A DETERMINISTIC ASSESSMENT OF EFFECTS OF GROUND MOTION INCOHERENCE

by

A.S. Veletsos[1] and Y. Tang[2]

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1 Brown and Root Professor, Department of Civil Engineering, Rice University
2 Post-Doctoral Research Associate, Department of Civil Engineering, Rice University

NATIONAL CENTER FOR EARTHQUAKE ENGINEERING RESEARCH
State University of New York at Buffalo
Red Jacket Quadrangle, Buffalo, NY 14261

PREFACE

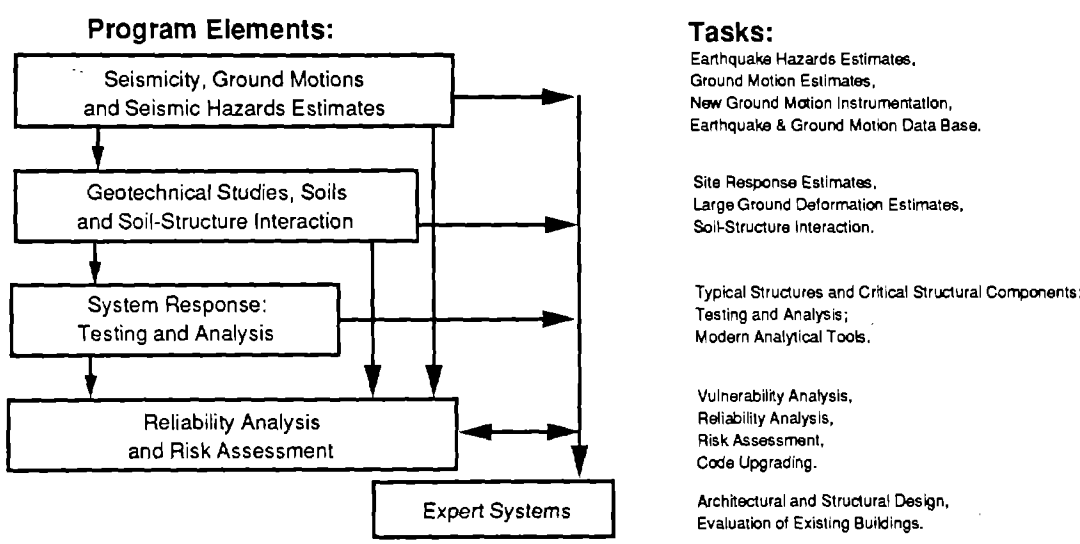
The National Center for Earthquake Engineering Research (NCEER) is devoted to the expansion and dissemination of knowledge about earthquakes, the improvement of earthquake-resistant design, and the implementation of seismic hazard mitigation procedures to minimize loss of lives and property. The emphasis is on structures and lifelines that are found in zones of moderate to high seismicity throughout the United States.

NCEER's research is being carried out in an integrated and coordinated manner following a structured program. The current research program comprises four main areas:

- Existing and New Structures
- Secondary and Protective Systems
- Lifeline Systems
- Disaster Research and Planning

This technical report pertains to Program 1, Existing and New Structures, and more specifically to geotechnical studies.

The long term goal of research in Existing and New Structures is to develop seismic hazard mitigation procedures through rational probabilistic risk assessment for damage or collapse of structures, mainly existing buildings, in regions of moderate to high seismicity. The work relies on improved definitions of seismicity and site response, experimental and analytical evaluations of systems response, and more accurate assessment of risk factors. This technology will be incorporated in expert systems tools and improved code formats for existing and new structures. Methods of retrofit will also be developed. When this work is completed, it should be possible to characterize and quantify societal impact of seismic risk in various geographical regions and large municipalities. Toward this goal, the program has been divided into five components, as shown in the figure below:



Geotechnical studies constitute one of the important areas of research in Existing and New Structures. Current research activities include the following:

1. Development of linear and nonlinear site response estimates.
2. Development of liquefaction and large ground deformation estimates.
3. Investigation of soil-structure interaction phenomena.
4. Development of computational methods.
5. Incorporation of local soil effects and soil-structure interaction into existing codes.

The ultimate goal of projects concerned with geotechnical studies is to develop methods of engineering estimation of large soil deformations, soil-structure interaction, and site response.

The problem of the response of a soil-structure system to incoherent earthquake ground motion has been addressed in the past by using a formal stochastic approach (NCEER-88-0021). In the present work, the same problem is addressed by an alternative formulation which may be more appealing to those who are accustomed or prefer to think in deterministic terms. Specifically, the input motion at the control point is described deterministically and not stochastically as in the formal stochastic approach. The results obtained by the proposed procedure have been shown to be consistent with those obtained in the original study by formal application of the stochastic approach.

ABSTRACT

An approximate deterministic method of analysis is presented for assessing the effects of ground motion incoherence and of the associated soil-structure interaction on the seismic response of structure-foundation-soil systems. The free-field ground motion in this approach is specified by an acceleration history and a spatial incoherence function. Numerical solutions are presented which illustrate the procedure and elucidate the nature and relative importance of the kinematic and inertial interaction effects. The results are shown to be consistent with those obtained in a companion recent study by formal application of the stochastic approach.

ACKNOWLEDGMENT

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INTRODUCTION

It is generally recognized that the motion which is experienced by the foundation of a structure during an earthquake may be substantially different from the motion that the ground would experience at its interface with the structure if the structure were not present (e.g., Veletsos 1977; Roesset 1980). Two factors contribute to this difference: (1) the inability of a rigid foundation to conform to the generally non-uniform, spatially varying ground motion; and (2) the interaction or coupling between the vibrating structure, its foundation and supporting soils.

The spatial variation of the free-field ground motion may be due to several factors. The seismic waves may emanate from different points of an extended source and may impinge the foundation at different instants or with different angles of incidence, or they may propagate through paths of different physical properties and may be affected differently in both amplitude and phase by the characteristics of the travel paths and by reflections from, and diffractions around, the foundation. Even when the seismic wave front is plane, it may impinge the foundation-soil interface obliquely, leading to ground motions which differ in phase from point to point. The spatial variability of the ground motion due to the propagation of a plane wave is known as the wave passage effect, whereas that due to the other, generally random, factors is known as the ground motion incoherence effect. It is the latter effect that this paper is concerned with.

An incoherent ground motion is normally specified stochastically in terms of a local power spectral density (psd) function and a spatial incoherence function; the latter function defines the interrelationship of the harmonic components of the motion at pairs of points. The foundation input motion and the response of the structure under these conditions are also expressed in stochastic terms.

The objective of the present paper is two-fold: (1) to present an alternative formulation of the problem with which deterministic estimates may be made of the responses of the structure and its foundation; and (2) to employ the procedure in a parametric study of the effects of the major parameters involved. The method of analysis presented should be particularly appealing to those who are accustomed or prefer to think in deterministic terms. Both kinematic and inertial interaction effects are examined. The kinematic effects basically reflect the effects of the nonuniformity of the ground motion, whereas the inertial interaction effects represent the effects of the dynamic coupling between the vibrating structure, foundation and supporting medium.

The structures investigated have one lateral and one torsional degrees of freedom in their fixed-base condition, and are presumed to be excited by horizontally polarized, vertically propagating, incoherent shear waves. The response quantities examined include the lateral and torsional components of the foundation input motion and of the associated structural deformations, particularly their peak values. The maximum structural deformations are displayed in the form of pseudovelocity response spectra, and they are compared, over wide ranges of the parameters involved, with those obtained for no soil-structure interaction and for kinematic interaction only. Simple, physically motivated interpretations are given for the observed differences, and the interrelationship of these results to those obtained in a companion recent study by formal application of the stochastic approach (Veletsos and Prasad 1989) is identified.

