NATIONAL CENTER FOR EARTHQUAKE ENGINEERING RESEARCH

State University of New York at Buffalo

SEISMIC INTERACTION OF STRUCTURES AND SOILS: STOCHASTIC APPROACH

by

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PREFACE

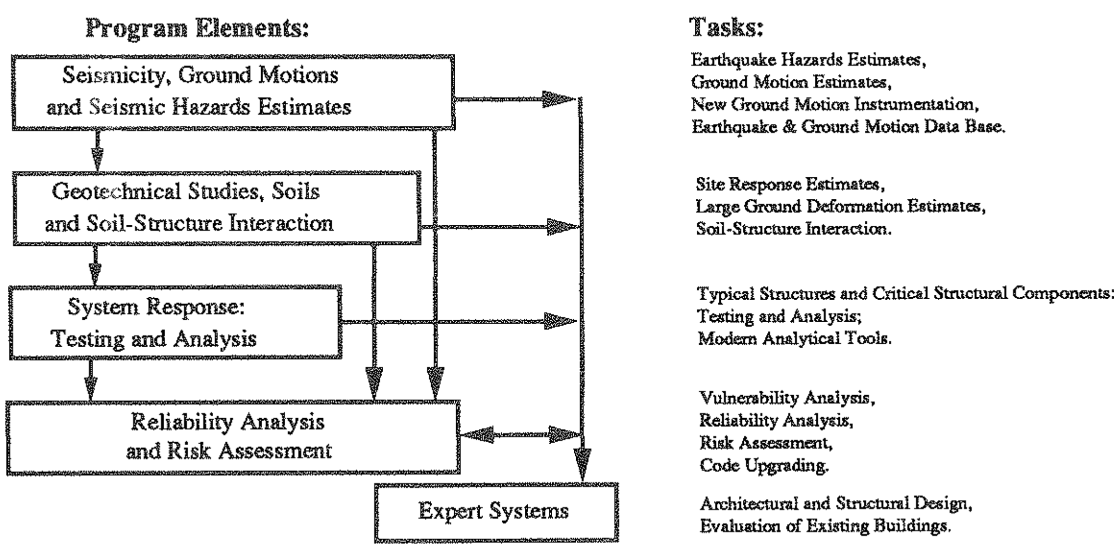
The National Center for Earthquake Engineering Research (NCEER) is devoted to the expansion of knowledge about earthquakes, the improvement of earthquake-resistant design, and the implementation of seismic hazard mitigation procedures to minimize loss of lives and property. The emphasis is on structures 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.

This report presents the result of a study of soil-structure interaction for seismically excited simple structures considering both kinematic and inertial interaction effects. The information and concepts presented elucidate the nature and relative importance of the two effects and make it possible to assess readily the influences of the more important parameters. The response quantities examined are the ensemble means of the peak values of the lateral and torsional components of the foundation input motion and of the associated structural deformations. The results are evaluated over wide ranges of the parameters involved and compared with those obtained for no soil-structure interaction and for kinematic interaction only. Simple, physically motivated interpretations are given for the observed differences. For the important special case of vertically incident incoherent waves, simple closed-form approximate expressions are presented for the transfer functions of circular massless foundations.

ABSTRACT

A study of soil-structure interaction for seismically excited simple structures is made considering both kinematic and inertial interaction effects. The information and concepts presented elucidate the nature and relative importance of the two effects and make it possible to assess readily the influences of the more important parameters. The response quantities examined are the ensemble means of the peak values of the lateral and torsional components of the foundation input motion and of the associated structural deformations. The results are evaluated over wide ranges of the parameters involved and compared with those obtained for no soil-structure interaction and for kinematic interaction only. Simple, physically motivated interpretations are given for the observed differences. For the important special case of vertically incident incoherent waves, simple closed-form approximate expressions are presented for the transfer functions of circular massless foundations.

