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ENERGY DISSIPATION IN SOIL-STRUCTURE INTERACTION

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SUMMARY

Energy dissipation as a means of reducing the seismic response of structures has become a popular topic among researchers and structural engineers who have developed and implemented devices, such as friction dampers, fluid dampers, and isolators, in the retrofit of structures. However, a natural source of energy dissipation is the interaction between a structure, its foundation, and the supporting soil medium. This interaction can be significant and potentially beneficial in certain situations, resulting in large reductions in seismic response. Unfortunately, this phenomenon is often ignored by many structural engineers because of their lack of knowledge of (1) theoretical principles governing soil-structure interaction (SSI), (2) circumstances when SSI is potentially important, (3) the field measurements of modal damping ratios, and (4) methods to estimate modal damping ratios in soil-structure systems. This knowledge gap exists primarily because the SSI subject is not generally taught at the undergraduate or graduate levels in university civil engineering departments. The theoretical principles, involving wave-propagation theory, boundary-value problems, and soil and structural mechanics/dynamics, are daunting to most civil engineering students with design-oriented career goals. Nevertheless, assuming the frequency-dependent foundation impedance functions can be obtained for a particular SSI system from the literature or from a consultant, relatively simple and practical systems-identification methods can be used to estimate the composite modal damping ratios for the significant modes of vibration. SSI experiments and theoretical calculations using these simple models have yielded relatively large modal damping ratios in certain situations for structures such as short-span bridges, offshore concrete gravity platforms, nuclear power plant containments, fuel storage tanks, short to mid-rise buildings, and nuclear waste processing plants.

INTRODUCTION

This paper, which is intended primarily for the professional structural engineer engaged in seismic design, first reviews the general state of SSI practice in the consulting engineering/structural design professions from the standpoint of energy dissipation in SSI systems. In dynamic analysis of structures, modal damping ratios between 0.03 and 0.07 (with a nominal value of 0.05) are typically used in practice. These values are generally acknowledged as structural damping ratios and are usually valid for a rigid or nearly rigid-base model of the structure. However, these values are often used in flexible-base models, which can result in an overestimation of the seismic loads.

In the section, Composite Modal Damping. technical justification for higher damping ratios for flexible-base models is provided by reviewing the concept of composite modal damping for soil-structure systems and the factors that affect it. Next, experimental data on modal damping of soil-structure systems are reviewed in the section, Measurements of Composite Modal Damping. In the subsequent section, Estimate of Composite Modal Damping, a simple systems identification procedure is described for estimating modal damping ratios in SSI systems with one or more foundations and frequency-dependent foundation impedance functions. Details of an example calculation of the modal damping ratios illustrating this procedure for a liquid fuel storage tank are described at the end of this section. A discussion of the implications of energy dissipation in SSI for seismic design is presented in the last section of this paper.

STATE OF PRACTICE

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The state of the practice for modeling energy dissipation in SSI analysis for structures other than nuclear power plants is typically as follows. The structural engineer usually performs the analysis and constructs a model of the structure from the element library in a commercial software program for structural dynamics. Most of these element libraries have rotational and translational springs of constant stiffness that can be attached to the base of the structural model to simulate the foundation-soil interaction. The structural engineer will usually consult a geotechnical engineer for these foundation stiffnesses, but will not usually request estimates of the foundation damping. The structural analysis software typically solves the equations of motion using modal superposition.

The input motion is usually defined by the geotechnical engineer or engineering seismologist and consists of design spectra corresponding to damping ratios specified by the structural engineer, who typically selects values in the aforementioned 0.03 – 0.07 range.

This practice of incorporating SSI effects has remained fairly constant during the last 20 years. While more structural engineers recognize the importance of including foundation flexibility in their models, there has been a reluctance to properly incorporate the damping into the system. Most popular commercial structural dynamics software programs do not include viscous damping elements. Even if the provision for specifying the element damping were available in these programs, most engineers would have difficulty specifying the viscous damping constants for each element because, unlike the relatively simple methods for generating the element stiffnesses, the procedures for estimating the values of a system damping matrix, are more complex. Available procedures are obscure, unknown, or difficult to understand by many structural engineers. Until they become familiar and comfortable with these procedures, the usual practice of arbitrarily adopting modal damping values around 0.05 will continue.

Many structural engineers are presently using software for nonlinear inelastic dynamic analysis of structures. The hysteretic damping from the specification of the load-deflection characteristics of the structural elements is automatically included in the structural models, but the damping for the foundation elements is more difficult to specify because of the anelastic (hysteretic) damping of the soil and the frequency-dependent radiation damping of the foundation-soil medium.

