THE ROLE OF SOIL ON THE COLLAPSE OF 18 PIERS OF THE HANSHIN EXPRESSWAY IN THE KOBE EARTHQUAKE

George MYLONAKIS[1], George GAZETAS[2], Sissy NIKOLAOU[3] And Odysseus MICHAELIDES[4]

SUMMARY

The paper investigates some of the causes of the collapse of a 630m segment of the elevated Hanshin Expressway in the Kobe earthquake. The bridge consisted of 18 single circular columns monolithically connected to the deck and founded on groups of 17 piles. There were 18 spans in total, all of which failed. Several factors associated with poor structural design have already been identified by earlier investigators. The scope of this paper is to complement the existing studies by examining the role of soil in the failure. Specifically, the following issues are discussed: (1) seismological and geotechnical information pertaining to the ground motion and the soil properties at the site; (2) free-field soil response; (3) response of the soil-foundation-superstructure system. Analytical results show that the soil role in the collapse could have been double: *First,* it modified the seismic waves so that the frequency of the surface motion at the site became disadvantageous for the particular structure. *Second,* the compliance of soil and foundation increased the period of the system and moved it to a region of stronger response. The associated increase in ductility demand on the piers may have exceeded 100% as compared to piers fixed at the base. The presented results are in contradiction with prevailed perceptions of an always beneficial role of seismic soil-structure interaction.

INTRODUCTION

In the devastation caused by the Kobe earthquake, the collapse and overturning of the 630m Fukae section of the Hanshin Expressway was perhaps the most spectacular failure. The bridge was part of the elevated Hanshin Expressway Route No 3 that runs parallel to the shoreline. Built in 1969, it consisted of single circular columns 3.1 meters in diameter and about 11 meters in height, founded on groups of 17 piles. The columns were connected monolithically to a concrete deck. A cross section of the bridge is shown in Fig 2. There were 18 spans in total all of which failed. This and other failures of bridges in recent earthquakes in California and Japan have raised serious concerns about our current understanding of the seismic behavior of pile supported structures on soft soil.

Detailed structural investigations of the behavior of Higashi-Nada bridge have been presented by, among others, Park (1996), Kawashima & Unjoh (1997) and Michaelides (1998). In those studies several factors contributing to the collapse associated with poor structural design were identified. These factors include: (i) inadequate transverse reinforcement in the piers; (ii) inadequate anchorage of longitudinal reinforcement; (iii) use of elastic methods (instead of capacity design procedures) for determining the design shear forces of the piers. Notwithstanding the importance of these parameters, there is evidence that local soil conditions and dynamic interaction between the foundation and the superstructure further contributed to the collapse.

Additional concerns come from the fact that Soil-Structure Interaction (SSI) has been traditionally considered beneficial for seismic response. Apparently this perception stems from oversimplifications in the nature of seismic demand adopted in code provisions. The most important of these simplifications with reference to SSI are (Mylonakis & Gazetas 1999): (1) design spectra with ordinates that decrease monotonically with increasing structural period; (2) response modification coefficients (used for deriving seismic design forces) which are

[1] *City University of New York, 195 Steinman Hall, Convent Ave at 138th Street NY 10031, USA*
[2] *National Technical University of Athens, Greece*
[3] *Mueser Rutledge Consulting Engineers, New York, USA*
[4] *Geotechnical Engineer, Nicosia, Cyprus*

either period independent or increase with increasing structural period; (3) foundation impedances derived assuming homogeneous halfspace conditions for the soil which tend to overpredict the damping of structures on actual soil profiles. The “beneficial” role of SSI has been essentially turned into a dogma. Thus, practicing engineers frequently avoid the complication of accounting for SSI effects --- a “conservative” simplification that would supposedly lead to improved safety margins. Results from the analysis of Higashi-Nada bridge are in contradiction with this perception. In fact, damage in structures associated with SSI effects has been proven or suspected in many cases in the past. For instance, the Mexico City earthquake of 1985 was particularly destructive to 10 to 12-story buildings (founded on soft clay) whose period increased from about 1.0 sec (for the fixed-base structure) to nearly 2.0 seconds due to SSI (Resendiz & Roesset 1985). Other evidence for a detrimental role of SSI has been presented by Meymand (1998), Gazetas & Mylonakis (1998), and Celebi (1998).