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SECTION 1
INTRODUCTION

In evaluating the response of structures to earthquakes, it is normally assumed that all points of the ground surface beneath the foundation are excited synchronously and experience the same free-field motion [3,6,26]; the latter term refers to the motion which would be induced at the foundation-soil interface if no structure were present. The assumption of synchronous interface free-field ground motions is strictly valid only for vertically propagating coherent wave fields; in reality, the motions may vary from one point to the next [1,8,13,34]. Even when the wave front is plane and propagates in a perfectly homogeneous medium, it may impinge the foundation at a finite angle, leading to motions at neighboring points which in the words of Kausel and Pais [11] are "delayed replicas" of each other. Known as the **wave passage effect**, the consequences of such action have been the subject of numerous previous studies [4,14,19,20,23,24,32,33] and are reasonably well understood.

Several additional factors contribute to the spatial variability of the free-field ground motion. The individual wave trains may emanate from different points of an extended source and may impinge the foundation at different instants and 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. The spatial variability of the ground motion due to these factors will be referred to as the **ground motion incoherence effect**. This effect, which would exist even for horizontally polarized vertically propagating shear waves, has been the subject of only exploratory recent studies [9,15,16,17,18,21,22].

The motion experienced by a rigid foundation is clearly different from the free-field ground motion. The actual motion may conveniently be evaluated in two steps. First, the so-called **foundation input motion** is computed; this is defined as the motion which would be experienced by the foundation if both it and the superimposed structure were massless.

Computed with due provision for the rigidity of the foundation, the foundation input motion includes both horizontal and torsional components even for a purely horizontal free-field ground shaking. The difference in the responses of the structure computed for the foundation input motion and the free-field motion at some reference or control point of the ground surface is known as the **kinematic interaction effect**. The greater the degree of ground motion incoherence or the plan dimensions of the foundation in comparison to the length of the dominant seismic waves, the more important this effect is likely to be.

The actual motion of the foundation is also influenced by its own inertia and the inertia of the structure, and by the interaction or coupling between them and the supporting soils. For a structure subjected to a purely horizontal free-field ground shaking, not only are the horizontal and torsional components of the actual foundation motion different from those of the corresponding input motion, but the actual motion may also include rocking components about horizontal axes. Contributed by the overturning tendency of the superstructure, the latter components may be particularly prominent for tall slender structures and for soft soils. These factors are provided for in the second step of the evaluation process.

The term **inertial interaction effect** refers to the difference in structural responses computed for the actual motion of the foundation and the foundation input motion. The total **soil structure interaction** is clearly the sum of the kinematic and inertial interaction effects.

Although the inertial interaction effects have been the subject of numerous studies [6,23,26,27,28], they have generally been examined at the exclusion of the kinematic interaction effects, and the interrelationship of the two effects has not been adequately assessed. The objectives of this paper are: to elucidate the nature of both types of interaction for seismically excited simple structures; to assess the effects and relative importance of the numerous parameters involved; and to present information and concepts with which the effects of the principal parameters may be evaluated readily. Primary emphasis is placed on the kinematic interaction effects.