SECTION 2
SYSTEM CONSIDERED

The system investigated is shown in Fig. 2-1. It is a linear structure of mass m and height H, which is supported through a foundation of mass m_f at the surface of a homogeneous, elastic halfspace. The circular natural frequencies for the lateral and torsional modes of vibration of the structure when fixed at its base are denoted by $p_x = 2\pi f_x$ and $p_\theta = 2\pi f_\theta$, respectively, in which f_x and f_θ are the associated frequencies in cycles per second (cps); and the corresponding percentages of critical damping are denoted by ζ_x and ζ_θ, respectively. The foundation mat is idealized as a rigid circular plate of negligible thickness and radius R which is bonded to the halfspace so that no uplifting or sliding can occur, and the columns of the structure are presumed to be massless and axially inextensible. Both m and m_f are assumed to be uniformly distributed over identical circular areas. The supporting medium is characterized by its mass density, ρ, shear wave velocity, v_s, and Poisson's ratio, ν. The structure may be viewed either as the direct model of a single-story building frame or, more generally, as the model of a multistory, multimode structure that responds as a system with one lateral and one torsional degrees of freedom in its fixed-base condition.

The free-field ground motion is considered to be due to horizontally polarized, incoherent shear waves that propagate vertically and induce horizontal motions in the plane of the paper. The detailed histories of the motions generally vary from point to point.

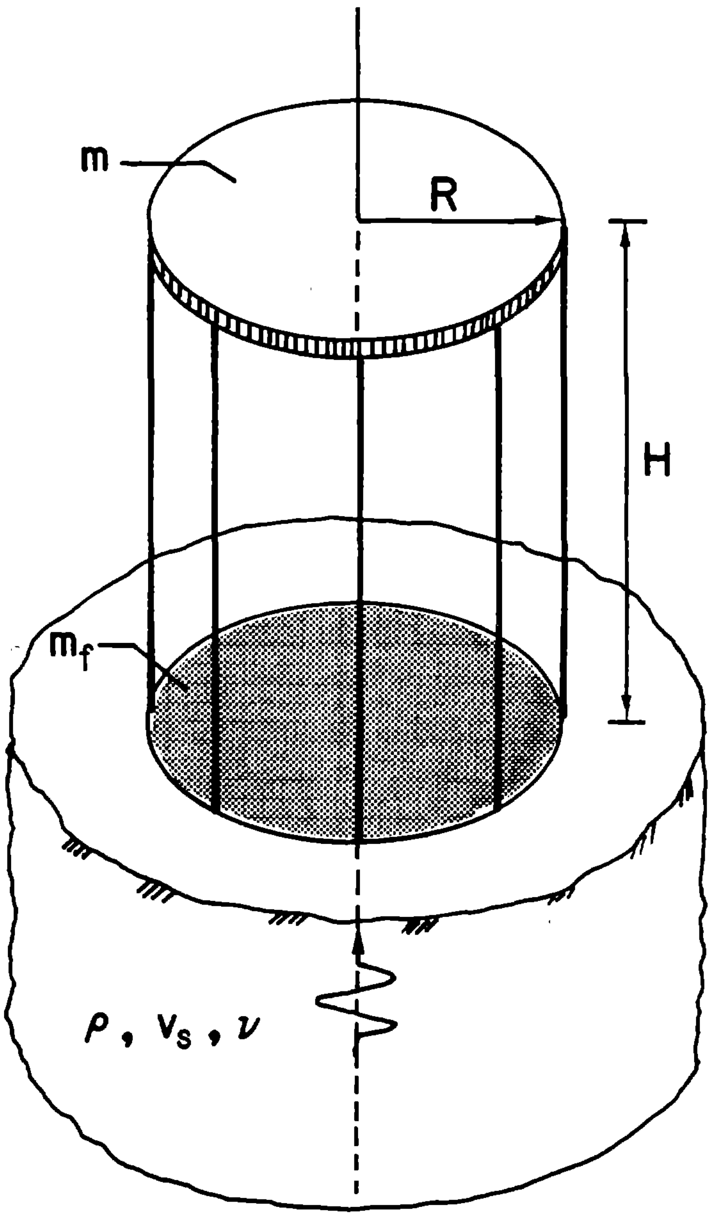

FIG. 2-1 System Considered

SECTION 3
STATEMENT OF PROBLEM

Let $\ddot{x}_g(\vec{r},t)$ be the value at any time, t, of the free-field acceleration at a point on the foundation-soil interface defined by the position vector $\vec{r}$, and let $\ddot{x}_i(t)$ and $\ddot{\theta}_i(t)$ be the horizontal and rotational components of the foundation input acceleration. The latter term refers to the acceleration that the foundation would experience if both it and the superimposed structure were massless. The foundation input motion is, therefore, independent of the properties of the superstructure. Let, further, the restraining action of the soil to the motion of the foundation be represented in the spirit of the approach used in previous analyses of wave passage effects (e.g., Bycroft 1980; Iguchi 1983; Scanlan 1976), by a series of isolated elastic springs. The foundation input accelerations are then given by

$$\ddot{x}_i(t) = \frac{1}{A} \int_A \ddot{x}_g(\vec{r},t)\, dA \tag{1}$$

$$\ddot{\theta}_i(t) = \frac{1}{I_\theta} \int_A d_n \ddot{x}_g(\vec{r},t)\, dA \tag{2}$$

in which dA = an elemental area of the foundation; A = its total area; I_θ = its polar moment area about a vertical centroidal axis; and d_n = the projection of $\vec{r}$ in a direction normal to the direction of ground motion.

The computation of $\ddot{x}_i(t)$ and $\ddot{\theta}_i(t)$ through Eqs. 1 and 2 presupposes that the spatial variation of the free-field ground motion may be defined in deterministic terms. This is too demanding a requirement which, at the present state of knowledge, cannot be realized in practice. The most meaningful characterization of the ground motion variation to date has been provided in stochastic terms, by means of a cross power spectral density function. Denoted by $S(\vec{r}_1,\vec{r}_2,\omega)$, this function defines the interrelationship of the amplitudes of the harmonic components of the motions at two points defined by the position vectors $\vec{r}_1$ and $\vec{r}_2$, and it is expressed in the form

$$S(\vec{r}_1, \vec{r}_2, \omega) = \Gamma(|\vec{r}_1 - \vec{r}_2|, \omega)\, S_g(\omega) \tag{3}$$

in which $S_g(\omega)$ = the power spectral density function of the free-field, control point motion; ω = the circular frequency of the motion; and Γ = the so-called incoherence function, which is a dimensionless, generally decreasing function of ω and of the distance between points, $|\vec{r}_1 - \vec{r}_2|$. It should be clear that, for $\vec{r}_1 = \vec{r}_2$ or $\omega = 0$, $\Gamma = 1$. For the solutions presented herein, the control point is taken at the ground surface beneath the foundation center.

3.1 Fundamental Background Information

It is desirable to review here the evaluation of the foundation input motion in a stochastic analysis of the problem. Let $S_x(\omega)$ = the psd function for the lateral component of foundation input motion, and $S_y(\omega)$ = the corresponding function for the circumferential component of motion along the perimeter of the foundation. These functions may be expressed in the form

$$S_x(\omega) = T_x^2(\omega)\, S_g(\omega) \tag{4a}$$

and

$$S_y(\omega) = T_y^2(\omega)\, S_g(\omega) \tag{4b}$$

in which $T_x(\omega)$ and $T_\theta(\omega)$ are transfer functions interrelating the amplitudes of the harmonics of the free-field control point motion and of the components of foundation input motion. More specifically, if $C_g(\omega)$ = the mean amplitude of the harmonic motions corresponding to $S_g(\omega)$, and $C_x(\omega)$ and $C_y(\omega)$ are the corresponding amplitudes for the horizontal and circumferential components of foundation input motion, then

$$C_x(\omega) = T_x(\omega)\, C_g(\omega) \tag{5a}$$

and

$$C_y(\omega) = T_y(\omega)\, C_g(\omega) \tag{5b}$$

On the assumption that the soil medium acts as a series of isolated springs, as previously indicated, the functions $T_x(\omega)$ and $T_y(\omega)$ are given by (see, for example, Veletsos and Prasad, 1989)

$$T_x^2(\omega) = \frac{1}{A^2} \int_A \int_A \Gamma(|\vec{r}_1 - \vec{r}_2|, \omega)\, dA_1\, dA_2 \tag{6}$$

and

$$T_y^2(\omega) = \frac{4}{A^2 R^2} \int_A \int_A d_1 d_2\, \Gamma(|\vec{r}_1 - \vec{r}_2|, \omega)\, dA_1\, dA_2 \tag{7}$$

in which dA_1 and dA_2 are elemental areas of the foundation, and d_1 and d_2 are the projections of $\vec{r}_1$ and $\vec{r}_2$ in a direction normal to the direction of the ground motion. Although defined specifically for displacement amplitudes, the functions T_x and T_y also interrelate the corresponding velocity and acceleration amplitudes.

