Estimation of Earthquake-Induced Pile Bending in a Thick Peat Deposit

STEVEN L. KRAMER, MATTHEW A. CRAIG, AND SUJAN PUNYAMURTHULA

An investigation of the nature of earthquake-induced deformation of piles supporting highway bridges that cross a thick peat deposit in Washington State is presented. The results of a field and laboratory testing program were used to predict free-field ground response for several soil profiles representative of subsurface soil conditions across the site. The computed free-field soil displacements were used in an approximate soil-pile interaction analysis to estimate the curvature demand on different types of piles. The interaction analysis was based on three-dimensional finite element analysis of a pile embedded in a column of soil. The results indicated that the curvature demands on the piles were much smaller than the free-field curvatures but were still quite large.

The seismic response of piles in soft soil deposits is a problem of considerable interest and importance to geotechnical engineers and owners of pile-supported structures such as highway bridges. The need for careful attention to potential problems with pile-supported bridges in soft soil areas has been illustrated in many previous earthquakes, perhaps most recently in the 1989 Loma Prieta earthquake. The large displacements and eventual failure of the Struve Slough bridge in Watsonville, California, during the Loma Prieta earthquake emphasized the need for evaluation of potential pile deformations in very soft peat deposits. This paper presents the results of an investigation of the potential levels of earthquake-induced bending of piles extending through a thick peat deposit in Washington State.

PILE BENDING DURING EARTHQUAKES

Earthquake-induced ground shaking causes vertical and horizontal displacements of soil at and below the ground surface. The nature of the deformations depends on the geometry and material properties of the various soil and underlying rock units at the site and on the characteristics of the seismic waves that reach the site. Ground displacement amplitudes are typically small for small or distant earthquakes or for very stiff or dense soil profiles; however, they may become quite large in soft soil deposits. Displacement amplitudes are generally largest at the ground surface and tend to decrease with depth.

In soft soil areas, significant transportation structures such as highway bridges are often supported on deep foundations. These deep foundations may be subjected to lateral loading at their heads from traffic, thermal expansion, and other sources; methods for evaluating their response to such loads are now well established (*1–4*). When deep foundation elements extend through soft soils in seismically active areas, however, they may be subjected to a different form of lateral loading-that which is imposed by earthquake-induced displacement of the surrounding soils. Since earthquake-induced soil displacements are not uniform with depth, the soil displacement profile will be curved, thereby inducing bending moments in the piles. Soil profile curvatures are generally greatest at boundaries between materials of different stiffness. Damage to pile foundations for highway bridges has been observed in a number of earthquakes (*5–7*).

Until relatively recently, designers commonly assumed that the mass and stiffness of piles were sufficiently small that piles would move with the surrounding soil during earthquakes (*8,9*). As a result, the curvature demand for piles was considered to be equal to the maximum computed free-field soil profile curvature. Margason and Holloway (*8*) used numerical analyses to predict the curvature profile for a soft soil site in the San Francisco Bay area. The soil curvature profile was far from constant, with large pile curvatures occurring over relatively short vertical distances. The largest computed curvature was located at the boundary between soft Bay mud and an underlying layer of stiff clay at which the small strain impedance ratio was approximately 1.6. This computed soil curvature, approximately 4.0×10^{-4} in.$^{-1}$ (1.6×10^{-4} cm^{-1}), corresponded to a radius of curvature of about 208 ft (63.4 m), which exceeds the elastic limit of many piles. Margason and Holloway (*8*) indicated that maximum pile curvatures of 2.0 to 4.0×10^{-4} in.$^{-1}$ (0.8 to 1.6×10^{-4} cm^{-1}) could be expected for magnitude (**M**) 7.0 earthquakes and 8.0×10^{-4} in.$^{-1}$ (3.2×10^{-4} cm^{-1}) could be expected for **M**8.0 earthquakes in the San Francisco Bay area.

Banerjee et al. (*10*) repeated the analysis of the San Francisco Bay area profile for which Margason and Holloway had obtained a maximum curvature of 4.0×10^{-4} in.$^{-1}$ (1.6×10^{-4} cm^{-1}). Banerjee et al., however, performed a two-dimensional, soil-structure interaction analysis with a 12-in. (30.5-cm) precast, prestressed concrete pile embedded in the soil profile and found that the maximum computed pile curvature was only 2.2×10^{-4} in.$^{-1}$ (0.9×10^{-4} cm^{-1}), or slightly more than half of the free-field soil profile curvature. These analyses showed that the flexural stiffness of the pile was sufficient to span short distances of high-soil-profile curvature with significantly lower pile curvature. Other methods for analysis of seismic pile-soil interaction have also been developed.

Department of Civil Engineering, University of Washington, Seattle, Wash. 98195.

SITE CONDITIONS

The Puget Sound area of Washington State is well known as a seismically active region. Strong ground motion has been induced by a number of historical earthquakes, most recently by the 1949 Olympia (M7.1) and 1965 Seattle-Tacoma (M6.5) earthquakes. These earthquakes are thought to have been caused by the release of strain energy accumulated by bending of the Juan de Fuca plate as it subducts beneath the North American plate; a maximum magnitude of 7.5 is generally accepted for this mechanism. Recent research, however, indicates that much larger subduction earthquakes occurred before the Pacific Northwest was settled (*11,12*). The possibility of subduction-zone earthquakes of magnitude up to 9.5 has been postulated for the Pacific Northwest.

Mercer Slough is a peat-filled extension of Lake Washington that covers several square miles in Bellevue, Washington (Figure 1). Lake Washington water levels are maintained at a nearly constant depth at the Hiram Chittenden Locks in Seattle; consequently, the groundwater level in Mercer Slough is generally within 1 ft of the ground surface. The thickness of the Mercer Slough peat is variable across the slough, with a maximum thickness of approximately 60 ft (18.3 m) along the alignment of Interstate 90, which crosses Mercer Slough by means of four pile-supported bridge structures. Although some of the deepest peat has been dated at more than 13,000 years old, most was deposited after Lake Washington was isolated from Puget Sound by the Cedar River alluvial fan some 7,000 years ago (*13*). Since that time, the peat appears to have accumulated at a average rate of approximately 0.07 in./year (0.18 cm/year). The peat is underlain by very soft to medium stiff silty clay and occasional loose to dense sand, which is in turn underlain by heavily overconsolidated, dense glacial till. Tertiary bedrock in the area is at a depth of about 1,000 ft (305 m) (*14*). A subsurface profile of Mercer Slough along the I-90 alignment is shown in Figure 2.