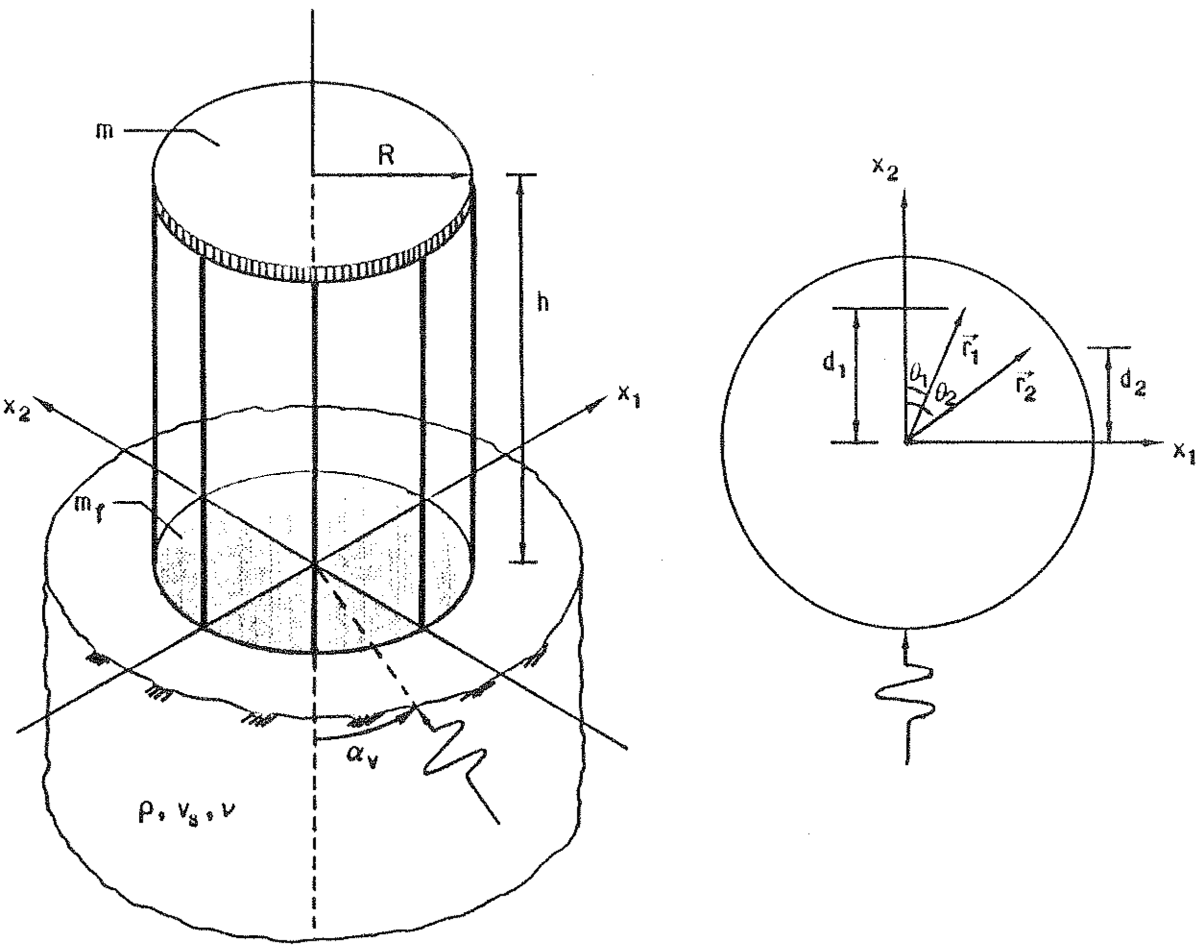
The structures investigated are presumed to have one lateral and one torsional degree of freedom in their fixed-base condition and to be excited by obliquely incident, horizontally polarized, incoherent shear waves. The temporal variation of the free-field ground motion is expressed stochastically by a local power spectral density (psd) function, and its spatial variability is specified by a cross psd function. The response quantities examined include the ensemble means of the peak values of the lateral and torsional components of the foundation input motion and of the corresponding structural deformations. These deformations are displayed in the form of pseudo-velocity response spectra and 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.

A fundamental step in the analysis of the a structure-foundation-soil system is the evaluation of the transfer functions of the foundation. Defined for harmonically excited massless foundations, these functions relate the amplitudes of the horizontal and torsional components of foundation input motion to the amplitude of the free-field ground motion. The relevant functions are evaluated herein by a relatively simple, approximate procedure, and their accuracy is assessed through comparisons with available exact solutions for special cases. In addition, simple closed-form expressions are presented for these functions for the important special case of vertically incident, incoherent waves.

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 of lateral and torsional modes of vibration for 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 unit of time; 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. This 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 for all points of the foundation-soil interface is considered to be a uni-directional excitation directed parallel to the horizontal x_1-axis, as shown in Fig. 2-1, with the detailed histories of the motions varying from point to point. Such motions may be induced by horizontally polarized, incoherent shear waves propagating either vertically or at an arbitrary angle with the vertical, α_v. The intense portions of the motions are represented by a stationary random process of limited duration, t_o, and a space-invariant, local psd function, $S_g = S_g(\omega)$, in which ω = the circular frequency of the motions. The spatial variability of the motions is defined by a cross psd function, $S(\vec{r}_1, \vec{r}_2, \omega)$, in which $\vec{r}_1$ and $\vec{r}_2$ are the position vectors for two arbitrary points.