Several different expressions have been recommended for the incoherence function (e.g., Harichandran and Vanmarcke 1986; Hoshiya and Ishii 1983; Kausel and Pais 1987; Loh 1985; Luco and Wong 1986; and Luco and Mita 1987), and there is no general agreement at this time on the form that may be the most appropriate for actual earthquakes. For the solutions presented here, the single-parameter, second-order function proposed by Luco and Mita (1987) is used

$$\Gamma(|\vec{r}_1 - \vec{r}_2|, \omega) = \exp\left[-\left(\frac{\gamma\omega|\vec{r}_1 - \vec{r}_2|}{v_s}\right)^2\right] \tag{8}$$

in which γ = a dimensionless factor with a value between zero and 0.5. The quantities T_x and T_θ in this case are functions of the single dimensionless parameter

$$b_o = \gamma a_o \tag{9}$$

in which

$$a_o = \frac{\omega R}{v_s} = \omega\tau \tag{10}$$

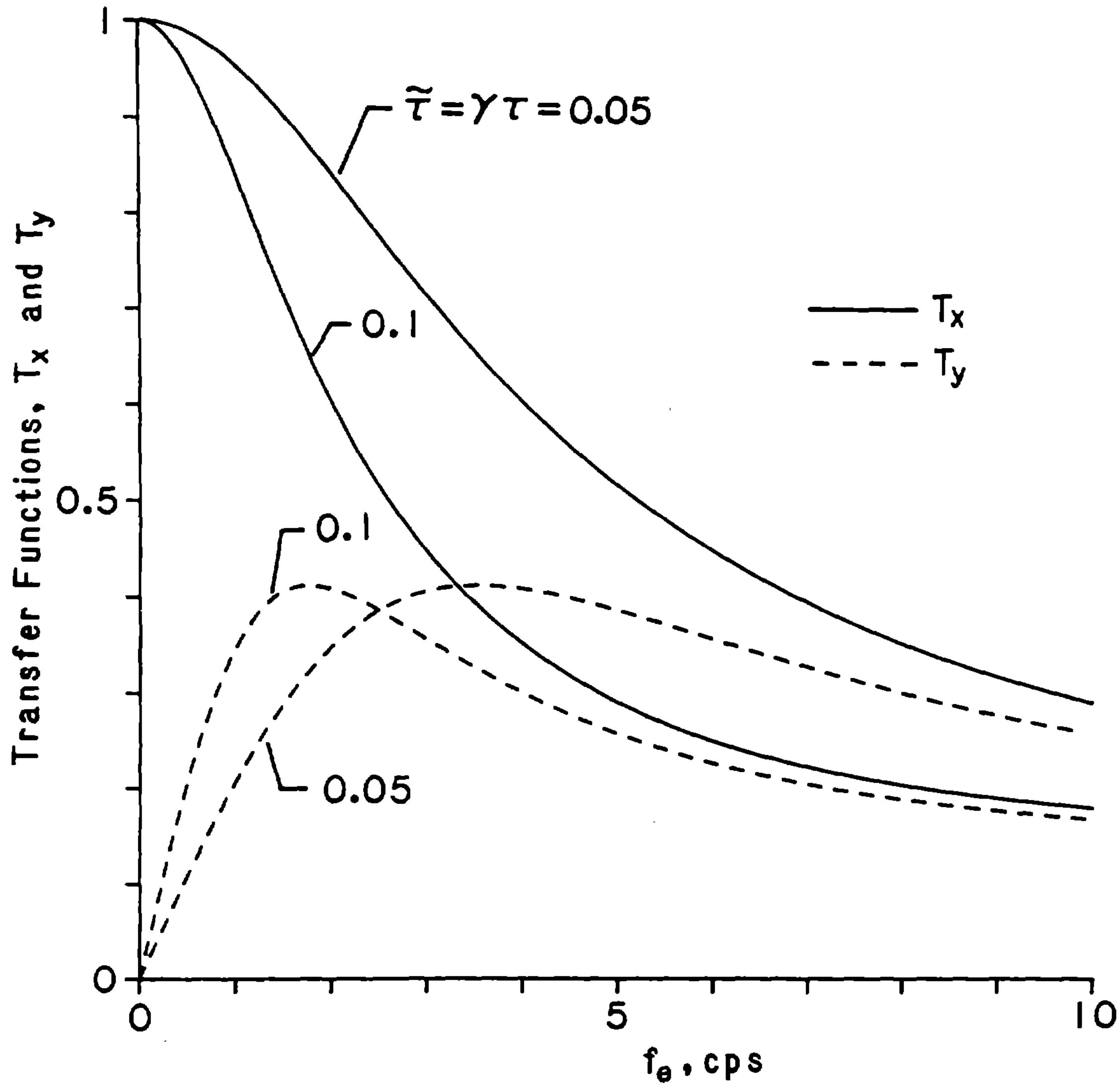

FIG. 3-1 Transfer Functions for Lateral and Circumferential Responses of Massless Foundations

is the well known frequency parameter encountered in analyses of vibrating foundations, and $\tau = R/v_s$ = the time required for the shear waves in the soil to traverse a distance equal to the radius of the foundation.

Closed-form expressions for T_x and T_θ for the incoherence function defined by Eq. 8 have been presented by Veletsos and Prasad (1989), and representative plots are given in Fig. 3-1 as a function of the cyclic value of the exciting frequency, $f_e = \omega/2\pi$, for two values of the effective wave transit time,

$$\tilde{\tau} = \gamma\tau \tag{11}$$

With the spatial variation of the ground motion defined stochastically by Eqs. 3 and 8, it is desired to evaluate the response of the structure-foundation-soil system to a deterministically specified free-field ground motion.

SECTION 4
PROPOSED METHOD

Let $\ddot{x}_g(t)$ be the acceleration history of the control point motion, and $\ddot{X}_g(\omega)$ be its Fourier transform. Further, let $\ddot{X}(\omega)$ be the Fourier transform of the lateral component of the resulting foundation input motion, and $\ddot{Y}(\omega)$ be the corresponding transform of the circumferential component of motion along the perimeter of the foundation. These transforms are, of course, complex-valued quantities that define both the amplitudes and phase angles of the harmonics in Fourier representations of the relevant motions.

The approach used is based on the premise that these quantities may be interrelated by expressions analogous to Eqs. 5, as

$$\ddot{X}(\omega) = T_x(\omega)\,\ddot{X}_g(\omega) \tag{12a}$$

$$\ddot{Y}(\omega) = T_y(\omega)\,\ddot{X}_g(\omega) \tag{12b}$$

Implicit in this approach is the assumption that only the amplitudes of the Fourier components of the free-field ground motion are modified by the presence of the foundation, their phase angles remaining unaltered. This is admittedly an approximation. Inasmuch as the spatial variation of the free-field motion employed is not completely defined, however, the precise change in phase angles cannot be determined. It is also worth noting that the histories of foundation input motion obtained by this approach are samples of possible realizations of the random process implied by the stochastic characterization of the ground motion incoherence, and that they are statistically admissible in the sense that they satisfy the amplitude relations defined by Eqs. 5. Furthermore, as either γ or ω tends to zero, it is physically apparent that there can be no change in the phase relationship of the harmonics for the free-field ground motion and the foundation input motion. These limiting conditions are satisfied by the assumed relationship. The same relation also is expected to hold true for the relatively small values of the modified frequency parameter, $b_o = \gamma a_o$, normally encountered in practice. Finally, the proposed approximation is deemed to be justified by the uncertainties that are currently involved in the specification of the incoherence function.

The ultimate test of the reasonableness of the proposed approach is, of course, the extent to which the results obtained by it agree with those obtained by formal application of the stochastic approach. It is shown in the following sections that the two sets of results are indeed in very good agreement for practical purposes.