The existing bridges across Mercer Slough are supported on vertical piles driven to depths sufficient to resist vertical design loads of 20 to 70 tons (178 to 623 kN) each. Piling

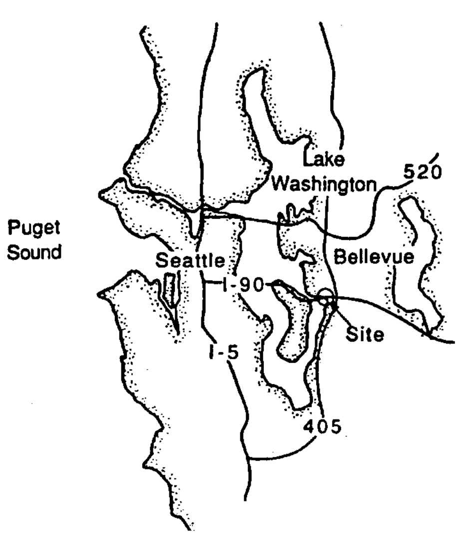

FIGURE 1 Location of Mercer Slough in Bellevue, Washington.

consists of timber piles and 14- and 18-in. (35- and 46-cm) concrete-filled steel pipe piles. A distant landslide along Lake Washington Boulevard caused lateral movement of the bridge and, presumably, the pile foundations supporting the bridge. A previous investigation associated with that landslide (*15,16*) indicated that the Mercer Slough peat offers very little resistance to lateral movement of piles under static conditions.

MERCER SLOUGH PEAT PROPERTIES

The unusual and problematic behavior of peats has been known for many years. Considerable progress has been made in understanding their long-term static behavior, thus allowing them to be dealt with rationally when they cannot be removed or avoided. Problems associated with the dynamic response of peat, however, have not been addressed.

Index and Static Properties

The Mercer Slough peat is fibrous at shallow depths and becomes less fibrous and more highly decomposed with increasing depth. In recent subsurface investigations, water contents were generally between approximately 500 percent and 1,200 percent, with no apparent trend with depth. Buoyant unit weights were generally less than 5 lb/in.^{3} (0.79 kN/m^3). The strength of the peat was very low (*16*); a cohesive strength of about 200 lb/ft^2 (9.6 kPa) appears to best represent its shearing resistance (*15*). A significant time-dependent response, in the form of creep and stress relaxation, has been observed in field and laboratory strength tests on Mercer Slough peat.

Dynamic Properties

Very little research has been performed on the dynamic properties of peat or on the dynamic response of peat deposits. The only such work identified in the literature was associated with the foundation investigation for a proposed (but never constructed) highway tunnel in the Union Bay area of Seattle, Washington (*17*). The Union Bay site, also located on Lake Washington but about 7 mi (11 km) northwest of Mercer Slough, consisted of up to 60 ft (18.3 m) of peat underlain by up to 80 ft (24.4 m) of soft to medium stiff clay. The clay was underlain by heavily overconsolidated, dense glacial till. In 1966, water contents in the Union Bay peat ranged from 700 to 1,500 percent at the surface, with a trend of slightly decreasing water content with increasing depth. Saturated unit weights ranged from 62.6 to 66.0 lb/in.^{3} (9.8 to 10.4 kN/m^3) and averaged 63.7 lb/ft^3 (10.0 kN/m^3). Atterberg limits tests showed liquid limits of 700 to 1,000 and plasticity indices of 200 to 600. The Union Bay and Mercer Slough peats were deposited under very similar conditions, and it is expected that their important geotechnical characteristics will also be similar. Seed and Idriss (*18*) interpreted the results of field and laboratory tests performed by Shannon & Wilson (*17*) to propose the dynamic properties shown in Figure 3. Shear moduli were obtained from shear wave velocity measurements and repeated loading tests. Damping ratios were estimated with the reasoning that "because of its fibrous and less co-

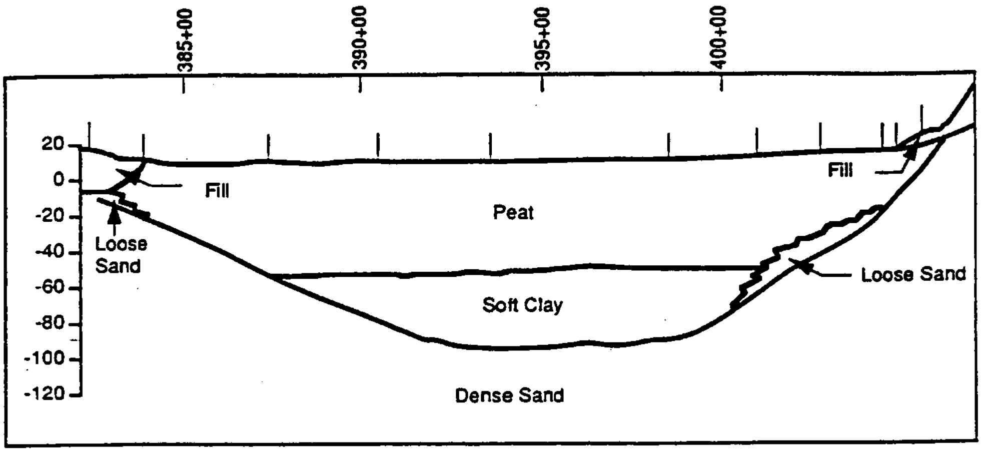

FIGURE 2 Subsurface profile of Mercer Slough along alignment of I-90 bridges (vertical scale exaggerated by factor of 10).