(a) Three Dimensional View

(b) Top View of Foundation

FIG. 2-1 System Considered

A decreasing function of the frequency ω and of the distance between the two points, $|\vec{r}_1 - \vec{r}_2|$, the function $S(\vec{r}_1,\vec{r}_2,\omega)$ is taken in the form suggested by Harichandran and Vanmarcke [7] as

$$S(\vec{r}_1,\vec{r}_2,\omega) = \Gamma(|\vec{r}_1 - \vec{r}_2|, \omega) \exp[-i\omega \frac{d_1 - d_2}{c}] S_g(\omega) \tag{1}$$

in which Γ, referred to as the incoherence function, is a dimensionless, decreasing function of $|\vec{r}_1 - \vec{r}_2|$; $i = \sqrt{-1}$; d_1 and d_2 = the components of $\vec{r}_1$ and $\vec{r}_2$ in the direction of propagation of the wave front (see Fig. 2-1b); and c = the apparent horizontal velocity of the front. The latter quantity is related to the angle of incidence of the waves, α_v, by

$$c = \frac{v_s}{\sin\alpha_v} \tag{2}$$

The product of the exponential term in Eq. 1 and S_g represents the wave passage effect, whereas the product ΓS_g represents the effect of ground motion incoherence. The peak value of Γ is unity and occurs at $\vec{r}_1 = \vec{r}_2$.

Several different expressions have been suggested for the incoherence function (e.g., Refs. 8,9,13,16,18), and there is no general agreement at this time on the form that may be the most appropriate for realistic earthquakes. In this study, the single-parameter, second order function recommended by Mita and Luco [18] is used,

$$\Gamma(|\vec{r}_1 - \vec{r}_2|, \omega) = \exp\left[-\left(\frac{\gamma\omega|\vec{r}_1 - \vec{r}_2|}{v_s}\right)^2\right] \tag{3}$$

in which γ is a dimensionless factor, taken between zero and 0.5.

A different approach to the study of this problem has been taken by Pais and Kausel [22]. They have attributed the ground motion incoherence to arrays of uncorrelated, obliquely incident waves arriving from different directions within a sector of the supporting medium. The kinematic interaction effects in this approach are represented by weighted averages of the component wave passage effects.

SECTION 3
KINEMATIC INTERACTION EFFECTS

3.1 Spectral Characterization of Foundation Input Motion

Let S_x be the psd function of the horizontal component of the foundation input displacement, and S_y be the corresponding function for the circumferential or tangential displacement component along the periphery of the foundation. Further, let S_{xy} for the cross spectral density function for the component displacements. Whereas S_x and S_y are real-valued, S_{xy} is generally complex-valued.

These functions were evaluated from the cross spectral density function, $S(\vec{r}_1,\vec{r}_2,\omega)$, by application of the averaging technique employed by Iguchi [10] and Scanlan [24] in their studies of wave propagation effects. This approach leads to

$$S_x = \frac{1}{A^2} \int_A \int_A S(\vec{r}_1,\vec{r}_2,\omega)\, dA_1\, dA_2 \tag{4a}$$

$$S_y = \frac{R^2}{I_\theta^2} \int_A \int_A d_1\, d_2\, S(\vec{r}_1,\vec{r}_2,\omega)\, dA_1\, dA_2 \tag{4b}$$

$$S_{xy} = \frac{R}{I_\theta A} \int_A \int_A d_2\, S(\vec{r}_1,\vec{r}_2,\omega)\, dA_1\, dA_2 \tag{4c}$$

in which dA_1 and dA_2 are elemental areas of the foundation; $A = \pi R^2$ = the area of the foundation; and $I_\theta = AR^2/2$ = its polar moment of inertia about a vertical centroidal axis.