The steps involved in the analysis of the system by the proposed procedure may now be summarized as follows:

1. Compute the complex-valued Fourier transform, $\ddot{X}_g(\omega)$, of the free-field, control point acceleration, $\ddot{x}_g(t)$. For a motion that is defined at discrete time intervals, this computation is naturally implemented by application of the Discrete Fourier Transform (DFT).

2. From Eqs. 12, compute the Fourier transforms of the horizontal and circumferential components of the foundation input acceleration, $\ddot{X}_i(\omega)$ and $\ddot{Y}_i(\omega)$.

3. If desired, the acceleration histories of the foundation input motion, $\ddot{x}_i(t)$ and $\ddot{y}_i(t)$, may then be determined by taking the inverse Fourier transforms of $\ddot{X}_i(\omega)$ and $\ddot{Y}_i(\omega)$.

With the foundation input accelerations and their Fourier transforms determined, the response of the structure may be computed by well established procedures either in the frequency domain, or more directly, in the time domain. The steps involved in these computations depend on whether the inertial interaction (II) effects are considered or not.

When the II effects are not considered, the actual motion of the foundation is the same as the foundation input motion, and the structure is analyzed for the latter motion considering it to be rigidly supported at the base. When the II effects are considered, the foundation motion is generally different from the foundation input motion and includes, in addition to lateral and torsional components, a rocking component about a horizontal centroidal axis normal to the direction of the free-field ground motion. The analysis in this case must make due provision for the flexibility of the supporting medium and for its capacity to dissipate energy by radiation of waves. In the frequency domain analysis, this is accomplished by use of complex-valued, frequency-dependent foundation impedances, whereas in a time domain analysis it is accomplished either by use of the foundation impulse response functions (Veletsos and Verbic 1973; Veletsos and Nair 1974b), or by use of the frequency-independent foundation models of Meek and Veletsos (1974) and Veletsos and Nair (1974a), as indicated by Wolf and Somaini (1986).

The solutions presented herein were obtained in the frequency domain by application of DFT techniques, taking due precautions to ensure that the aliasing error involved in their application was negligibly small. The foundation impedances for the horizontal and rocking responses were computed from the approximate closed-form expressions of Veletsos and Verbic (1973), and those for the torsional response were computed from the corresponding expressions of Veletsos and Nair (1974a). The cross-coupling terms between horizontal and rocking actions were considered to be negligible. The details of analysis have already been described (e.g., Veletsos and Meek 1974; and Veletsos and Prasad 1988) and need not be repeated here. It may simply be noted that the relatively simple system examined here has three degrees of freedom for lateral response, and two degrees of freedom for circumferential or torsional response.

SECTION 5
PARAMETRIC STUDIES

As an illustration of the application of the procedure and of the effects of the various parameters affecting the response, solutions are presented for two free-field control point motions: (1) A relatively simple motion, for which the acceleration history consists of a sequence of three triangular pulses as shown in part (a) of Fig. 5-1; and (2) the first 6.24 sec. of the NS component of the 1940 El Centro, California earthquake record, shown in part (b) of the figure. Also shown in this figure are the associated velocity and displacement histories. The abscissa of the plots for the simpler input is normalized with respect to the duration of each velocity half-cycle, t_o. The spatial variation of the ground motion in both instances is defined by the incoherence function given in Eq. 8.

5.1 Foundation Input Motion

Fig. 5-2 shows the histories of the horizontal and circumferential components of the foundation input motion induced by the simple excitation, and Fig.5-3 shows the corresponding histories for the El Centro record. The displacement histories, $x_i(t)$ and $y_i(t)$, are normalized with respect to x_g, the maximum value of the free-field, control point displacement for the particular ground motion under consideration, and the velocity and acceleration histories are normalized with respect to the corresponding velocity and acceleration values, $\dot{x}_g$ and $\ddot{x}_g$. The solutions for the El Centro record are given for fixed values of the effective wave transit time parameter, $\tilde{\tau} = \gamma\tau$, whereas those for the simpler input are given for fixed values of $\tilde{\tau}/t_o$. A value of $\tilde{\tau} = 0$ corresponds to a fully coherent free-field motion. The following trends are observed in these plots:

1. The histories for the lateral component of foundation input motion are similar to those of the free-field ground motion, but their peak ordinates decrease with increasing value of $\tilde{\tau}$.

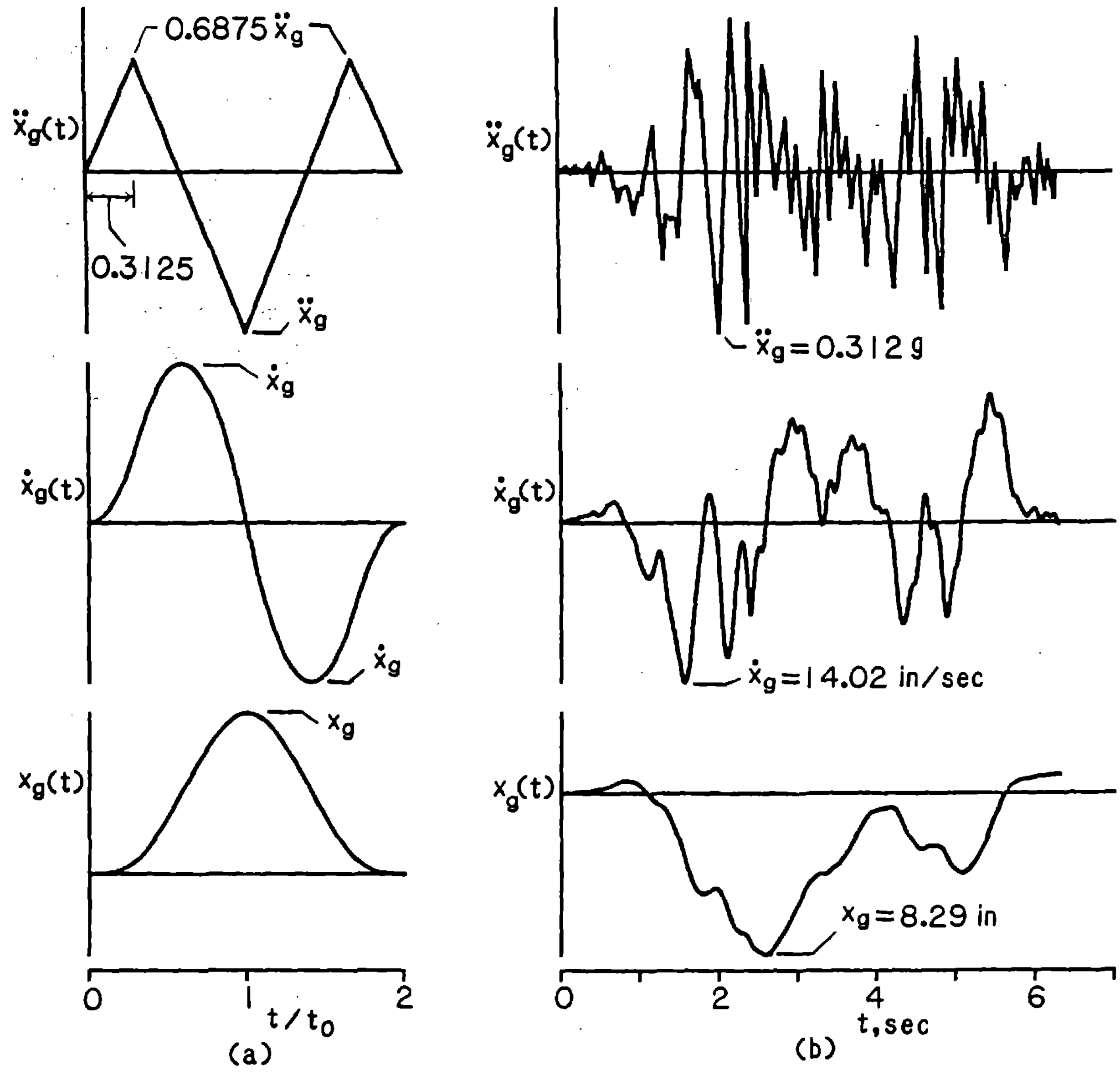

FIG. 5-1 Free-Field Control Point Motions Considered: (a) Simple Motion, (b) El Centro Record