In the penultimate section of this paper, a simple example is provided illustrating the calculation of modal damping ratios for the fundamental mode of vibration of a tank-foundation system. Even in this example, the geotechnical engineer needed to recognize and properly estimate the two components of foundation damping (hysteretic and radiation).

COMPOSITE MODAL DAMPING

The composite modal damping ratio for each mode of vibration of a soil-structure system depends on the foundation damping, the structural damping, and the nature and degree of interaction between the structure and supporting soil. The foundation damping consists of the material (or hysteretic) damping of the soil and the radiation damping associated with the generation and propagation of seismic waves into the soil medium by the motion of the foundation relative to the free-field earthquake motion. The material damping primarily depends on strain induced in the soil during the shaking [e.g., Seed and Idriss, 1970], whereas the radiation damping depends on the elastic properties of the surrounding soil and the shape and embedment of the foundation. For a given soil profile and foundation geometry, the radiation damping depends on the mode of foundation vibration, e.g., vertical translation, horizontal translation, rotation, or a combination of translation and rotation [Richart et al., 1970; Gazetas, 1983].

Generally, foundation damping is highest for vertical and horizontal vibration and lowest for rocking motion. However, even for rocking modes, the damping ratios can be significantly larger than the nominal 5% critical damping ratio typically assumed for structures such as buildings and bridges. Thus, foundation damping is normally much higher than structural damping. In qualitative terms, the composite modal damping for a given soil-structure system with given amounts of foundation and structural damping will depend on the amount of deformation in the structure relative to the foundation movement. For example, the composite modal damping for stiff structures on flexible soils is expected to be greater than the composite modal damping for flexible structures on stiff soils. This conclusion is easily seen in the limiting cases of an infinitely rigid structure on a flexible soil or a flexible structure founded on hard bedrock (i.e., rigid-base structure).

MEASUREMENTS OF COMPOSITE MODAL DAMPING

Composite modal damping ratios have been measured in a variety of soil-structure systems including simple footings, pile foundations, multistory buildings, bridges, and a scale-model nuclear plant containment structure.

These damping ratios were obtained from soil-structure responses measured during forced vibration tests or earthquake excitations. The results for several soil-foundation and soil-foundation-structural systems familiar to the author are presented in Table 1. A summary of these case histories is provided in Crouse [1999] and Crouse and Werner [1995]; further details can be found in the references listed in Table1. A comprehensive compilation of SSI parameters (including modal damping ratios) estimated from earthquake responses recorded at 58 building sites is presented in Stewart and Stewart [1997].

Table 1. Measured Modal Damping Ratios

No.	Structure	Foundation Type & Soil	Excitation	Damping Ratios	Reference
1	Concrete pads 1.2m x 1.2m x (0.1-0.6m)	Footing on moderately stiff clay and/or sand	Forced vibration	0.20 – 0.40	Crouse et al, 1984; Crouse & Hushmand, 1989
2	Masonry block shelter 2.4m x 2.4m x 2.4m	Perimeter wall on moderately stiff clay and sand	Forced vibration	0.29	Crouse et al, 1992
3	Nuclear power plant containment - ¼-scale	Mat on moderately stiff alluvium	Forced vibration	0.10 – 0.21	Tajimi Eng. Ser., 1989
			Earthquake	0.22	
4	1-span RC bridge length = 31.4m	Footing on stiff sand (abutments)	Forced vibration	0.035 – 0.15	Crouse & et al, 1987
5	2-span RC bridge length = 63.4m	Footing on stiff clay (abutments)	Forced vibration	0.032 – 0.12	Werner et al, 1990
			Earthquake	0.10 – 0.26	Werner et al, 1993
6	3-span RC bridge length = 43.3m	Footing on dense silty sand (abutments)	Forced vibration	0.062 – 0.091	Rodehauer, 1993

The damping ratios reported in Table 1 pertain to lateral excitation inducing coupled translation and rocking displacements in the foundation. For the three bridges, the forced vibration excitation was in the transverse direction, and the lower limit damping ratios for these cases generally resulted from vibrational modes with much greater deformation in the superstructure relative to the foundation displacement. The variation in damping ratio reported for the 2-span bridge (Meloland Road Overcrossing in El Centro, California) during earthquake excitation (1979 Imperial Valley event) pertains to the fundamental transverse mode only, and this range of values resulted primarily from uncertainties associated with the lack of response measurements to adequately characterize the soil-structure interaction.