As is true of the corresponding exact expressions presented by Luco and Mita [15], Eqs. 4 represent weighted averages of $S(\vec{r}_1,\vec{r}_2,\omega)$. However, whereas the weighting functions in the exact formulation are the complex distributions of the actual tractions at the foundation-soil interface, in the procedure employed herein they are taken as linear functions. This is tantamount to representing the restraining action of the supporting medium by a series of mutually independent springs of the Winkler type [24]. There are two main advantages to the use of the approximate formulation over the exact formulation: (a) it reduces the number of independent parameters that must be considered, thereby simplifying the interpretation of the results; and (b) for important special cases, it leads to simple, closed-form expressions for the desired quantities. Additionally, the results are generally of good accuracy.

For the circular foundations examined herein, it is convenient to express $\vec{r}_1$ and $\vec{r}_2$ in Eqs. 1 and 3 in terms of polar coordinates. On substituting Eq. 1 into Eqs. 4, and making use of the appropriate coordinate transformation, one obtains

$$\frac{S_x}{S_g} = \frac{1}{\pi^2} \int_0^1 \int_0^1 \int_0^{2\pi} \int_0^{2\pi} \xi_1 \xi_2 \exp(-b_o^2 \Delta_1) \cos(c_o \Delta_2) \, d\theta_1 d\theta_2 d\xi_1 d\xi_2 \tag{5a}$$

$$\frac{S_y}{S_g} = \frac{4}{\pi^2} \int_0^1 \int_0^1 \int_0^{2\pi} \int_0^{2\pi} \xi_1^2 \xi_2^2 \exp(-b_o^2 \Delta_1) \cos(c_o \Delta_2) \cos\theta_1 \cos\theta_2 \, d\theta_1 d\theta_2 d\xi_1 d\xi_2 \tag{5b}$$

$$\frac{S_{xy}}{S_g} = -\frac{2}{\pi} i \int_0^1 \int_0^1 \int_0^{2\pi} \int_0^{2\pi} \xi_1 \xi_2^2 \exp(-b_o^2 \Delta_1) \sin(c_o \Delta_2) \cos\theta_2 \, d\theta_1 d\theta_2 d\xi_1 d\xi_2 \tag{5c}$$

in which

$$\Delta_1 = \xi_1^2 + \xi_2^2 - 2\xi_1 \xi_2 \cos(\theta_1 - \theta_2) \tag{6a}$$

$$\Delta_2 = \xi_1 \cos\theta_1 - \xi_2 \cos\theta_2 \tag{6b}$$

ξ_1 and ξ_2 are the radial distances of the two points normalized with respect to the radius, R; θ_1 and θ_2 are the corresponding angular coordinates measured from the direction of wave propagation, as shown in Fig. 2-1(b); and b_o and c_o are dimensionless parameters related to the well known frequency parameter, $a_o = \omega R/v_s$, as follows:

$$b_o = \gamma a_o \tag{7}$$

and $c_o = (v_s/c)\, a_o = (\sin\alpha_v)\, a_o$ (8)

In the exact formulation of the problems presented in Refs. 14 and 15, the quantities γ, a_o and v_s/c appear independently.

3.1.1 Integration of Equations. For vertically incident incoherent waves, $c_o = 0$ and the interrelationship of the free-field ground motion and the foundation input motion is defined by the single parameter b_o. Equations 5 in this case can be integrated exactly to yield

$$S_x = \frac{1}{b_o^2}\left\{1 - \exp(-2b_o^2)\,[I_o(2b_o^2) + I_1(2b_o^2)]\right\}S_g \qquad (9a)$$

$$S_y = \frac{1}{b_o^2}\left\{1 - \exp(-2b_o^2)\,[I_o(2b_o^2) + 2I_1(2b_o^2) + I_2(2b_o^2)]\right\}S_g \qquad (9b)$$

$$S_{xy} = 0 \qquad (9c)$$

in which I_o, I_1 and I_2 are modified Bessel functions of the first kind of the order indicated by the subscript. Eq. 9c indicates that the horizontal and torsional components of the foundation input motion are statistically uncorrelated. The derivation of these expressions is given in Appendix A.