The work reported in this paper involves: (1) evaluation of pertinent seismological and geotechnical information on the ground motion and the soil properties at the site; (2) analysis of the free-field soil response; (3) analyses of the response of the foundation-superstructure system. Both linear and non-linear models are considered to this end; (4) evaluation of results through comparison with earlier studies that did not consider SSI effects.

The First Role of Soil: Influence on the Pattern and Intensity of Ground Motion

Kobe is built in the form of an elongated rectangle with length of about 30 km and width 2-3 km along the shoreline. The soil in the region consists primarily of sand with gravel of variable thickness (10-80 m), underlain by soft rock. The granitic bedrock that outcrops in the mountain region to the north of the city dips steeply in the northwest-southwest direction; in the shoreline it lies at a depth of about 1 to 1.5 km. Fig 1 shows an approximate geological plan and a cross section of the region as well as the locations of nearby strong motion accelerometers. Different soil thickness from one recording station to another may be responsible for the significant differences in the intensity and frequency content of the recorded motions (Soils & Foundations 1996).

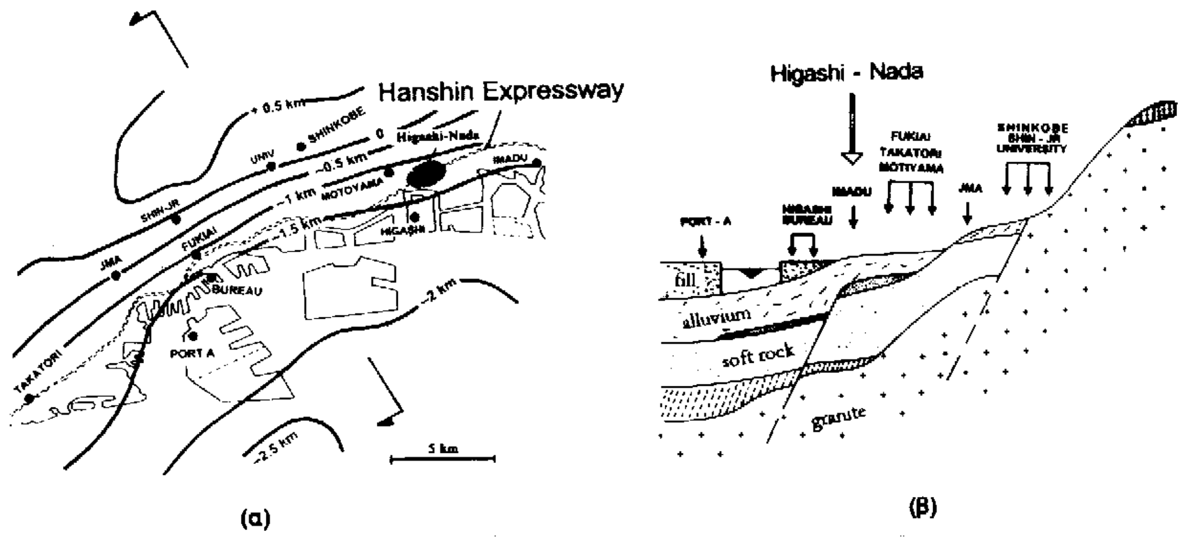

Figure 1. (a) Contours of bedrock elevation and location of accelerometers; (b) Approximate geological section of (a).

The differences in the thickness of the soil at various locations were among the reasons for the difference in recorded spectra, as shown in Fig 3. Of course, seismic “directivity” (Somerville 1998) was one of the phenomena that took place in Kobe: it undoubtedly led to the large differences between spectral values in directions normal and parallel to the fault rupture zone as well as to pronounced vertical motions (not shown herein) and large spectral values at periods around 1 second in rock. Notwithstanding the importance of these effects, it is believed that soil further amplified the incoming seismic waves and produced variations in the characteristics of the records, depending on the differences in the local soil conditions from site to site.