ESTIMATION OF COMPOSITE MODAL DAMPING

This section first describes a systems-identification method for the estimation of modal damping in SSI systems where the foundation impedance functions are frequency and strain dependent. Next, the method is illustrated in an example calculation of composite modal damping for a liquid natural gas (LNG) tank.

Systems-Identification Method

Systems-identification methods typically have been used to estimate system parameters (e.g., natural frequencies, damping ratios, and foundation stiffnesses) from response data recorded during vibration tests or earthquakes. These methods also can be adapted to SSI analyses for structures in the design stage in cases where the structures are supported on multiple foundations and/or where the frequency dependence of the foundation impedance functions is significant. For such cases, closed-form solutions for modal damping ratios do not presently exist, although solutions have been developed for the case of single foundations supported on a medium in which the foundation impedance functions (stiffness and damping coefficients) are frequency independent [e.g., Luco, 1981; ASCE, 1986].

The systems-identification method, presented herein, is applicable to linear SSI systems and is as follows. Equations of motion are derived in the frequency domain for the SSI system in generalized spatial coordinates and in modal coordinates. These two sets of equations are used to develop transfer functions (one for generalized coordinates and another for modal coordinates) that express the ratio of the motion at some point on the structure to the free-field motion as a function of frequency. The transfer function of the model in generalized coordinates is presumed to be known because the structural masses, moments of inertia, stiffnesses and damping constants, as well as the foundation impedance functions, are known or can be computed. On the other hand, the natural frequencies and damping ratios in the transfer function for the SSI model based on modal coordinates are unknown. The values of these parameters are varied until the two transfer functions are similar.

In the generalized coordinate system, the specification of the structural damping is straightforward for a 1 degree-of-freedom (d.o.f.) representation of the structure, which is an acceptable approximation for a number of SSI systems. In this case, the damping coefficient, *c*, is computed using the formula, $c = 2\varsigma\sqrt{km}$, for a 1 d.o.f. oscillator, where ζ is the assumed modal damping ratio for the oscillator (fixed-base structure), and *k* and *m* are the known oscillator stiffness and mass, respectively. In cases where a multi-d.o.f. model is required for the structure, the structural damping matrix can be computed by fairly simple matrix algebra [e.g., Tsai, 1969] by (1) assigning modal damping ratios (ζ_i) for each mode of vibration of the fixed-base model of the structure, and (2) assuming these ratios are factors in the diagonal elements, $2\ \omega_i\zeta_i$, of the diagonal damping matrix of the modal equivalent of the fixed-base structure, which is assumed to possess classical normal modes.

For many applications in practice, the foundation impedance functions can be obtained from the literature or from commercially available computer codes, e.g., DYNA3 [Novak et al, 1991], SUPELM [Kausel, 1992].

The impedance functions in the literature are usually presented in graphs or tables for assumed values of the material damping of the soil. In the case of uniform material damping of the soil medium, the published impedance functions can be easily adjusted to account for a different material damping [Wong and Luco, 1981]. This material damping ratio can be estimated from published strain-dependent damping ratio curves or can be derived from laboratory tests on soil samples. In either case, the effective shear strain in the soil must be estimated based on the design ground motion.

Example Calculation of Composite Modal Damping for a LNG Tank

Dames & Moore recently participated in a project involving the seismic design of a large cylindrical steel flat-bottom LNG tank in a region of moderate seismic activity. The tank was to be supported on a mat foundation resting on the surface of improved soil. The structural designer was planning to conduct a linear dynamic analysis using the design response spectra corresponding to the appropriate modal damping ratios associated with the impulsive and convective modes of the tank-fluid-foundation-soil system.

The composite modal damping of primary interest was for the fundamental impulsive mode. The model used to compute this ratio was similar to the model of Veletsos and Tang [1990]. It consisted of a single-degree-of-freedom oscillator representing the impulsive mode of the fixed-base tank attached to a circular foundation mat supported on a visco-elastic half space. The relevant parameters of the model for the Operating Basis Earthquake (OBE) and Safe Shutdown Earthquake (SSE) levels of shaking (0.2 g and 0.4 g, respectively) are summarized in Table 2.