For obliquely incident coherent waves, for which $\gamma = b_o = 0$, the interrelationship of the two motions is defined completely by c_o, and Eqs. 5 can again be integrated exactly to yield

$$S_x = \left[2\,\frac{J_1(c_o)}{c_o}\right]^2 S_g \qquad (10a)$$

$$S_y = \left[4\,\frac{J_2(c_o)}{c_o}\right]^2 S_g \qquad (10b)$$

$$S_{xy} = i\left[8\,\frac{J_1(c_o)J_2(c_o)}{c_o^2}\right]S_g \qquad (10c)$$

in which J_1 and J_2 are Bessel functions of the first kind of order one and two, respectively. The latter expressions have been presented previously in Ref. 22. Note that S_{xy} is purely imaginary, indicating that there is a 90° phase angle in this case between the horizontal and torsional components of foundation input motion.

For the more general case involving combinations of wave passage and incoherence effects, formal integration of Eqs. 5 has not proved possible, and the relevant expressions were integrated numerically.

3.1.2 Presentation of Results. The quantities $\sqrt{S_x/S_g}$ and $\sqrt{S_y/S_g}$ define the transfer functions for the amplitudes of the horizontal and rotational components of the foundation input motion, and the magnitude of $S_{xy}/\sqrt{S_x S_y}$ define the degree of correlation or coherence of the components of the motion. A numerical value of unity for the latter quantity indicates that the component motions are fully correlated, while a zero value indicates that they are uncorrelated. These quantities are plotted conveniently in Figs. 3-1 and 3-2 as functions of the modified frequency parameter,

$$\tilde{a}_o = \sqrt{b_o^2 + c_o^2} = \sqrt{\gamma^2 + \sin^2\alpha_v}\ a_o \qquad (11)$$

and the modified incoherence parameter,

$$\tilde{\gamma} = b_o/c_o = \gamma/\sin\alpha_v \qquad (12)$$

For incoherence effects only, $\alpha_v = 0$, $\tilde{\gamma} = \infty$ and $\tilde{a}_o$ reduces to $\gamma a_o = b_o$. Similarly, for wave passage effects only, $\gamma = \tilde{\gamma} = 0$ and $\tilde{a}_o$ reduces to $a_o \sin\alpha_v = c_o$.

Note that whereas the transfer function for the lateral component of the foundation input motion, $\sqrt{S_x/S_g}$, decreases monotonically in Fig. 3-1 with increasing $\tilde{a}_o$, the corresponding function for the torsional component, $\sqrt{S_y/S_g}$, increases from zero to a peak and then decreases monotonically.

3.1.3 Accuracy of Solutions. As a measure of the accuracy of the reported data, the results computed for incoherence effects only and for wave passage effects only are compared in Fig. 3-3 with the corresponding exact solutions of Luco and Mita [14,15]. Since the factors γ and $\sin\alpha_v = v_s/c$ appear independently in the exact solutions, several different values are considered for these parameters. No comparisons are made for combinations of incoherence and wave passage as the exact solutions are not available in this case.