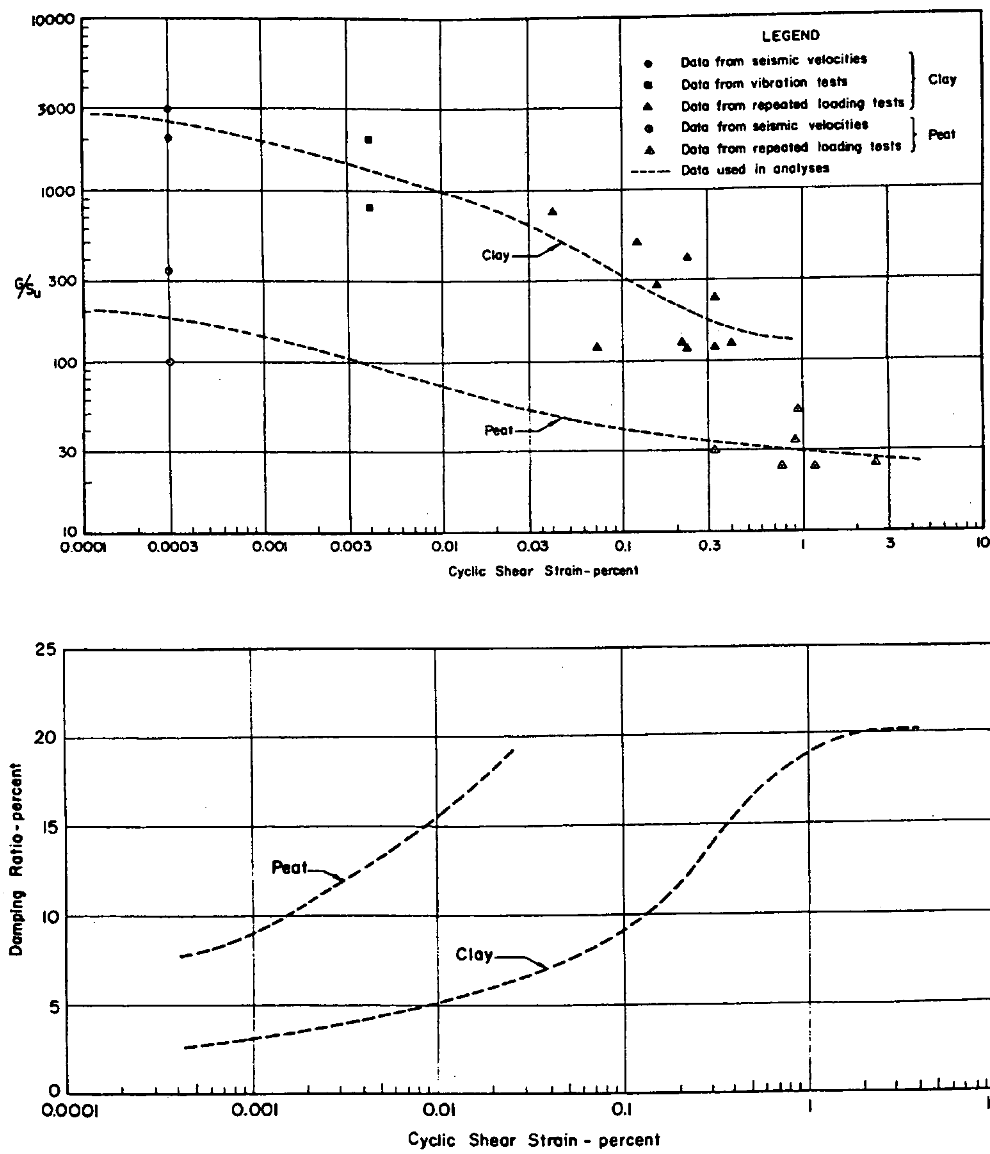

FIGURE 3 Peat properties assumed by Seed and Idriss (*18*).

hesive characteristics, damping for peat would be expected to be higher than for clay" (*18*). These damping ratios were approximately three times as large as those used at the time for clay.

A field and laboratory testing program was undertaken to evaluate the dynamic properties of the Mercer Slough peat. To estimate the low-strain stiffness of the peat, a seismic cone sounding was performed near the edge of a parking lot fill. The results of the sounding, shown in Figure 4, indicated that the peat had an average shear wave velocity of approximately 100 ft/sec (30 m/sec) despite having been consolidated to some extent by the overlying fill. An initial series of cyclic triaxial tests on undisturbed samples taken from the same location showed shear moduli greater than those inferred from the shear wave velocities. These results were considered unsatisfactory, and it was deemed necessary to obtain sample-specific low-strain stiffness data. The cyclic triaxial testing equipment was then modified by the addition of piezoelectric bender elements for subsequent testing. The resulting variation of normalized shear modulus with cyclic shear strain amplitude observed in a small number of cyclic triaxial-bender element tests on Mercer Slough peat is shown in Figure 5 (*top*); the shear modulus of the Mercer Slough peat samples degraded at a slower rate than that observed for inorganic clays (*19*). Damping ratio data, shown in Figure 5 (*bottom*), was typically scattered but showed a distinct and consistent trend of decreasing damping ratio with increasing strain amplitude above strain amplitudes of approximately 0.02 percent. The reason for this behavor is not yet understood; it may be related to stretching and straightening of the peat fibers at larger strain levels. Damping ratios could not be measured at lower strain amplitudes with the available testing equipment. Further research on the dynamic properties of the Mercer Slough peat and other peats is continuing.

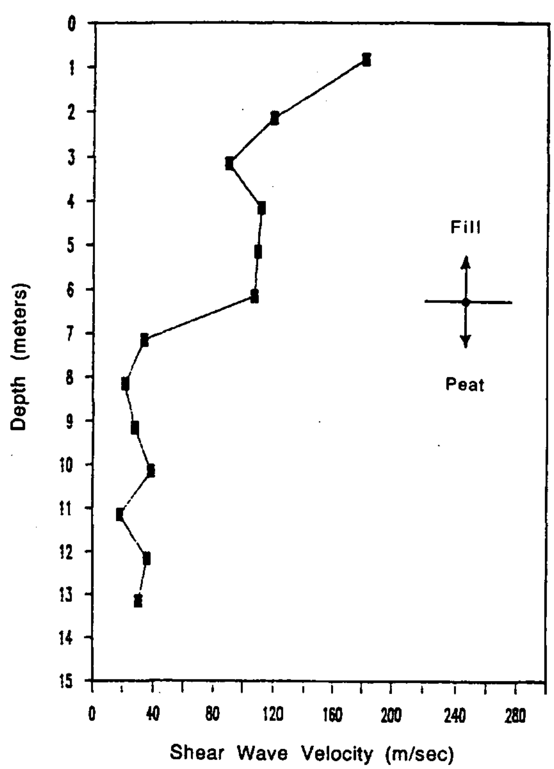

FIGURE 4 Results of seismic cone penetration test sounding.

GROUND RESPONSE ANALYSES

Very little information on the dynamic response of peat deposits is available in the literature. Seed and Idriss (*18,20*) analyzed recorded motions at Union Bay from a small (M4.5), distant (R = 49 km) earthquake using an equivalent linear, lumped mass technique. The motion measured in the glacial till, which had a peak acceleration of 0.03 g, was applied as a rigid base motion (I. M. Idriss, personal communication, 1991) at the clay-till interface, and reasonable agreement was obtained between the measured and predicted accelerations. Because the level of shaking was very low, however, the nonlinear behavior of the peat was not significant.