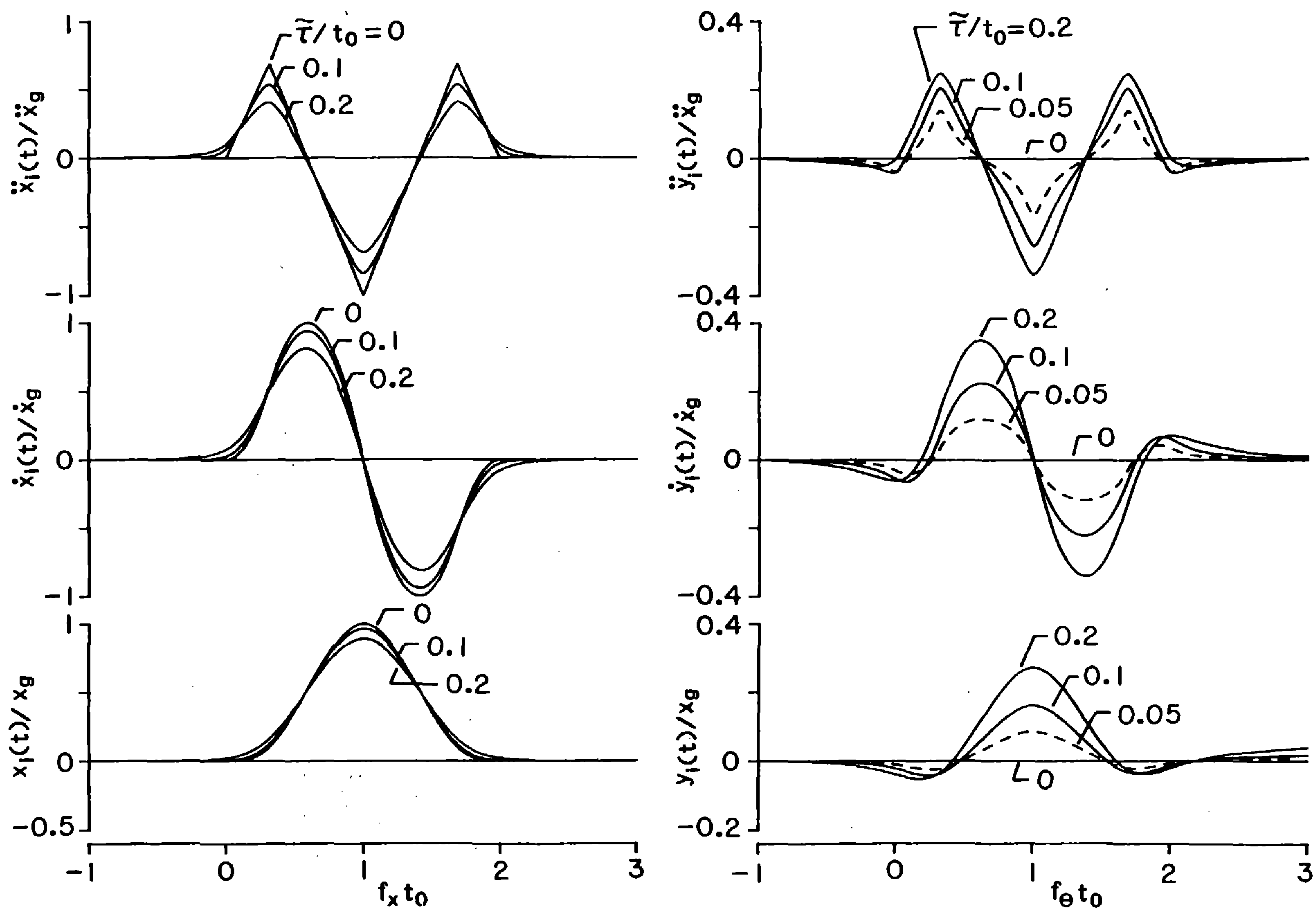

FIG. 5-2 Histories of Lateral and Circumferential Components of Foundation Input Motion for Systems Subjected to Simple Free-Field Ground Motion

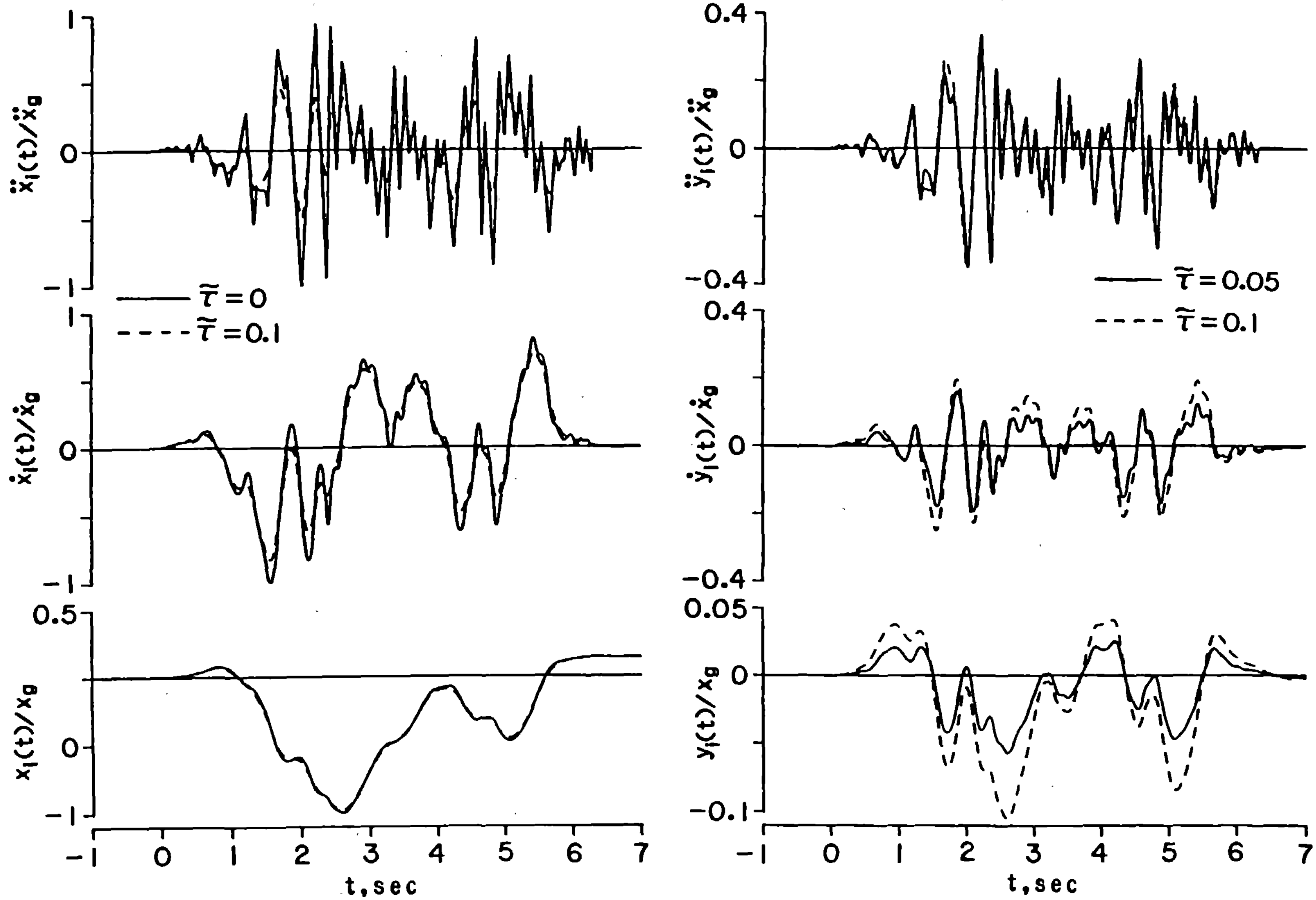

FIG. 5-3 Histories of Lateral and Circumferential Components of Foundation Input Motion for Systems Subjected to El Centro Free-Field Ground Motion

2. A substantial torsional component of foundation motion is induced, the peak value of which generally increases with increasing $\tilde{\tau}$.

