeled as elastic.

At the sites where bridge structures are located, the foundations consist of steel H piles and steel encased concrete piles (SEC). To examine the performance of the SEC pile under the soil and design conditions of the region, the site GiBL, next to the Wabash River (Fig. 2) is chosen as an example that may represent the performance and behavior of SEC piles. The structure is a three span road bridge. The piles have a diameter of 356mm (14in) with 25mm thick (1in) steel shell. The steel casing extends throughout pile. The piles in this bridge were not driven down to bedrock since they reached refusal before that. The unfactored design load of each pile was estimated as 356kN.

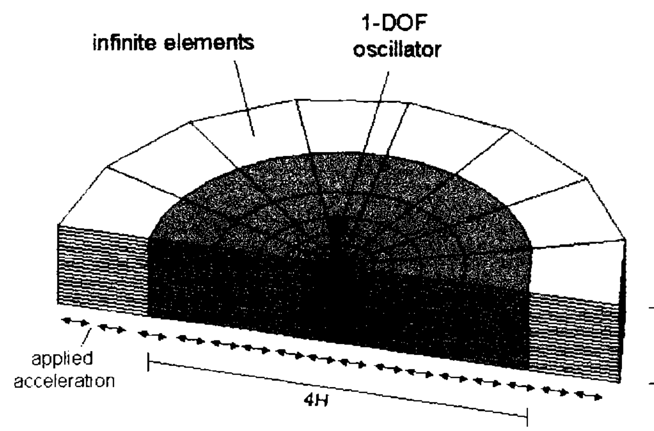

Fig. 6. Mesh used in finite element method analyses.

The analyzed pile has a length of 10m and belongs to a single row of piles supporting the abutment. The thickness of the mesh for this analysis is H=13.37m. A constant skin friction angle $\delta = 16.7^{\circ}$ ($\tan\delta=0.3$) along the entire pile length is considered, The value of the superstructure mass is set to the axial pile load design of 356kN. The values of the elastic modulus E are 21.5GPa for concrete and 200GPa for steel. The soil profile is composed of silt and silty clay with very low N_{SPT} values, underlain by dense sand and gravelly sand, which is the bearing stratum. The elastic modulus for each layer of elements is determined by reducing the initial modulus E_o to the equivalent linear secant modulus according to the results of the SHAKE analyses for the same site and earthquake scenario. The E_o values are estimated based on the Imai and Tonouchi (1982) relationships. Rayleigh factors consistent with the peak damping ratio from the response analysis are used. By making this consideration, despite the soil linearity assumed in the numerical model, the ground response is similar to that computed by SHAKE near the peak acceleration values.

Two runs are performed, one for the existing SEC piles and the other one for a reinforced concrete pile having the same diameter to observe the effect of the steel casing. The pile is analyzed for a Wabash Valley fault system earthquake scenario because it is the most critical. The results are summarized in Fig. 7. The bending moment is computed by integrating the axial stress at each layer of elements. For the SEC pile, the maximum bending moment, 207.7kNm, occurs at the pile head and corresponds to a maximum tensile stress in the concrete equal to $\sigma_{t,max}$=4.91MPa. Assuming a concrete tensile strength f_{ct}=2.4MPa (f_c=3000psi), the induced tensile stress is capable of initiating limited cracking in the concrete near the pile head.

In the concrete pile, the skin friction angle is set to be $\delta = 21.8^{o}$ (tanδ=0.4). The maximum bending moment is located at the pile head. It has a magnitude of 101.4kNm, and is smaller than the SEC pile (Fig. 7). However, the corresponding maximum tensile stress in the concrete, $\sigma_{t,max}$= 19.1MPa, is much larger than the assumed concrete tensile strength. The results show that a SEC pile would sustain less amount of damage than a reinforced concrete.

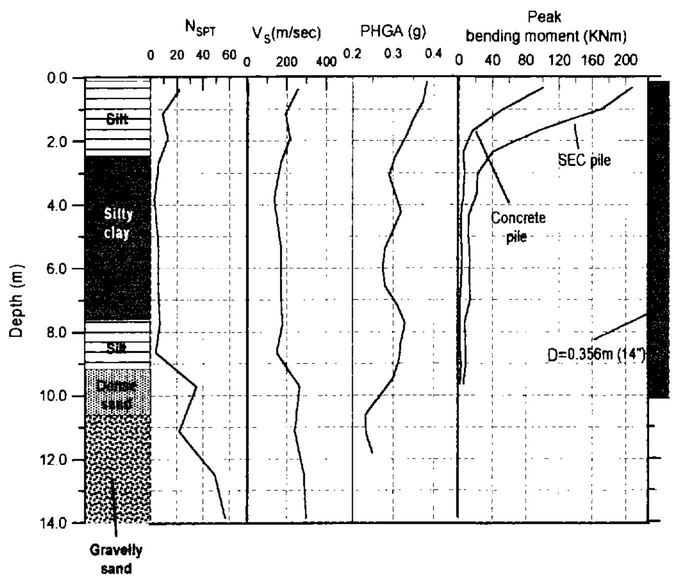

Fig. 7. Soil profile, peak horizontal accelerations and peak bending moment for 10m long concrete and steel casing concrete (SEC) piles at site GiBL for a WVFS earthquake scenario.

CONCLUSIONS

An earthquake from the Wabash Valley Fault System is the most critical for Southwestern Indiana, with peak surface accelerations that can be up to 0.5g. Soil liquefaction is possible at certain sites even in the case of an earthquake originating in the more distant New Madrid Seismic Zone. This is consistent with evidence of possible liquefaction as a result of ancient earthquakes of similar magnitude. Given that pile foundations are susceptible to severe damage in cases of liquefaction and lateral spreading, the effects of these phenomena should be investigated for bridge foundations relying on piling. Ground shaking may cause pile damage also in the absence of liquefaction because of the large inertial loads applied to the pile head by the superstructure. Additional damage can occur at the interface between soil layers with large difference in stiffness. A limited number of simulations show that damage starts at the pile head and may propagates down the pile.

The practice of using steel H piles and steel encased concrete piles in bridge foundations reduces the potential of heavy damage due to major seismic events. Although our analysis shows that larger moments may be developed in a SEC pile than in a concrete pile, the stresses in the concrete are smaller. Steel casing concrete piles seem to be a efficient method for earthquake resistant pile foundations, but additional detailed work is required on this subject.

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