Table 2. Model Parameters for LNG Tank Example

Model Element	**Parameter**	**Parameter Value**	
		OBE	**SSE**
Tank Shell & Fluid	Impulsive Liquid Mass	3.35 x 107 kg	3.35 x 107 kg
	Impulsive Mode Frequency	3.9 Hz	3.9 Hz
	Impulsive Mode Damping Ratio	0.03	0.07
	Height of Impulsive Mass	11.9 m	11.9 m
	Height of Liquid, H	31.8 m	31.8 m
	Inner Tank Radius, a	40.0 m	40.0 m
Tank Foundation	Radius	44.5 m	44.5 m
	Mass	1.23 x 107 kg	1.23 x 107 kg
	Mass Moment of Inertia	7.10 x 109 kg-m2	7.10 x 109 kg-m2
Half Space	Shear-Wave Velocity	263 m/s	204 m/s
	Density	1.76 gm/cc	1.76 gm/cc
	Poisson's Ratio	0.3	0.3
	Material Damping Ratio	0.10	0.15

The modal damping ratio was estimated by deriving equations for the Transfer Function (TF) between the tank-displacement motion and the free-field, ground-displacement motion induced by the earthquake. One TF was expressed in terms of the generalized coordinates, mass, damping, and stiffness quantities, while the other TF was expressed in terms of the modal quantities (i.e., natural frequencies, modal damping ratios, mode shapes, and participation factors). The material damping ratio of the soil was incorporated by modifying the foundation impedance functions for a circular disc on an elastic half space without material damping. The procedure for the modification is given in Gazetas [1983] or Wong and Luco [1981].

The modal damping ratio for the fundamental mode of the system that provided similarity in the moduli of the two TFs was estimated. Using best estimates for the parameters of the tank-foundation soil model (Table 2), modal damping ratios of 0.19 and 0.28 were estimated for the OBE and SSE, respectively. Comparisons between the resulting two TFs of the OBE and SSE are shown in Figures 1a and 1b, respectively.

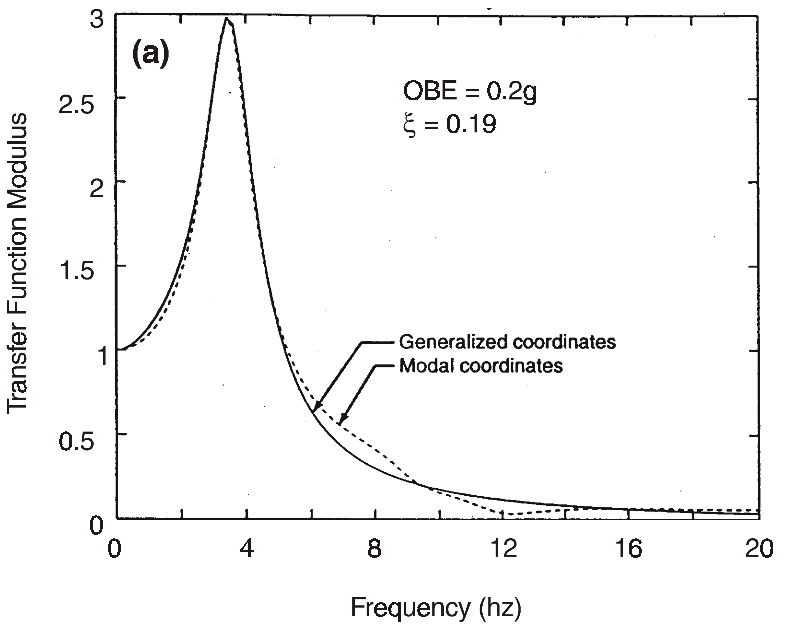
The relatively high modal damping ratios estimated for the OBE and SSE are the result of the significant interaction between the tank foundation and underlying soil. These results are consistent with those in Veletsos and Tang [1990] in the sense that composite modal damping values much higher than the structural damping value are expected for tanks with small liquid height to inner tank radius ratios (H/a) on relatively flexible soils.

DISCUSSION

Experimental test results and predictions from theoretical models clearly demonstrate that relatively large composite modal damping ratios are possible when SSI effects are significant. Furthermore, when properly substantiated by appropriate SSI analysis procedures, the use of relatively large composite damping values for the computation of seismic loads is accepted practice in the U.S. nuclear power industry. All of this experimental/theoretical evidence and the nuclear industry precedent suggest that in the case of other important structures, composite modal damping values larger than the structural damping ratios should be considered in the calculation of the seismic loads. However, the composite modal damping values determined by theoretical models are not necessarily those that should be used in final design. The final modal damping values should consider uncertainties associated with the SSI model and its parameters, relevant experimental data, and the degree of conservatism desired for the design.