3. Because different points of the foundation-soil interface are excited differently, both components of foundation input motion start prior to the start of, and terminate after the end of, the control point motion.

4. The reduction with increasing $\tilde{\tau}$ in the absolute maximum value of the horizontal component of foundation input motion, and the corresponding increase in the circumferential component, are greatest for acceleration, much smaller for velocity, and almost negligible for displacement.

Table 5-1 lists the peak values of response for both components of foundation input motion and both forms of excitation. Since the foundation filters the high-frequency wave components more effectively than the low-frequency components, the acceleration traces of the ground motion, which are richer in high-frequency content than the velocity and displacement traces, are influenced more than the latter traces.

Considering that the responses of high-frequency systems are acceleration sensitive, whereas those of low-frequency systems are displacement-sensitive, it should be clear that the effects of kinematic interaction would be important for high-frequency systems and inconsequential for low-frequency systems. Furthermore, medium-frequency systems, which are velocity-sensitive, would be expected to be affected moderately. That this is indeed the case is confirmed by the data presented in the following sections.

5.2 Kinematic Interaction Effects

This term refers to the difference in the responses of the structure computed for the free-field, control point motion and the foundation input motion. Let U_x = the maximum value of the structural deformation induced by the lateral component of foundation input motion, and U_y = the corresponding deformation induced along the peripheries of the deck and

TABLE 5-1 Normalized Values of Horizontal and Circumferential Components of Foundation Input Motions for Two Control Point Motions Considered

$\frac{\tilde{\tau}}{t_o}$	$\tilde{\tau}$ sec	$\frac{x_i}{x_g}$	$\frac{\dot{x}_i}{\dot{x}_g}$	$\frac{\ddot{x}_i}{\ddot{x}_g}$	$\frac{y_i}{x_g}$	$\frac{\dot{y}_i}{\dot{x}_g}$	$\frac{\ddot{y}_i}{\ddot{x}_g}$
		For Simple Motion					
0		1	1	1	0	0	0
0.05		0.990	0.985	0.921	0.086	0.120	0.167
0.10		0.965	0.941	0.840	0.162	0.226	0.256
0.20		0.891	0.813	0.688	0.273	0.351	0.339
		For El Centro Record					
	0	1	1	1	0	0	0
	0.02	0.998	0.986	0.889	0.024	0.106	0.336
	0.05	0.992	0.933	0.717	0.057	0.196	0.341
	0.10	0.977	0.832	0.501	0.105	0.251	0.358

foundation by the torsional component. For a specified control point motion, these deformations depend on the natural frequencies of the structure, f_x and f_θ, the associated damping factors, ζ_x and ζ_θ, and the effective wave transit time parameter, $\tilde{\tau}$. The results for the two free-field ground motions examined here are displayed in Figs. 5-4 and 5-5 in the form of tripartite logarithmic response spectra. The plots at the top refer to the lateral response, and those at the bottom refer to the circumferential or torsional response. Several values of $\tilde{\tau}$ are considered, including the limiting of value $\tilde{\tau} = 0$ for which there is no kinematic interaction. The damping factors for both modes of response in these solutions are taken as $\zeta_x = \zeta_\theta = 0.02$.

The left-hand diagonal scales for the top plots in Figs. 5-4 and 5-5 represents U_x normalized with respect to x_g; the vertical scale represents the corresponding pseudovelocity, $V_x = p_x U_x$, normalized with respect to $\dot{x}_g$; and the right-hand diagonal scale represents the corresponding pseudoacceleration, $A_x = p_x V_x = p_x^2 U_x$, normalized with respect to $\ddot{x}_g$. In an analogous manner, the three scales in the lower parts of these figures represent the deformation ratio U_y/x_g; the pseudovelocity ratio $V_y/\dot{x}_g$, in which $V_y = p_\theta U_y$; and the pseudoacceleration ratio, $A_y/\ddot{x}_g$, in which $A_y = p_\theta V_y = p_\theta^2 U_y$.

As anticipated from examination of the peak values of the foundation input motions, the lateral components of the responses of high-frequency systems in Figs. 5-4 and 5-5 are reduced significantly by ground incoherence. The reductions are materially less pronounced for medium-frequency systems, and very small to negligible for low-frequency systems. For very high-frequency systems, for which A_x may be considered to be equal to the peak value of the horizontal component of input acceleration, the percentage reductions are, of course, identical to those indicated in Figs. 5-2 and 5-3 for the foundation input accelerations.

The general trends of the response spectra for the circumferential components of deformation in Figs. 5-4 and 5-5 are consistent with those which would be expected from the corresponding histories of foundation input motion. In particular, their low-frequency and high-frequency

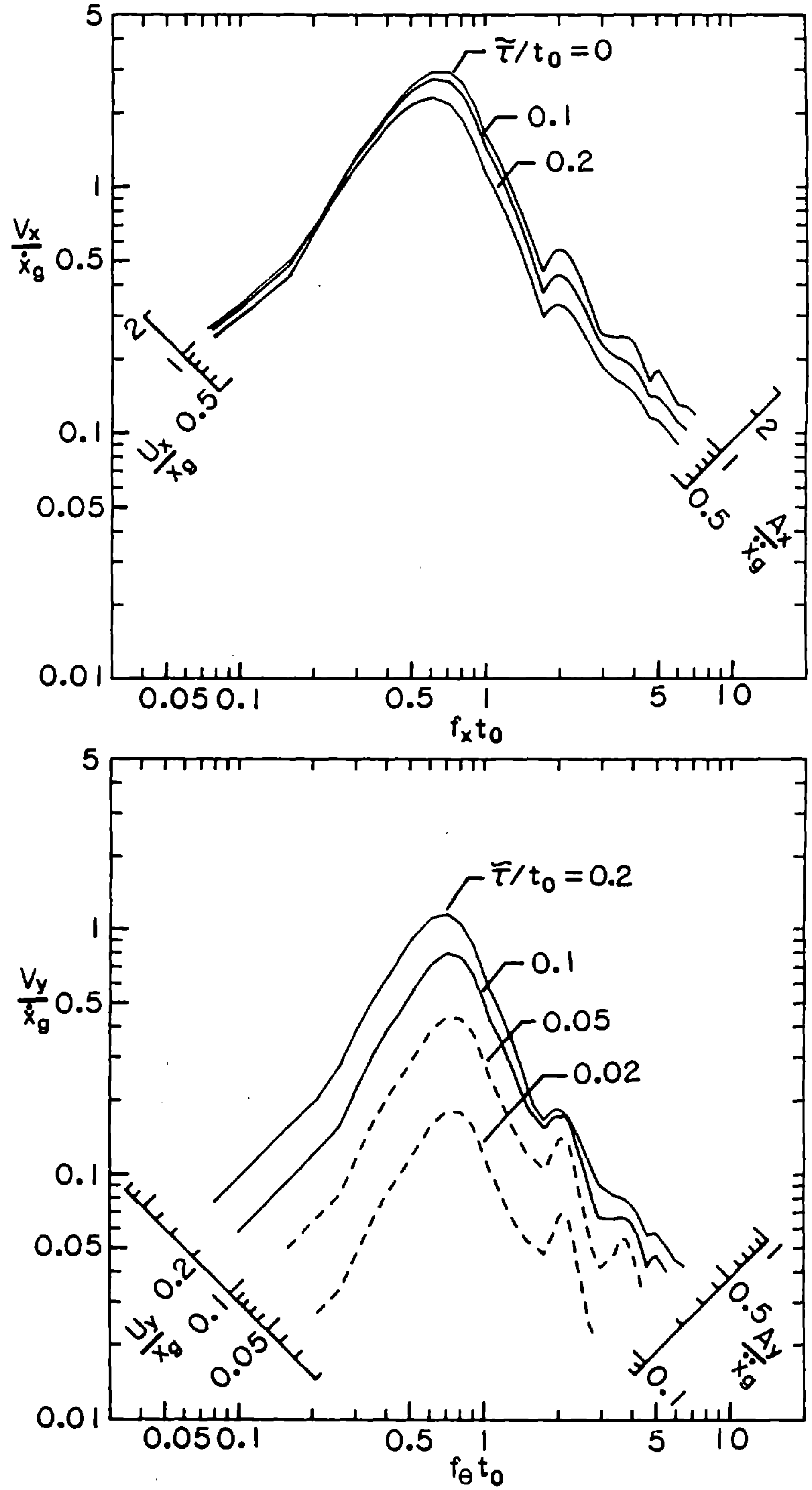

FIG. 5-4 Response Spectra for Lateral and Circumferential Deformations of Systems with $\zeta_x = \zeta_\theta = 0.02$ Subjected to Simple Free-Field Ground Motion; No Inertial Interaction