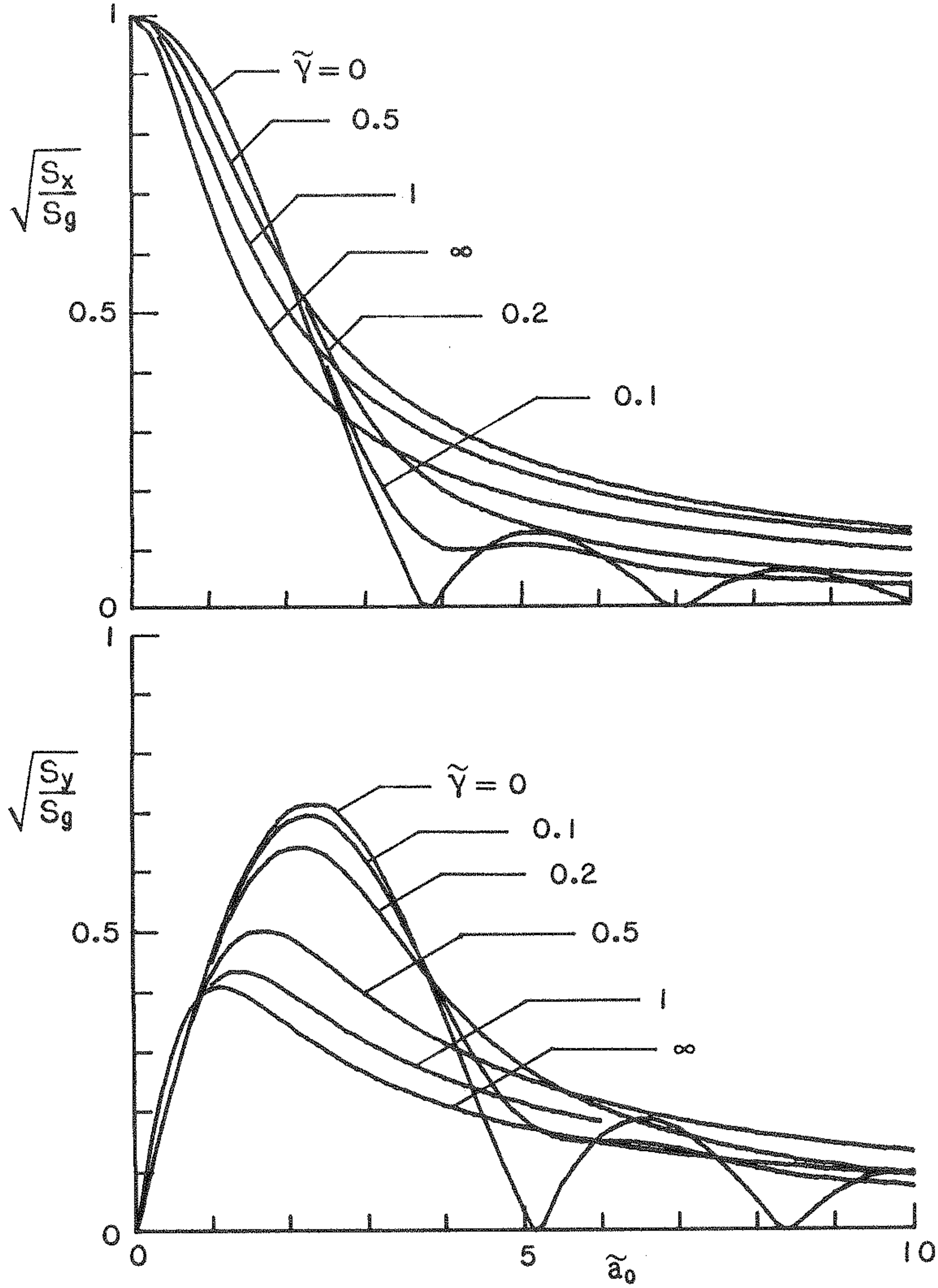


FIG. 3-1 Magnitudes of Transfer Functions between Free-Field Ground Motion and Foundation Input Motions