For a large, design-level ground motion at the Mercer Slough site, nonlinear behavior is expected to be significant. In order to investigate the effects of the measured dynamic peat properties on ground-shaking characteristics during such earthquakes, a series of ground response analyses was performed.

Soil Profiles

Soil conditions along the I-90 alignment were divided into the five ground-response zones shown in Figure 6. The typical soil profiles for the ground-response zones are given in Table 1. As indicated by the total thickness of each soil profile, the depth to bedrock was assumed to be 1,000 ft (305 m). Subsurface investigations indicated that the fill was loose and cohesionless with Standard Penetration Test resistances similar to that of the loose sand layer. The fill and loose sand layer were consequently assigned a K_{2max}-value of 51.8. The soft clay deposit beneath the Mercer Slough peat in the central portion of the site had an average plasticity index of approximately 20 and was modeled as such using the modulus and damping ratio curves of Vucetic and Dobry (*19*). The glacial till consisted primarily of dense sand and gravel and was modeled as a dense sand with K_{2max} = 80. Bedrock was assigned a shear wave velocity of 8,000 ft/sec.

Input Motions

The design ground motion was assumed, for consistency with other nearby Washington State Department of Transportation projects, to have a peak bedrock acceleration of 0.30 g and a predominant period of 0.36 sec. The response of each soil profile was computed using three separate bedrock motions, which were obtained by scaling the El Centro, Lake Hughes, and Castaic historical records to produce the characteristics of the design ground motion.

Computed Ground Motions

Ground response was computed using the well-known one-dimensional, equivalent linear, complex response method in-

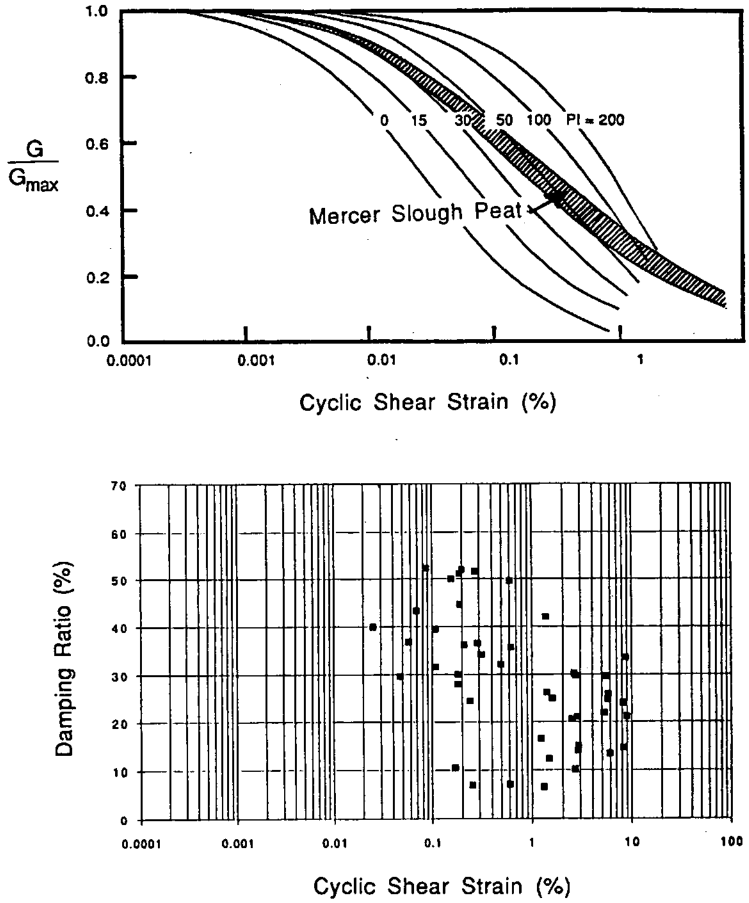

FIGURE 5 Measured dynamic properties of Mercer Slough peat: (*top*) normalized shear modulus and (*bottom*) damping ratio.

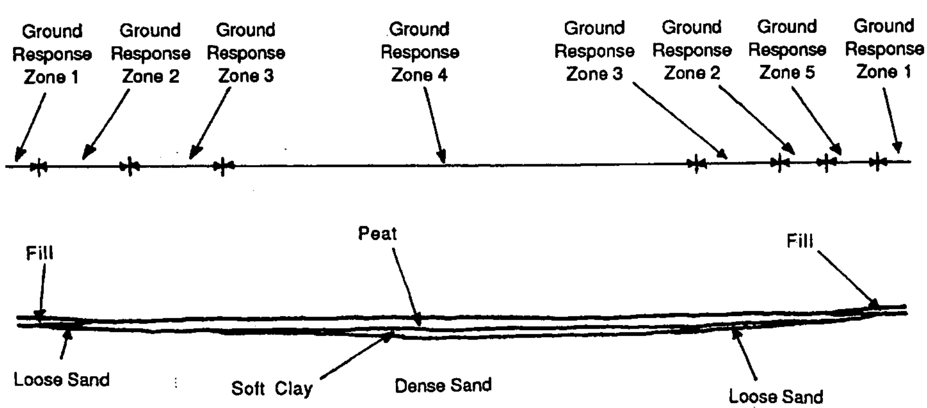

FIGURE 6 Location of ground-response zones (identical vertical and horizontal scales).

TABLE 1 Soil Profiles Used in Ground-Response Analyses

	Thickness of Layer in ft (m)				
Layer	Zone 1	Zone 2	Zone 3	Zone 4	Zone 5
Fill	23 (7.0)	0	0	0	8 (2.4)
Peat	0	30 (9.1)	44 (13.4)	60 (18.3)	11 (3.4)
Soft Clay	0	0	0	43 (13.1)	0
Loose Sand	0	0	11 (3.4)	0	0
Glacial Till	977 (298)	970 (296)	945 (288)	897 (273)	981 (299)

corporated in the computer program SHAKE (*21*). The input motions, applied at the bedrock-till interface, induced significant strains in the very soft Mercer Slough peat. The ground-response analyses predicted the following peak ground surface accelerations:

Zone	*Avg Peak Ground Surface Acceleration (g)*
1	0.16
2	0.12
3	0.12
4	0.08
5	0.12