One potentially beneficial effect that was not considered in the SSI model for the tank example is the filtering or reduction in high frequency ground motion by the passage of seismic waves across the tank foundation. During the last 25 years, this effect has been observed in buildings with foundation areas similar to that of the example tank. The high-frequency filtering phenomenon for foundations of this size on soils of roughly the same stiffness has been observed for frequencies greater than about 2 Hz [e.g., Yamahara, 1970; Crouse and Jennings, 1975; Newmark et al., 1977; Fenves and Serino, 1992; Stewart and Stewart, 1997]. The filtering, or kinematic interaction, is caused by (1) incoming seismic waves at angles of incidence less than 90° (vertical propagation), and (2) incoherence in the ground motion due to wave scattering from inhomogeneities in the local geology and anelastic attenuation.

The size of the example tank foundation (~ 80 m diameter), the flexibility of the underlying soil, and the estimated fundamental impulsive frequency of the tank-soil system (~ 3½ Hz) were factors collectively suggesting that filtering of ground motion at this frequency may occur, thus reducing the impulsive seismic load on the tank. The observational data indicate that a modest reduction in the OBE and SSE design spectra at short periods to account for this effect (which is equivalent to increasing the damping) was appropriate. However, the inclusion of this effect in the development of site-specific design spectra for particular structures should be coordinated with the structural engineer performing the dynamic analysis and design. The basis and amount of any reduction in the design spectra due to kinematic interaction should be well documented by the professional developing the design spectra.



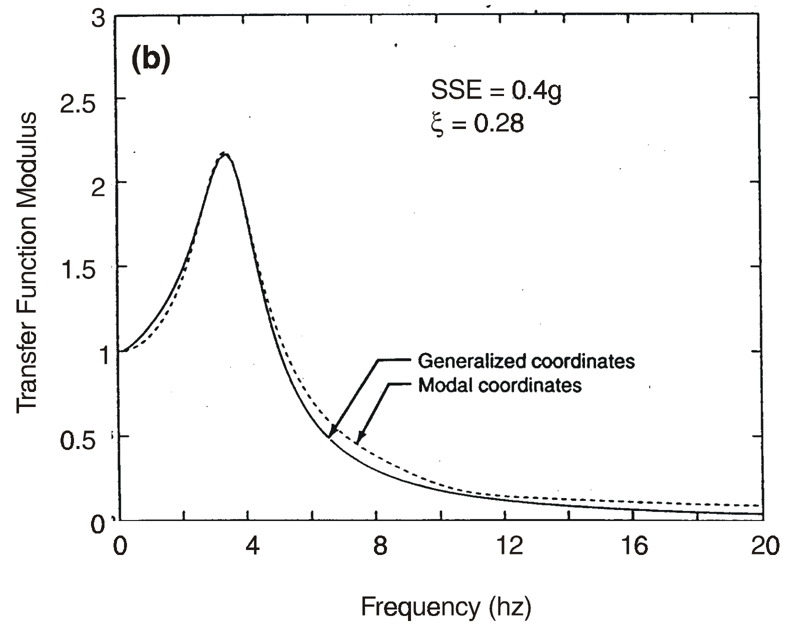


Figure 1. Transfer functions and fundamental modal damping ratios for tank example for: (a) OBE, and (b) SSE.

REFERENCES

ASCE (1986), *Seismic Analysis of Safety-Related Nuclear Structures and Commentary on Standard for Seismic Analysis of Safety-Related Nuclear Structures*, American Society of Civil Engineers, ASCE Standard 4-86.

Crouse, C.B. (1999), "Energy dissipation in soil-structure interaction: a consultant's perspective", Proceedings UJNR Workshop on Soil-Structure Interaction, pp 6-1 – 6-17.

Crouse, C.B. and Hushmand, B. (1989), "Soil-structure interaction at CDMG and USGS accelerograph stations", *Bull. Seism. Soc. Am.*, 79, 1, pp 1-14.

Crouse, C.B. and Hushmand, B., and Martin, G.R. (1987), "Dynamic Soil-Structure Interaction of a Single-Span Bridge", *J. Earthq. Eng. And Struct. Dyn.*, 15, pp 711 – 729.

Crouse, C.B. and Werner, S.D. (1995), "Estimation of modal damping for bridges", *Proceedings of 4th U.S. Conf. on Lifeline Earthq. Eng.*, pp 408-415.