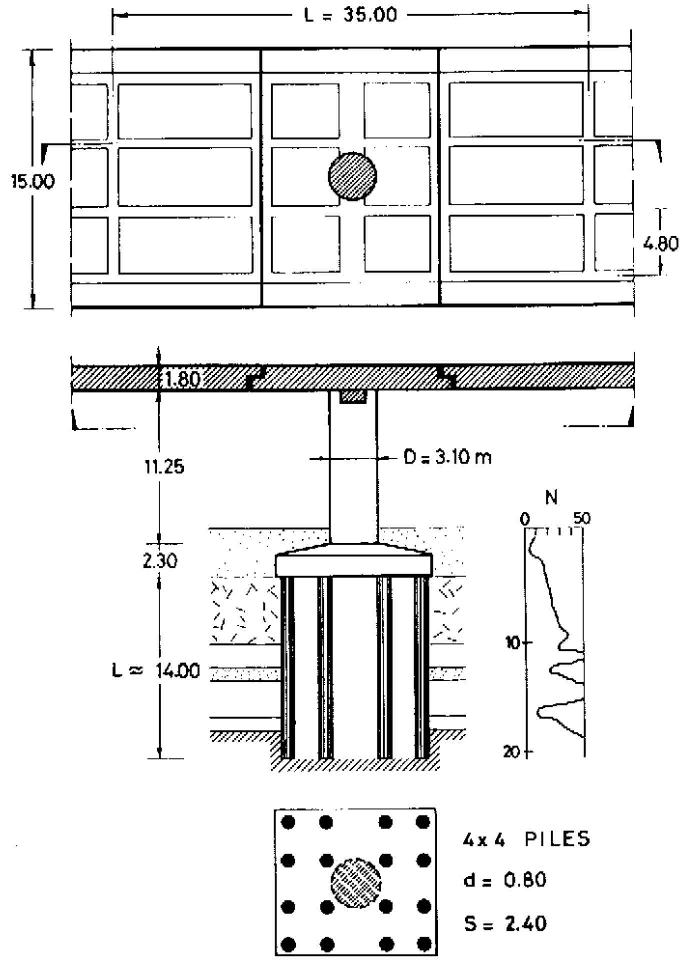

Figure 2. Geometric characteristics of the Higashi-Nada bridge.

The Second Role of Soil: Soil-Pile-Superstructure Interaction

The bridge consisted of 19 single circular columns, 3.1 meters in diameter and about 11 meters in height, monolithically connected to the deck and founded on groups of 17 reinforced concrete piles. The piles have length of about 15 m and diameter of 1 m, connected through a rigid 11x11 m cap. The soil surrounding the piles consists of medium dense sand with gravel. SPT values for the upper 20 meters of the soil are given in Fig 1. Corresponding shear wave velocities were found to be between 200 to 300 m/s down to 30 m depth. The structural parameters used in the following paragraphs have been taken from Park (1996), Kawashima & Unjoh (1997) and Michaelides (1997).

The mass of the bridge during the earthquake was found to be about 1,100 Mg (this includes some trucks which were on the bridge at the time of the earthquake), while the rotational moment of inertia of the deck with respect to the longitudinal axis was about 40,000 Mg m^2. The horizontal stiffness of the pier (uncracked conditions) was estimated to be of the order of 150 MN/m. In addition, it was found that about 0.7g horizontal acceleration at the bridge deck was needed to cause the column to reach its probable yield strength, and that the available displacement ductility capacity of the column was of the order of 2. From the above data, assuming the pier to be perfectly fixed at the base and considering the rotational inertia of the deck, the fundamental low-strain period of the bridge is estimated to be (Michaelides 1997)

$$T_s \approx 0.65\,sec \tag{1}$$

Note that, if cracked conditions had been assumed for the pier, the fixed-base period would increase to about 0.75 sec (Kawashima & Unjoh 1997). The dynamic interaction between soil, foundation and superstructure increases substantially the natural period of the bridge. An estimate of the period of the system can be obtained from the approximate formula of NEHRP-97 once the compliance of the pile group has been determined. More appropriately, Mylonakis et al (1997) and Gerolymos (1997) have studied the system shown in Fig 1 and found that the actual period of the bridge including SSI, was

$$\tilde{T} \approx 0.93\, sec \tag{2}$$

This preliminary result highlights the possible role of SSI, as it increases the "effective" period of the bridge by an appreciable 30%. The result of Eqn (3) was verified with more comprehensive analyses using the computer code SPIAB (Mylonakis et al 1997).