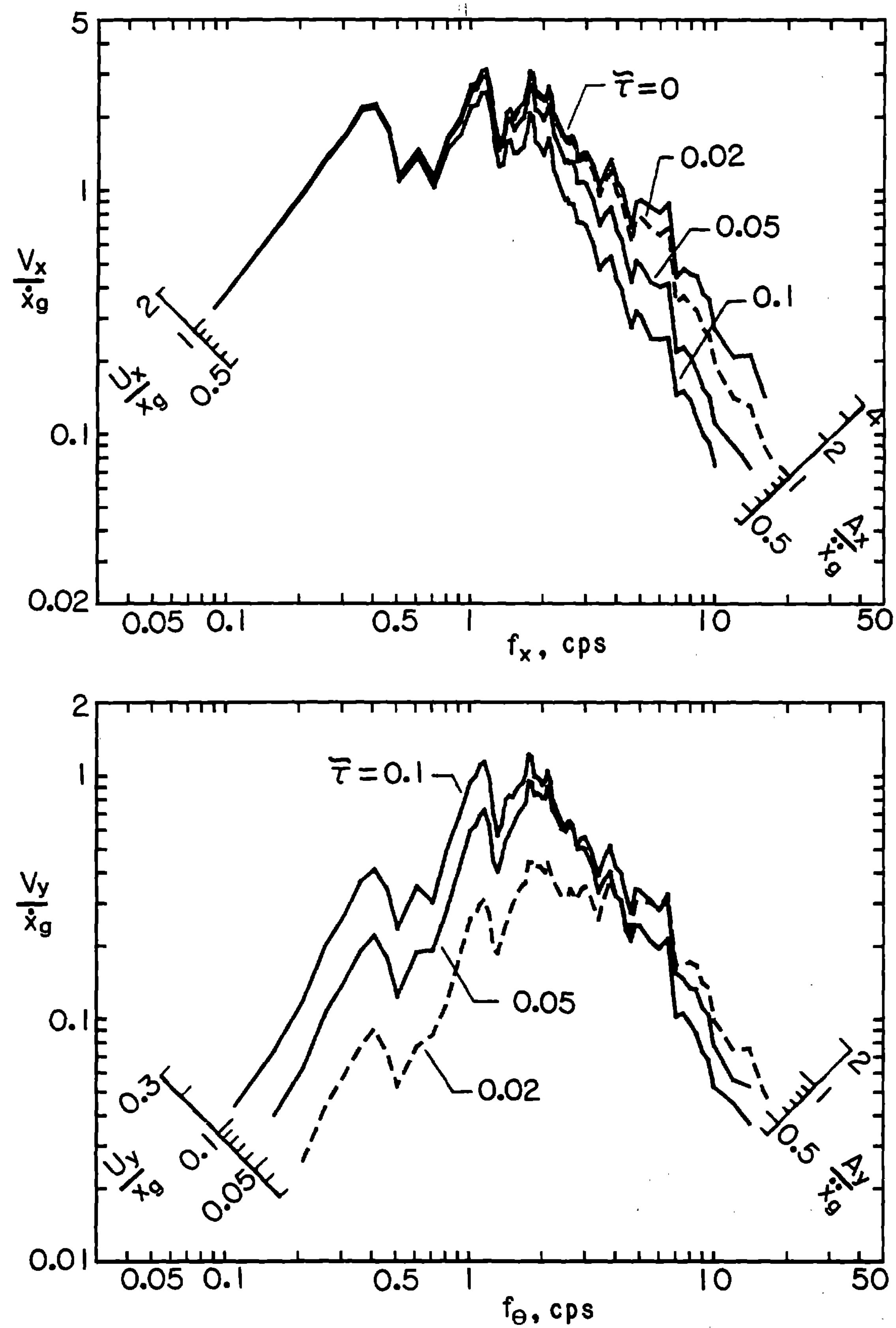

FIG. 5-5 Response Spectra for Lateral and Circumferential Structural Deformations of Systems with $\zeta_x = \zeta_\theta = 0.02$ Subjected to El Centro Free-Field Ground Motion; No Inertial Interaction

limits are equal to the peak values of the displacement and acceleration histories of the input motion, respectively.

The circumferential component of structural response is generally substantially smaller than the lateral component, and the peak value of the combination of the two components is normally only slightly greater than that of the lateral component. The peak value of the combination should be evaluated by the square-root of the sum of the squares rule.

5.3 Total Effects of Interaction

When both the kinematic and inertial interaction effects are considered, the response of the system depends not only on the parameters identified in the preceding section, but also on the ratio of the height to foundation radius for the structure, H/R, and the mass ratio $\delta = m/(\rho\pi R^2 H)$, in which m = the mass of the structure, and the denominator represents the total mass of the structure when filled with the supporting soil. Other parameters affecting the response of the system are Poisson's ratio for the supporting medium, ν; the mass ratio for the foundation and structure, m_f/m; the ratio I_f/I of the mass moments of inertia of the foundation and structure about horizontal centroidal axes; and the ratio J_f/J of the corresponding polar moments of inertia about vertical centroidal axes. For the solutions presented herein, $\zeta_x = \zeta_\theta = 0.02$, $\delta = 0.15$, $\nu = 1/3$, and m_f (and hence I_f and J_f) are considered to be negligible. It should be noted that, whereas the kinematic interaction effects are defined completely by the effective transit time, $\tilde{\tau}$, the evaluation of the inertial interaction effects requires the separate specification of the parameters γ and τ.

Fig. 5-6 displays the response spectra for the lateral and circumferential deformations induced by the El Centro record in systems with the values of H/R, γ and τ identified on the figure. Three sets of solutions are presented: (1) making no provision for soil-structure interaction (SSI), i.e., considering the foundation motion to be equal to the free-field control point motion; (2) providing only for the kinematic inter-