The average computed peak accelerations for all ground-response zones were lower than the peak bedrock acceleration. For ground-response Zones 2 through 5, which contained varying thicknesses of Mercer Slough peat, the peak ground surface accelerations were considerably lower than the peak bedrock acceleration, illustrating the deamplifying effect of the very soft peat. The effects of the peat were also reflected in the frequency content of the computed ground surface motions. Examination of the time histories of ground motion further illustrates the influence of the thick peat layer on ground surface motions. Figure 7 shows time histories of acceleration, velocity, and displacement computed for ground-response Zone 3 in response to the scaled El Centro input motion. Though the computed peak ground surface acceleration is only 0.2 *g*, it occurs at a very low frequency and consequently produces very large displacements. The maximum computed relative displacements between the ground surface and the top of the glacial till are as follows (1 ft = 0.3 m):

Zone	*Maximum Relative Displacement (ft)*
1	0.01
2	0.95
3	4.04
4	3.73
5	0.58

These maximum relative displacements were generally produced when the displacements of the ground surface and the top of the glacial till were out of phase. As indicated by the tabulation above, the maximum relative displacement was

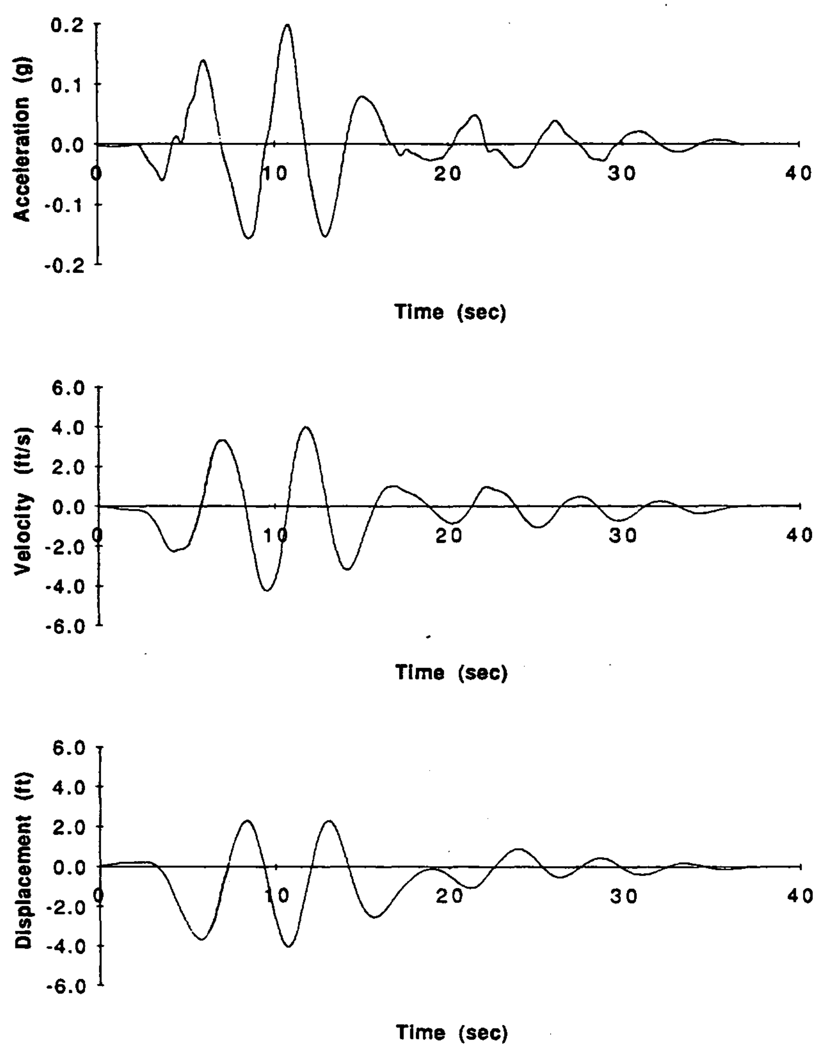

FIGURE 7 Computed time histories of acceleration, velocity, and displacement for ground-response Zone 3 (scaled El Centro input motion).

very small for ground-response Zone 1, which contained no peat, and very large for the other ground-response zones.

Average, normalized ground surface response spectra for each of the five ground-response zones are shown, along with the Applied Technology Council standard design response spectra (*9,22*), in Figure 8. Although the standard design spectra adequately describe the computed response for ground-response Zone 1, which contained no peat, they are not consistent with the computed response of the ground-response zones that did contain peat.

Free-field soil profile curvatures were computed for ground-response Zones 2 and 3. In both cases, the time at which the maximum soil profile curvature occurred did not coincide with the time at which the maximum relative displacement occurred. The soil curvature profiles at the time of maximum soil curvature are shown in Figure 9. In both cases, the peak computed curvature occurred at the bottom of the peat layer, where the computed impedance ratios were 16 and 11 for ground-response Zones 2 and 3, respectively. These impedance ratios were much larger than those from the San Francisco Bay area sites analyzed by Margason and Holloway (*8*) and Banerjee et al. (*10*). As a result, the maximum computed soil profile curvatures were also very large: 14.5×10^{-4} in.$^{-1}$ (5.7×10^{-4} cm^{-1}) for ground-response Zone 2 and 150×10^{-4} in.$^{-1}$ (59×10^{-4} cm^{-1}) for ground-response Zone 3. These soil curvatures correspond to radii of curvature of 57 ft (17.5 m) for ground-response Zone 2 and 5.5 ft (1.7 m) for ground-response Zone 3. These extraordinarily large computed curvatures resulted from the unusually low stiffness and density of the Mercer Slough peat.

SOIL-PILE INTERACTION

The existence of locally high curvatures suggests that the free-field soil displacement profile exhibits "kinks" at the depths corresponding to high curvature, as shown in Figure 9. The assumption that the concrete-filled steel pipe piles and timber piles supporting the I-90 bridges will move with the soil and assume the same kinked shape is clearly inappropriate, particularly in light of the very low stiffness and strength of the Mercer Slough peat.

Method of Analysis

To estimate the curvature demand on the piles, approximate soil-pile interaction analyses were performed using a three-dimensional finite element model of a column of soil containing a pile. The meshes used in the analyses are shown in Figure 10. To minimize potential boundary effects, all boundaries were located at least 10 pile diameters from the center of the pile. The analyses were performed in the following manner:

1. With the pile elements assigned the same properties as the surrounding soil at the same depth, the finite element mesh was deformed such that the lateral displacements of the nodal points along the centerline of the pile were equal to the computed free-field soil profile displacements at the time of maximum soil profile curvature (Figure 9).
2. The nodal point forces required to deform the mesh into this configuration were computed and stored.