Crouse, C.B., and Jennings, P.C. (1975), "Soil-structure interaction during the San Fernando earthquake", *Bull. Seism. Soc. Am.*, 65, 1, pp 13-36.

Crouse, C.B., Trahern, E., and Price, T. (1992), *Nonlinear Site Response at the Differential Array Station during the 1979 Imperial Valley, California, Earthquake,* Report to U.S. Nuclear Regulatory Commission and National Science Foundation.

Crouse, C.B., Liang, G.C., and Martin, G.R. (1984), "Experimental study of soil-structure interaction at an accelerograph station", *Bull. Seism. Soc. Am.*, 74, 5, pp 1995-2013.

Fenves, G.L. and Serino, G. (1992), *Evaluation of Soil-Structure Interaction in Buildings during Earthquakes*, Calif. Strong Motion Instrumentation Program, CSMIP/92-01.

Gazetas, G. (1983), "Analysis of machine foundation vibrations, state of art", *J. Soil Dyn. and Earthq. Eng.*, 2, 1, pp 2-42.

Kausel, E. (1992), SUPELM, Ver. 2.0, *Foundations Embedded in Layered Media: Dynamic Stiffnesses and Response to Seismic Waves*.

Luco, J.E. (1981), *Linear Soil-Structure Interaction*, Lawrence Livermore Laboratory Report (UCRL-53011) to U.S. Nuclear Regulatory Commission (NUREG/CR-1780).

Nwmark, N.M., Hall, W.J., and Morgan, J.R. (1977), "Comparison of Building Response and Free-Field Motion in Earthquakes", *Proceedings 6th World Conf. Earthq. Eng.*, 3, pp 972-978.

Novak, M., Sheta, M., El-Hifnawy, L., El-Marsafawi, H., and Ramadan, O. (1991), *DYNA3, A Computer Program for Calculation of Foundation Response to Dynamic Loads, User Manual,* Univ. of Western Ontario Geotechnical Research Centre, Rept. No. GEOP90-02, 1, Rev. 2

Richart, F.E., Hall J.R. and Woods, R.D. (1970), *Vibrations of Soils and Foundations*, Prentice-Hall, Englewood Cliffs, NJ.

Rodenhauer, S.M. (1993), *Dynamic Testing and Analysis of a Reinforced Concrete Bridge*, M.S. thesis, University of Washington, Department of Civil Engineering.

Seed, H.B. and Idriss, I.M. (1970), *Soil Moduli and Damping Factors for Dynamic Response Analysis*, University of California, Berkeley, Earthquake Engineering Research Center, ERRC 70-10.

Stewart, J.P. and Stewart, A.F. (1997), *Analysis of Soil-Structure Interaction Effects on Building Response from Earthquake Strong Motion Recordings at 58 sites,* University of California, Berkeley, Earthquake Engineering Research Center, Report No. UBC/EERC-97/01.

Tajimi Engineering Services, Ltd. (1989), "Lotung SSI analysis using a 3-D harmonic Green's function method", *Proceedings EPRI/NRC/TPC Workshop on Seismic Soil-Structure Interaction Analysis Techniques using Data from Lotung, Taiwan*, 2, EPRI NP-6154, pp 16-1 to 16-50.

Tsai, N-C (1969), *Influence of Local Geology on Earthquake Ground Motion,* Ph.D. Thesis, California Institute of Technology, Pasadena, CA, 201 p.

Veletsos, A.S. and Tang, Y. (1990), "Soil-Structure Interaction Effects for Laterally Excited Liquid Storage Tanks", *J. Earthq. Eng. and Struct. Dyn.*, 19, pp 473-496.

Werner, S.D., Nisar, A., and Beck, J.L. (1990), *Dynamic Response Characteristics of Meloland Road Overcrossing during Quick-Release Hydraulic Ram Tests*, Dames & Moore Informational Report to Caltrans.

Werner, S.D., Crouse, C.B., Katafygiotis, L., and Beck, J.L. (1993), *Model Identification and Seismic Analysis of Meloland Road Overcrossing,* Dames & Moore Informational Report to Caltrans.

Wong, H.L., and Luco, J.E. (1981), *Identification of Sensitive Parameters for Soil-Structure Interaction,* Lawrence Livermore Laboratory Report (UCRL-15493) to U.S. Nuclear Regulatory Commission (NUREG/CR-3044).

Yamahara, H. (1970), "Ground motions during earthquakes and the input loss of power to an excitation of buildings", *Soils and Foundations*, 10, 2, pp 145-161.