The uncertainty on the exact characteristics of the soil profile at the location of bridge (the nearest complete soil profile available to the authors was about 300 m away from the end of the bridge), dictated the use of different scenaria regarding the seismic excitation at the bridge site. Three acceleration records (Fig 3) with peak ground acceleration ranging between 0.65 and 0.83 g and quite different frequency characteristics were used in the analyses:

- The accelerogram *JMA*, with a peak value of 0.83g, was recorded on a relatively stiff soil formation (thickness of soft soil about 10-15 m)
- The accelerogram *Fukiai*, with a peak value of about 0.80 g, was recorded on a softer and deeper deposit (thickness of alluvium about 70 m)
- The accelerogram *Takatori*, with a peak value of 0.65 g, was recorded on a soft and deep deposit (thickness of alluvium about 80 m)

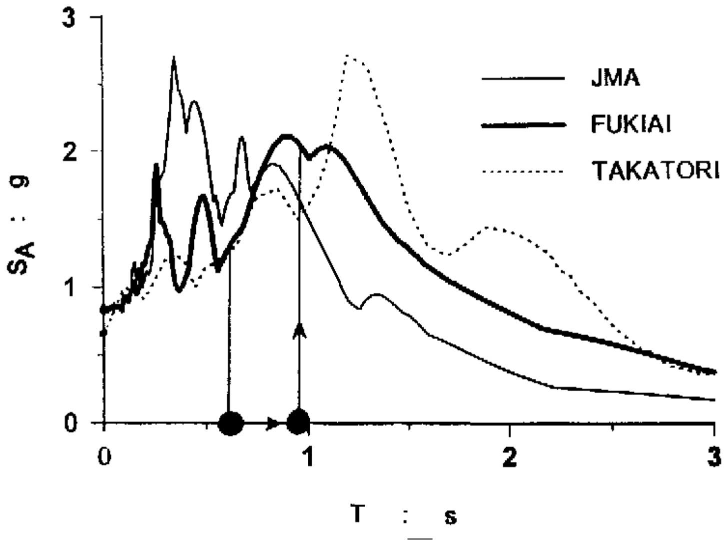

Figure 3. Acceleration response spectra of main records during the Kobe earthquake (ζ = 5%).

It should be noted that, as suggested by the geology of Fig 3, the soil conditions at the bridge location seem to be closer to those of Fukiai and Takatori, rather than to JMA.

From the spectra of Fig 3, the effect of SSI on the response of the bridge start becoming apparent. If the actual excitation was similar to the JMA record, the increase in period due to SSI and the progressive cracking of the

pier would tend to slightly reduce the response, as indicated by the decreasing trend ("de-resonance") of the spectrum beyond about 0.8 sec. In contrast, with either Fukiai or Takatori as excitation, SSI would lead to progressively larger accelerations in excess of 1g. As a first approximation, for a best estimate of SA = 1.4 g, the force reduction factor based on a calculated strength of the column of about 0.7g would be equal to approximately 2. Taking the equal displacement rule as approximately valid, the ductility demand on the pier would be:

$$\mu_d \approx 2 \tag{3}$$

which is probably higher than the corresponding ductility capacity due to the inadequate transverse reinforcement of the pier (Park 1996; Michaelides 1997).

Detailed analyses were also performed to verify the results of the above simplified analysis. They include: (i) equivalent-linear SSI analyses (using the computer code SPIAB) and (ii) non-linear dynamic analyses in which the foundation stiffness had been computed independently. A typical set of results of the SPIAB analyses (from Michaelides 1998) is shown in Fig 4 using the Fukiai record as excitation. The acceleration histories predicted for the deck with and without SSI exhibit different peaks and different frequency characteristics. Indeed the complete response (with SSI) is 25% higher than the response assuming fixed base --- a perhaps crucial difference contributing to the failure of the bridge.