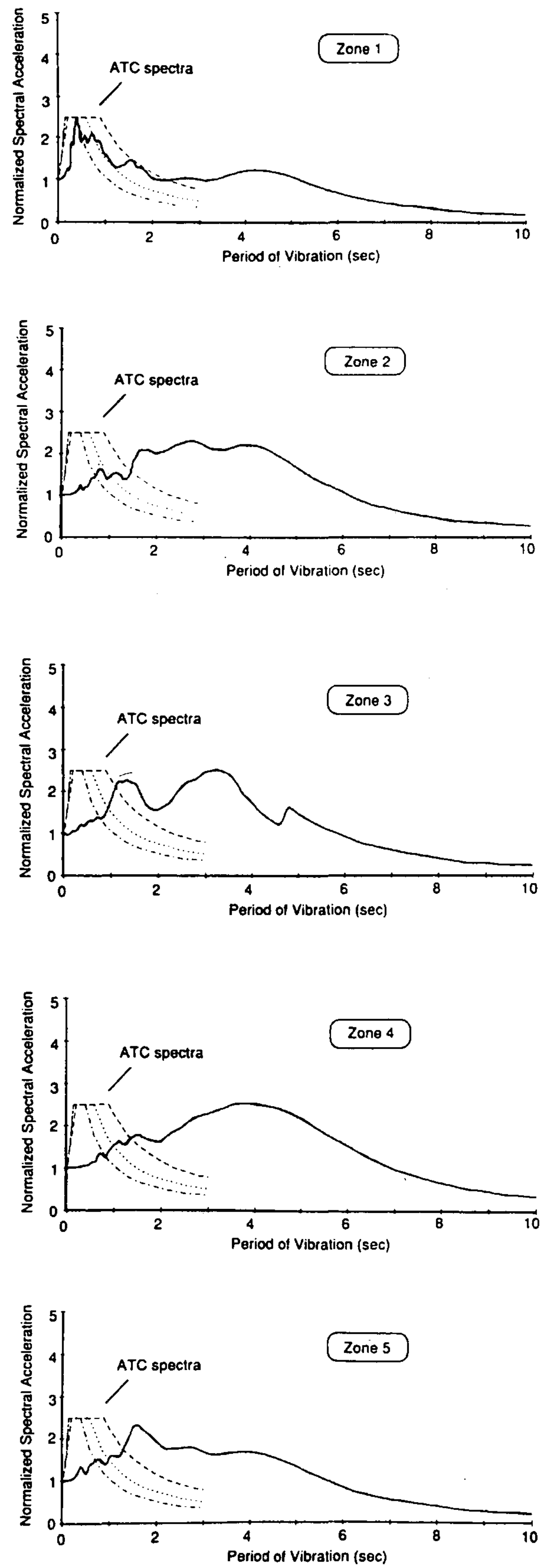

FIGURE 8 Computed normalized response spectra for ground-response Zones 1 through 5.

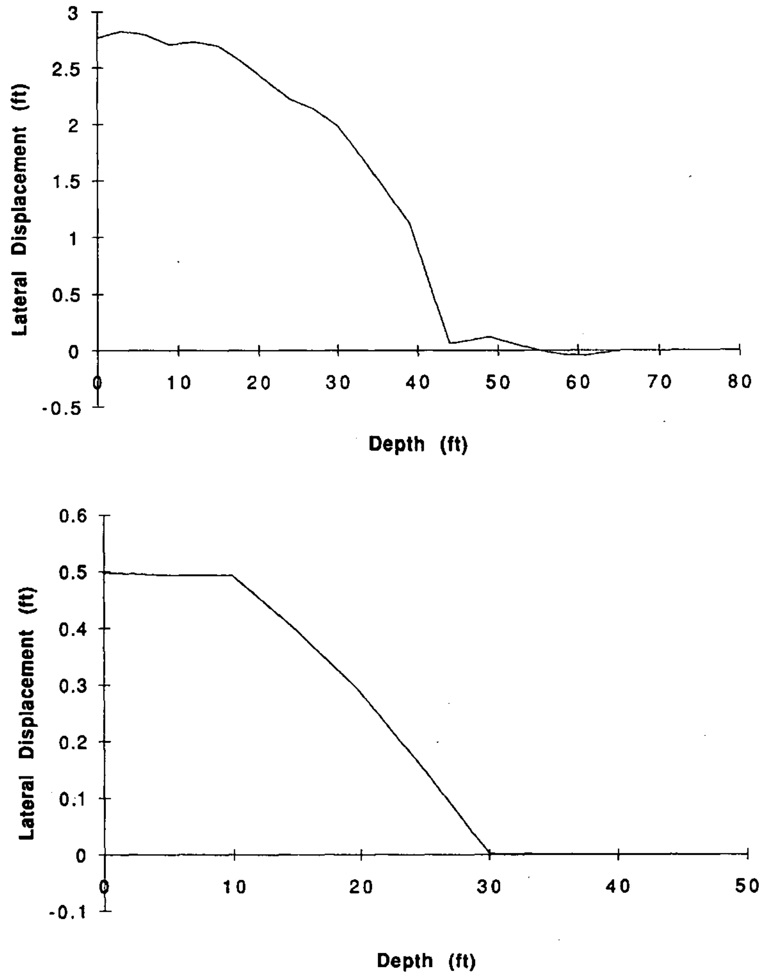

FIGURE 9 Computed free-field soil displacement profiles at time of maximum curvature: (*top*) ground-response Zone 2 and (*bottom*) ground-response Zone 3.

3. The pile elements were then assigned properties that produced the flexural stiffnesses of the various piles being considered.

4. The modified finite element model was subjected to the nodal point forces computed in Step 2. The resulting lateral displacements of the nodal points along the centerline of the pile were then used to compute the curvature demand on the piles.

Assumptions and Justification

This approximate method of interaction analysis involved a number of simplifying assumptions. First, the interaction analysis was static, not dynamic. Second, inertial forces at the pile heads resulting from the mass of the superstructure were not modeled. Third, the P-D effect associated with large lateral deflections was not modeled.