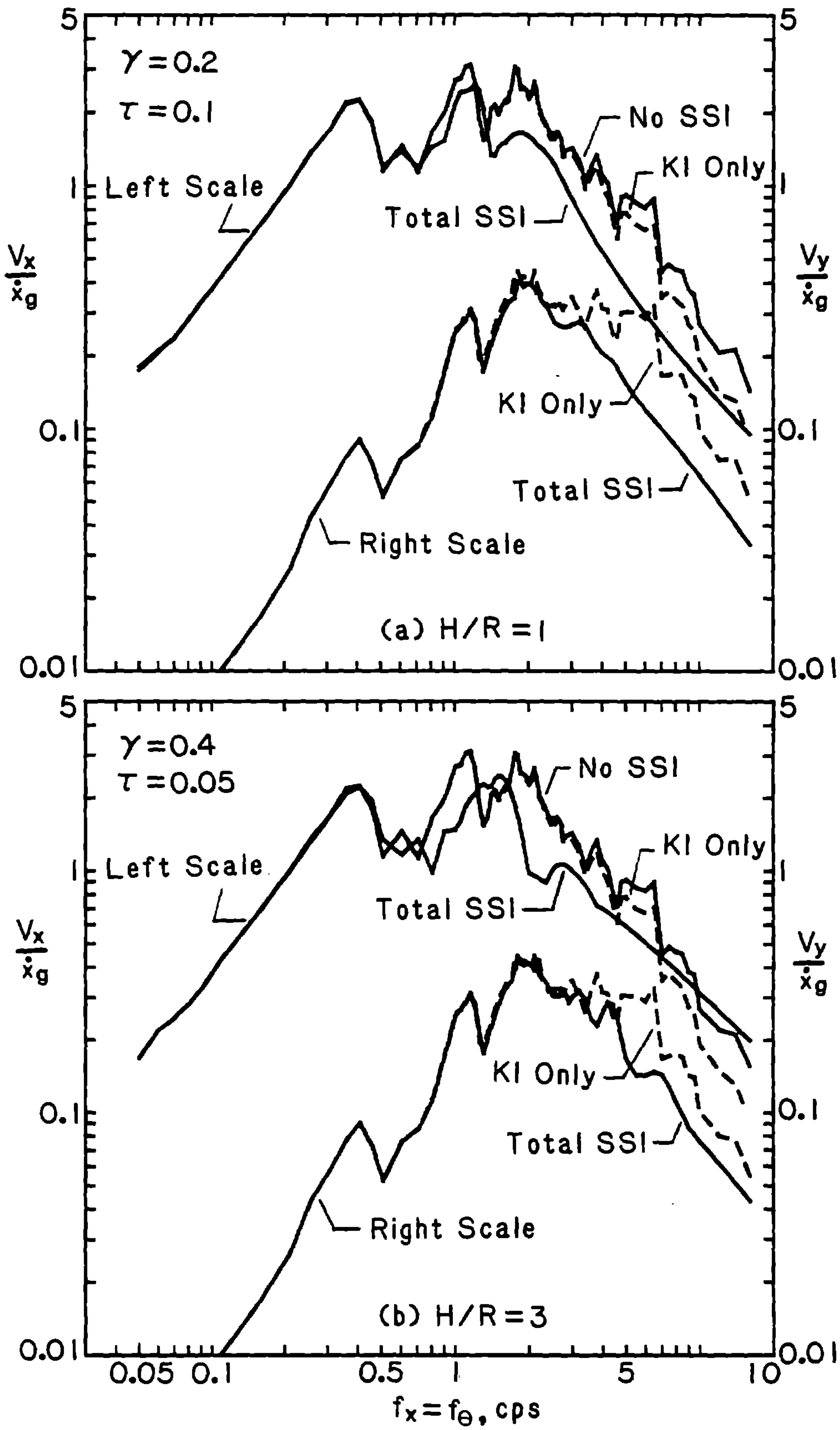

FIG. 5-6 Response Spectra for Lateral and Circumferential Deformations for Systems Without and With Inertial Interaction; Systems With $\zeta_x = \zeta_\theta = 0.02$ Subjected to El Centro Free-Field Ground Motion

action effects, i.e., taking as base excitation the foundation input motion; and (3) providing for both kinematic and inertial interaction effects, i.e., analyzing the structure-foundation-soil system exactly as a coupled system. Solutions (1) are independent of H/R, γ and τ, and solutions (2) are valid for all combinations of γ and τ for which $\tilde{\tau} = \gamma\tau = 0.02$ sec. The following trends may be observed in this figure:

1. Like kinematic interaction (KI), inertial interaction (II) may affect significantly the responses of systems in the medium- and high-frequency spectral regions.

2. The II effects are generally considerably more important than the KI effects.

3. Unlike kinematic interaction which generally decreases the lateral response, inertial interaction may increase the response of tall, slender systems in the very high-frequency spectral region.

4. The II effects for low-frequency, highly compliant structures are negligible because such systems "see" the half-space as a very stiff, effectively rigid medium.

These trends, as well as those enumerated earlier for the foundation input motion and the KI effects, are in excellent agreement with those obtained in a companion recent study by Veletsos and Prasad (1989) by formal application of the stochastic approach. This agreement effectively confirms the reliability of the proposed method of analysis.

SECTION 6
CONCLUSION

An approximate procedure has been presented with which the effects of ground motion incoherence on the seismic response of surface-supported structure-foundation-soil systems may be evaluated deterministically. The free-field ground motion in this approach is specified by the acceleration history of the motion at a reference or control point, and by a spatial incoherence function. Information and concepts have been presented which elucidate the nature of kinematic and inertial interaction, and the relative importance of the two effects. The results obtained by the proposed procedure have been shown to be consistent with those obtained in a companion recent study by formal application of the stochastic approach.

SECTION 7
NOTATION

The following symbols are used in this report:

a_o = $\omega R/v_s$ = frequency parameter;

A = foundation contact area;

A_x = $p_x^2 U_x$ = pseudoacceleration corresponding to U_x;

A_y = $p_\theta^2 U_y$ = pseudoacceleration corresponding to U_y;

b_o = γa_o = modified frequency parameter incorporating effect of ground motion incoherence;

$C_g(\omega)$ = mean amplitude of the harmonics in a Fourier representation of the free-field control point motion;

$C_x(\omega)$, $C_y(\omega)$ = mean amplitudes of the harmonics in Fourier representations of the horizontal and circumferential components of foundation input motion;

d_1, d_2 = components of $\vec{r}_1$ and $\vec{r}_2$ normal to the direction of free-field ground shaking;

f_e = $\omega/2\pi$ = exciting frequency, in cps;

f_x = fixed-base natural frequency of structure for lateral mode of vibration, in cps;

f_θ = fixed-base natural frequency of structure in torsional mode of vibration, in cps;

H = height of structure;

I, I_f = mass moments of inertia of structure and foundation about horizontal centroidal axes;

I_θ = polar area moment of inertia of foundation about vertical centroidal axis;

m, m_f = mass of structure and foundation, respectively;

p_x = $2\pi f_x$ = fixed-base circular natural frequency of structure in lateral mode of vibration;

p_θ = $2\pi f_\theta$ = fixed-base circular natural frequency of structure in torsional mode of vibration;

psd = power spectral density;

$\vec{r}_1$, $\vec{r}_2$ = position vectors for two arbitrary points on foundation-soil interface;

R = radius of foundation;

$S(\vec{r}_1,\vec{r}_2,\omega)$ = cross psd function for motions at points $\vec{r}_1$ and $\vec{r}_2$;

$S_g(\omega)$ = local psd function for displacement histories of free-field ground motion;

$S_x(\omega)$, $S_y(\omega)$ = psd functions for displacement histories of horizontal and circumferential components of foundation input motion;

t_o = duration of velocity half-cycle pulse for simple free-field ground motion considered;

T_x, T_y = dimensionless transfer functions relating the lateral and circumferential components of the foundation input motion to those of the free-field control point motion;

$U(\omega)$ = psd function of structural deformation induced by the lateral component of foundation input motion;

U_x = maximum structural deformation induced by the lateral component of foundation input motion;

U_y = maximum circumferential deformation along the perimeter of the structure induced by the torsional component of foundation input motion;

v_s = velocity of shear wave propagation in soil medium;

V_x = $p_x U_x$ = pseudovelocity value corresponding to U_x;

V_y = $p_\theta U_y$ = pseudovelocity value corresponding to U_y;

x = lateral component of actual foundation displacement;

x_i = lateral component of foundation input displacement;

$\ddot{X}_g$ = Fourier transform of free-field ground acceleration at control point;

$\ddot{X}_i$ = Fourier transform of lateral component of foundation input acceleration;

$\ddot{Y}_i$ = Fourier transform of circumferential component of input acceleration along the perimeter of the foundation;

γ = dimensionless incoherence parameter;

δ = mass density ratio for structure;

ζ_x, ζ_θ = percentages of critical damping for structure in lateral and torsional modes of vibration;

ν = Poisson's ratio for soil medium;

ρ = mass density of soil medium;

$\tau = R/v_s$ = transit time;

$\tilde{\tau} = \gamma\tau$ = effective transit time;

ω = circular frequency of excitation and of resulting motion.

SECTION 8

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An approximate deterministic method of analysis is presented for assessing the effects of ground motion incoherence and of the associated soil-structure interaction on the seismic response of structure-foundation-soil systems. The free-field ground motion in this approach is specified by an acceleration history and a spatial incoherence function. Numerical solutions are presented which illustrate the procedure and elucidate the nature and relative importance of the kinematic and inertial interaction effects. The results are shown to be consistent with those obtained in a companion recent study by formal application of the stochastic approach.

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