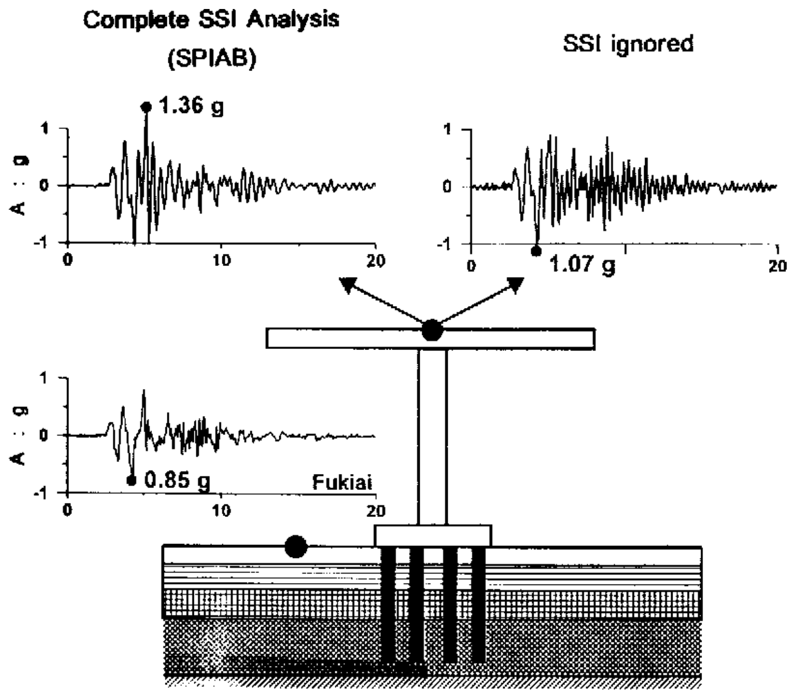

Figure 4. Effect of dynamic SSI on the acceleration response of the failed Route 3 Section of the Hanshin Expressway.

Non-linear Inelastic Analyses

To gain further insight on the importance of SSI on the performance of the system, a series of non-linear inelastic analyses were performed. To this end, a 2-degree-of-freedom inelastic model of the bridge was developed, similar to that used by Ciampoli & Pinto (1995). In the present study, the compliance of the foundation was modeled using a series of linear springs and dashpots attached at the base of the pier. A yielding strength of 7,500 kN was considered for the pier corresponding to a yielding deck acceleration of about 0.7g. A post yielding stiffness equal to 10% of the elastic stiffness of the pier was also assumed. The mass of the pile cap plus ½ the mass of the pier were considered lumped at the base of the pier (m_b = 541 Mg). 5% and 15% damping under elastic conditions were assumed for the superstructure and the foundation respectively. All three earthquake records (JMA, Fukiai, Takatori) were used in the analyses.

The distress of the system was obtained using two different indices: (i) the force reduction factor R (=elastic force demand / yielding strength) ; (ii) the ductility demand of the pier, μ_c ; (iii) the so-called “system ductility” μ_s. For an elastic-fully plastic system, μ_s is related to μ_c as (Priestley & Park 1987):

$$\mu_s = \frac{c + \mu_c}{c + 1} \tag{4}$$

where c denotes the ratio of the stiffness of the foundation to the stiffness of the pier.

The role of SSI in the performance of the bridge is visible in Table 1. First, using the JMA record the ductility demand in the column decreases when SSI effects are considered (i.e., from 2.86 for the fixed-base structure to 2.58 for the flexibly-supported). In contrast, with Fukiai and Takatori records SSI is detrimental, increasing the ductility demand in the pier. This is in (qualitative) agreement with the trends observed in Fig 3. In particular, Fukiai excitation leads to a substantial increase in μ_c : from 1.99 to 4.24 without and with SSI, respectively. The latter value is much higher than the estimated displacement ductility capacity of the pier ($\mu_{capacity} \approx 2$). Such an excessive amount of seismic demand may explain the spectacular failure of all 17 piers of the bridge. Accordingly, this could indicate that the actual excitation at the site resembled more the Fukiai rather than the JMA or Takatori excitations. It is also worth mentioning that the "system ductility" ms is always smaller than mc and provides a misleading index for assessing the bridge performance (it does not reflect the actual distress in the pier). Nevertheless, μ_s appear to match better the values of R (for the relatively long period system examined herein), so μ_s may be more appropriate for developing R-μ relationships.

Table 1. Tabulated results from inelastic analyses of the bridge response.

Excitation	Analysis type	Effective natural period (s)	Peak deck acceleration (g)	Peak deck displacement (cm)	Peak drift displacement (cm)	R	System ductility μ_s	Column ductility μ_c	Role of SSI
JMA	fixed base	0.65	0.87	21.0	21.0	2.67	2.86	2.86	*beneficial*
	flexible base	0.93	0.89	21.0	19.0	2.13	1.93	2.58	
Fukiai	fixed base	0.65	0.80	14.6	14.6	1.95	1.99	1.99	*detrimental*
	flexible base	0.93	1.02	41.9	31.2	2.56	2.79	4.24	
Takatori	fixed base	0.65	0.74	10.0	10.0	1.36	1.34	1.34	*detrimental*
	flexible base	0.93	0.79	24.4	13.4	1.67	1.62	1.83	

CONCLUSIONS

The role of soil in the collapse was double and detrimental: *First*, it modified the incoming seismic waves such that the resulting motion at the surface become detrimental for the bridge at hand (amplification of spectral accelerations in the range T ≈ 0.8 to 1.3 secs). *Second*, the presence of compliant soil at the foundation resulted to an increased natural period of the bridge which moved to a region of stronger response. In the case of Fukiai record, the increase in seismic demand due to SSI was dramatic. Of course, both phenomena might simply worsen an already dramatic situation for the bridge due to its proximity to the fault and inadequate structural design.