Though the interaction analysis itself was performed statically, the displacements imposed on the piles were obtained by dynamic analysis. At the time of maximum curvature for each ground-response zone, accelerations were small, particularly at the depth of maximum curvature; consequently the error involved in the static interaction analysis was assumed to be small. Because accelerations were low, inertial forces at the pile head were not large, and those that would develop would be resisted by the soil near the ground surface rather than the soil at the depth of maximum curvature; consequently, the error associated with neglecting these inertial forces was assumed to be small. Because the vertical loads were relatively small, bending moments due to the P-D effect were estimated to be much smaller than those caused by seismically induced curvature of the surrounding soil profile.

Results

The approximate interaction analyses indicated that the curvatures induced in the piles would be considerably lower than the free-field soil profile curvatures. The influence of the flexural stiffness of the piles is most easily illustrated by comparing the deflected shape of the piles and the free-field soil profile at the times of maximum curvature, as shown in Figure 11, in which it may be seen that the flexural stiffness of the piles allows the kink in the free-field soil profile to be bridged with considerably smaller pile curvatures. The reduction in maximum curvature for the various types of pile is shown in Table 2. Obviously, the maximum computed pile curvatures are considerably smaller than the maximum computed free-field soil profile curvatures. The curvature reduction clearly

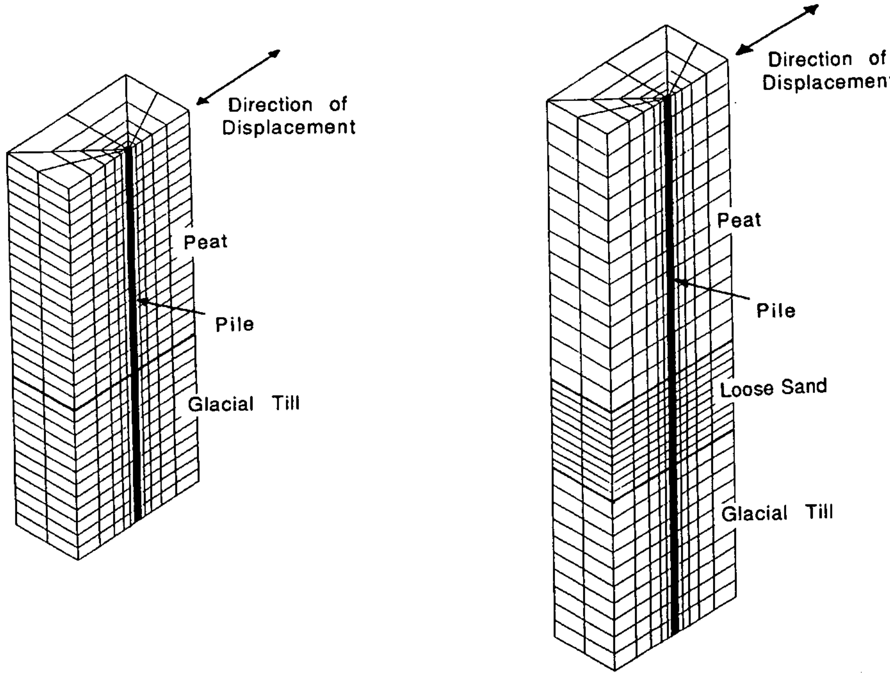

FIGURE 10 **Finite element meshes used in approximate soil-pile interaction analyses: (*left*) ground-response Zone 2 and (*right*) ground-response Zone 3.**

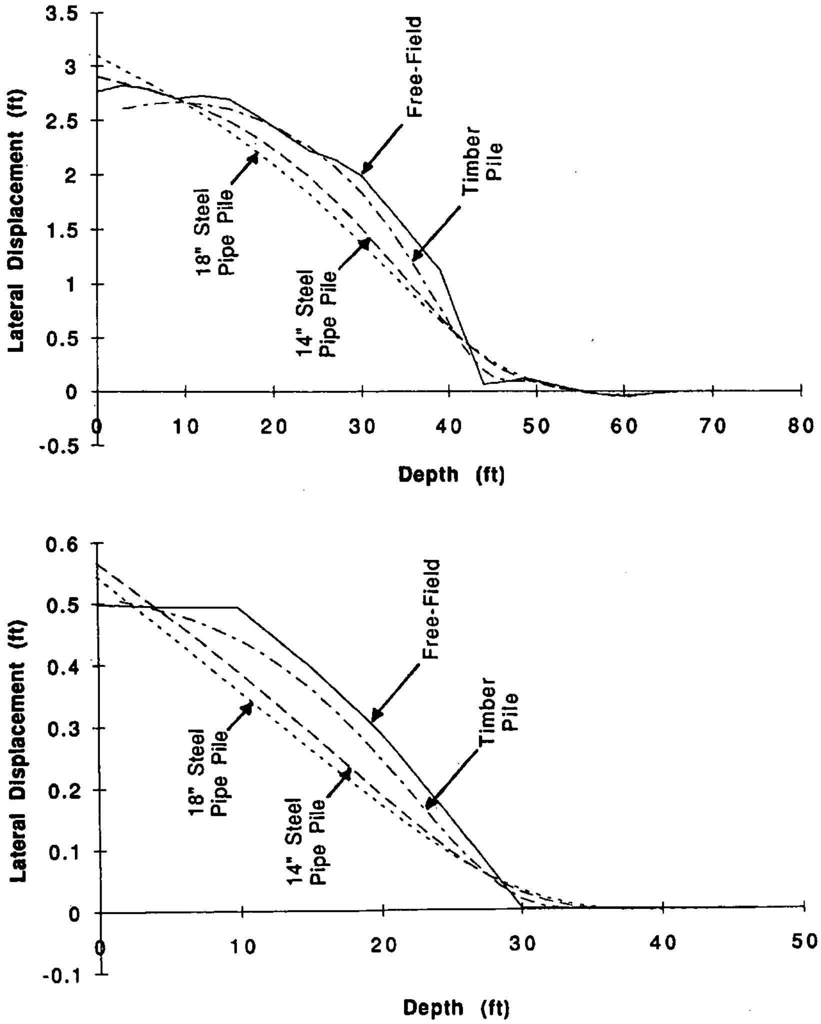

FIGURE 11 **Computed pile and free-field soil displacement profiles: (*top*) ground-response Zone 2 and (*bottom*) ground-response Zone 3.**

TABLE 2 Maximum Computed Pile and Free-Field Soil Curvatures

	Ground Response Zone 2			Ground Response Zone 3		
	14" Pipe	18" Pipe	Timber	14" Pipe	18" Pipe	Timber
$\phi_{max,pile}$	1.5 (0.59)	1.1 (0.45)	3.6 (1.40)	29.2 (11.5)	25.7 (10.1)	42.4 (16.7)
$\phi_{max,soil}$	14.5 (5.7)	14.5 (5.7)	14.5 (5.7)	150 (59)	150 (59)	150 (59)
$\frac{\phi_{max\ pile}}{\phi_{max\ soil}}$	0.103	0.079	0.246	0.195	0.171	0.283

Note: Values are in $in^{-1} \times 10^{-4}$ ($cm^{-1} \times 10^{-4}$).

varies with the flexural stiffness of the pile; greater curvature reduction is associated with greater flexural stiffness. However, the computed pile curvatures are still very large. Analysis of the influence of such large curvatures on the structural integrity of the piles is currently under way.