ACKNOWLEDGMENTS

Seismological and geotechincal data for the bridge site were provided to the authors by Dr. Takashi Tazoh, General Manager, Civil Engineering Department, Shimizu Corporation. His help is gratefully acknowledged.

REFERENCES

Celebi, M. (1998). “Turkish Earthquakes: Two Reports. Lessons from the Adana-Ceyhan Quake and the Dinar Aftershock”, *EERI Newsletter*, Vol. 32, No. 9, 8 pages

Ciampoli, M. and Pinto, P.E. (1995). “Effects of Soil-Structure Interaction on Inelastic Seismic Response of Bridge Piers”, *Journal of Structural Engineering*, ASCE, Vol. 121, No 5, pp. 806-814

Gazetas, G. and Mylonakis, G. (1998). “Seismic Soil-Structure Interaction: New Evidence and Emerging Issues” State of the Art Paper, *Geotechnical Earthquake Engineering and Soil Dynamics Geo-Institute ASCE Conference*, Seattle, August 3-6, 1998, Vol. II, pp. 1116-1174

Gerolymos, N, Gazetas, G., and Mylonakis, G. (1998). “Fundamental Period and Effective Damping of Pile-Supported Bridge Piers”, *11th European Conference on Earthquake Engineering*, Topic 3, Paris, September 1998

Ishihara, K. (1997). “Geotechnical Aspects of the 1995 Kobe Earthquake”, Terzaghi Oration, *14th ICSMFE*, Hamburg

Kawashima, K. and Unjoh, S. (1997). “The Damage of Highway Bridges in the 1995 Hyogo-Ken Nambu Earthquake and its Impact on Japanese Seismic Design”, *Journal of Earthquake Engineering*, Vol. 1, No 3, pp. 505-541

Meymand, P.J. (1998). “Shaking Table Scale Model Tests of Nonlinear Soil-Pile-Superstructure Interaction in Soft Clay”, Ph.D. Dissertation, *University of California*, Berkeley

Michaelides (1997). “Non-linear dynamic analysis of piles”, Ph.D. Dissertation, *National Technical University of Athens*, Greece (in Greek).

Mylonakis, G. and Gazetas, G. (1999). “Seismic Soil-Structure Interaction: Is it beneficial or detrimental?”, Journal of Earthquake Engineering, (under review).

Mylonakis, G., Nikolaou, A., and Gazetas, G. (1997). “Soil-Pile-Bridge Seismic Interaction: Kinematic and Inertial Effects. Part I : Soft Soil”, *Earthquake Engineering. & Structural Dynamics*, Vol. 26, pp. 337-359

Park, R. (1996). “An Analysis of the Failure of the 630 m Elevated Expressway in Great Hanshin Earthquake”, *Bulletin of the New Zealand National Society for Earthquake Engineering* , Vol. 29, No. 2

Pender, M. (1993). “Aseismic Pile Foundation Design Analysis”, *Bulletin of the New Zealand National Society for Earthquake Engineering* , Vol. 26, No. 1, pp. 49-160

Priestley, M.J.N. and Park, R. (1987). “Strength and Ductility of Concrete Bridges Columns Under Seismic Loading”, *ACI Structural Journal*, Vol. 84, No 1, pp. 61-76

Resendiz, D. and Roesset, J.M. (1987). “Soil-Structure Interaction in Mexico City During the 1985 Earthquake”, The Mexico Earthquakes -1985, *ASCE special publication*

Soils and Foundations, Vol. 36, No. 1 (1998). “Special Issue on Geotechnical Effects of the Hyogo Ken Nanmbu Earthquake”

Stewart, J.P., Seed, R.B., and Fenves, G.L. (1999). “Seismic Soil-Structure Interaction in Buildings. II: Empirical Findings”, *Journal of Geotechnical Engineering., ASCE*, Vol. 125, No. 1, pp. 38-48