SUMMARY AND CONCLUSIONS

Ground-response and approximate soil-pile interaction analyses were performed to estimate the curvature demand on existing piles supporting bridges that cross a peat-filled slough. The ground-response analyses indicated that large strains would be induced in the peat near its interface with the underlying soil and that these large strains would produce large, localized curvature in the soil displacement profile. The approximate soil-pile interaction analysis indicated that the maximum pile curvatures would be considerably smaller than the maximum soil profile curvatures, but that they would still be very large.

ACKNOWLEDGMENTS

The research described in this paper was supported by the Washington State Department of Transportation and the National Science Foundation. Graduate and undergraduate students Fan-Yie von Laun, N. Sivaneswaran, Jun Xu, Jerry Wu, Peter Sajer, Terry Martin, and Kuan Sian Go assisted in various field testing, laboratory testing, and analytical aspects of the research.

REFERENCES

1. Matlock, H., and L. C. Reese. Generalized Solution for Laterally Loaded Piles. *Journal of the Soil Mechanics and Foundations Division,* ASCE, Vol. 86, No. SM5, 1960, pp. 63–91.
2. Matlock, H. Correlations for Design of Laterally Loaded Pile in Soft Clays. Paper OTC 1204. *Second Annual Offshore Technology Conference,* Houston, Texas, 1970, Vol. 1, pp. 577–594.
3. Nogami, T., and M. Novak. Resistance of Soil to a Horizontally Vibrating Pile. *Journal of Earthquake Engineering and Structural Dynamics,* Vol. 5, 1977, pp. 249–261.
4. Blaney, G. W., and M. W. O'Neill. *Lateral Response of a Single Pile in Overconsolidated Clay to Relatively Low-Frequency Pile-Head Loads and Harmonic Ground Surface Loads.* Research Report UHCE 83-19. Department of Civil Engineering, University of Houston, Houston, Texas, 1983.
5. Kachedoorian, R. *Effects of Earthquake of March 27, 1964 on the Alaska Highway System.* Professional Paper 545-C. U.S. Geological Survey, Department of Interior, Washington, D.C., 1968.
6. Elliot, A. L., and I. Nagai. Earthquake Damage to Freeway Bridges. In *San Fernando, California Eathquake of February 9, 1971,* Vol. 2, National Oceanic and Atmospheric Administration, U.S. Department of Commerce, 1973.
7. Hideaki, K. Damage of Reinforced Precast Piles During the Miyagi-Ken-Oki Earthquake of June 12, 1972. *Proc., 7th World Conference on Earthquake Engineering,* Vol. 9, Istanbul, Turkey, 1980.
8. Margason, E., and D. M. Holloway. Pile Bending During Earthquakes. *Proc., 6th World Conference on Earthquake Engineering,* Vol. 4, New Delhi, India, 1977.
9. *Tentative Provisions for the Development of Seismic Regulations for Buildings.* Report ATC-3-06. Applied Technology Council, Redwood City, Calif., 1984, 599 pp.
10. Banerjee, S., J. F. Stanton, and N. M. Hawkins. Seismic Performance of Precast Prestressed Concrete Piles. *Journal of Structural Engineering,* ASCE, Vol. 113, No. 2, 1987, pp. 381–396.
11. Heaton, T. H., and H. Kanamori. Seismic Potential Associated with Subduction in the Northwestern United States. *Bulletin of the Seismological Society of America,* Vol. 74, No. 3, 1984, pp. 933–941.
12. Atwater, B. F. Evidence for Great Holocene Earthquakes Along the Outer Coast of Washington State. *Science,* Vol. 236, No. 4804, 1987, pp. 942–944.
13. Newman, D. E. *A 13,500 Year Pollen Record from Mercer Slough, King County, Washington.* M.S. thesis. Department of Geological Sciences, University of Washington, Seattle, 1983.
14. Galster, R. W., and W. T. Laprade. Geology of Seattle, Washington, United States of America. *Bulletin of the Association of Engineering Geologists,* Vol. 28, No. 3, 1991, pp. 235–302.
15. Kramer, S. L., R. Satari, and A. P. Kilian. Evaluation of In Situ Strength of a Peat Deposit from Laterally Loaded Pile Test Results. In *Transportation Research Record 1278,* TRB, National Research Council, Washington, D.C., 1990, pp. 103–109.
16. Kramer, S. L. *Behavior of Piles in Full-Scale, Field Lateral Loading Tests.* Washington State Transportation Center, Seattle, Oct. 1991, 79 pp.
17. *Thomson Way Crossing of Union Bay, Seattle, Washington.* Foundation Engineering Report Technical Supplement, Shannon & Wilson, Nov. 1967, 108 pp.
18. Seed, H. B., and I. M. Idriss. Analyses of Ground Motions at Union Bay, Seattle During Earthquakes and Distant Nuclear Blasts. *Bulletin of the Seismological Society of America,* Vol. 60, No. 1, 1970, pp. 125–136.
19. Vucetic, M., and R. Dobry. Effect of Soil Plasticity on Cyclic Response. *Journal of Geotechnical Engineering,* ASCE, Vol. 117, No. 1, 1991, pp. 89–107.
20. Idriss, I. M., and H. B. See. Seismic Response of Soil Deposits. *Journal of the Soil Mechanics and Foundations Division,* ASCE, Vol. 96, 1970.
21. Schnabel, P. B., J. Lysmer, and H. B. Seed. SHAKE: A Computer Program for Earthquake Response Analysis of Horizontally Layered Sites. Report EERC 72-12. Earthquake Engineering Research Center, University of California, Berkeley, Dec. 1972, 88 pp.
22. *Seismic Design Guidelines for Highway Bridges.* Report ATC-6. Applied Technology Council, Redwood City, Calif., 1986, 204 pp.

Publication of this paper sponsored by Committee on Engineering Geology.