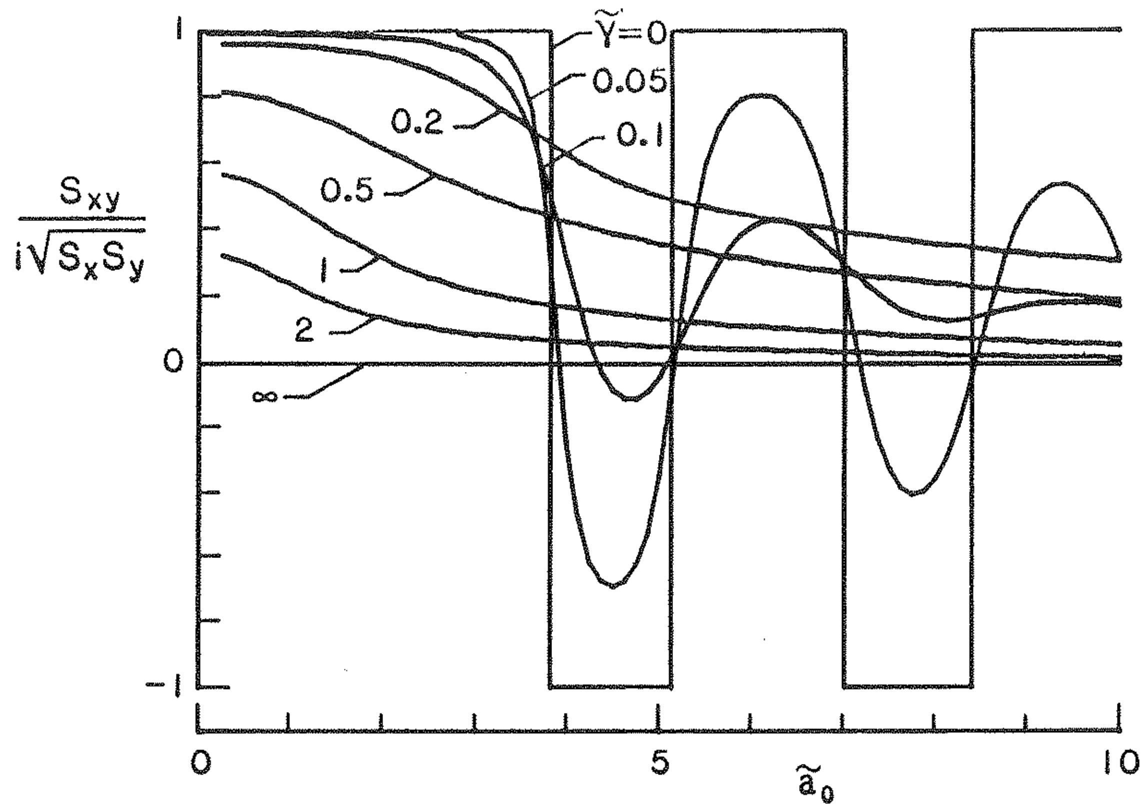


FIG. 3-2 Normalized Cross PSD Function for Horizontal and Torsional Components of Foundation Input Motion

Considering the uncertainties that are inherent in the definition of the incoherence function and in the choice of the parameter γ, the degree of agreement in the two sets of results displayed in Fig. 3-3 is deemed to be quite satisfactory. Note should also be taken of the fact that, excepting the narrow frequency ranges where the curves for wave passage only exhibit notch-like trends, the approximate solutions overestimate the amplitudes of foundation input motions.

3.1.4 Other Meanings for Results. Although defined specifically for the displacement histories of the foundation input motion, the spectral density ratios S_x/S_g, S_y/S_g and S_{xy}/S_g also define the ratios $S_{\dot{x}}/S_{\dot{g}}$, $S_{\dot{y}}/S_{\dot{g}}$, $S_{\dot{x}\dot{y}}/S_{\dot{g}}$ and $S_{\ddot{x}}/S_{\ddot{g}}$, $S_{\ddot{y}}/S_{\ddot{g}}$, $S_{\ddot{x}\ddot{y}}/S_{\ddot{g}}$ of the corresponding velocity and acceleration histories. Recall that the psd function for the first derivative of a process is given by the product of $(2\pi f)^2$ and the psd function of the original process.

3.2 Spectral Characterization of Structural Response

With the psd functions of the foundation input motion established, the corresponding functions of the structural response can be obtained by well-established procedures (e.g., Ref. 12). Let S_u be the psd function of the structural deformation, u, induced by the lateral component of the foundation input motion; and let S_v be the corresponding function of the deformation, $v = \psi R$, induced along the perimeter of the structure by the torsional component of response. The quantity ψ represents the angular deformation of the structure. These functions are related to the psd functions of the foundation input accelerations, $S_{\ddot{x}}$ and $S_{\ddot{y}}$, by

$$S_u = |H_u|^2 S_{\ddot{x}} \tag{13}$$

and $$S_v = |H_v|^2 S_{\ddot{y}} \tag{14}$$

in which H_u = the transfer function for lateral response, given by

$$H_u = -\frac{1}{p_x^2}\,\frac{1}{1-(\omega/p_x)^2 + i2\zeta_x(\omega/p_x)} \tag{15}$$

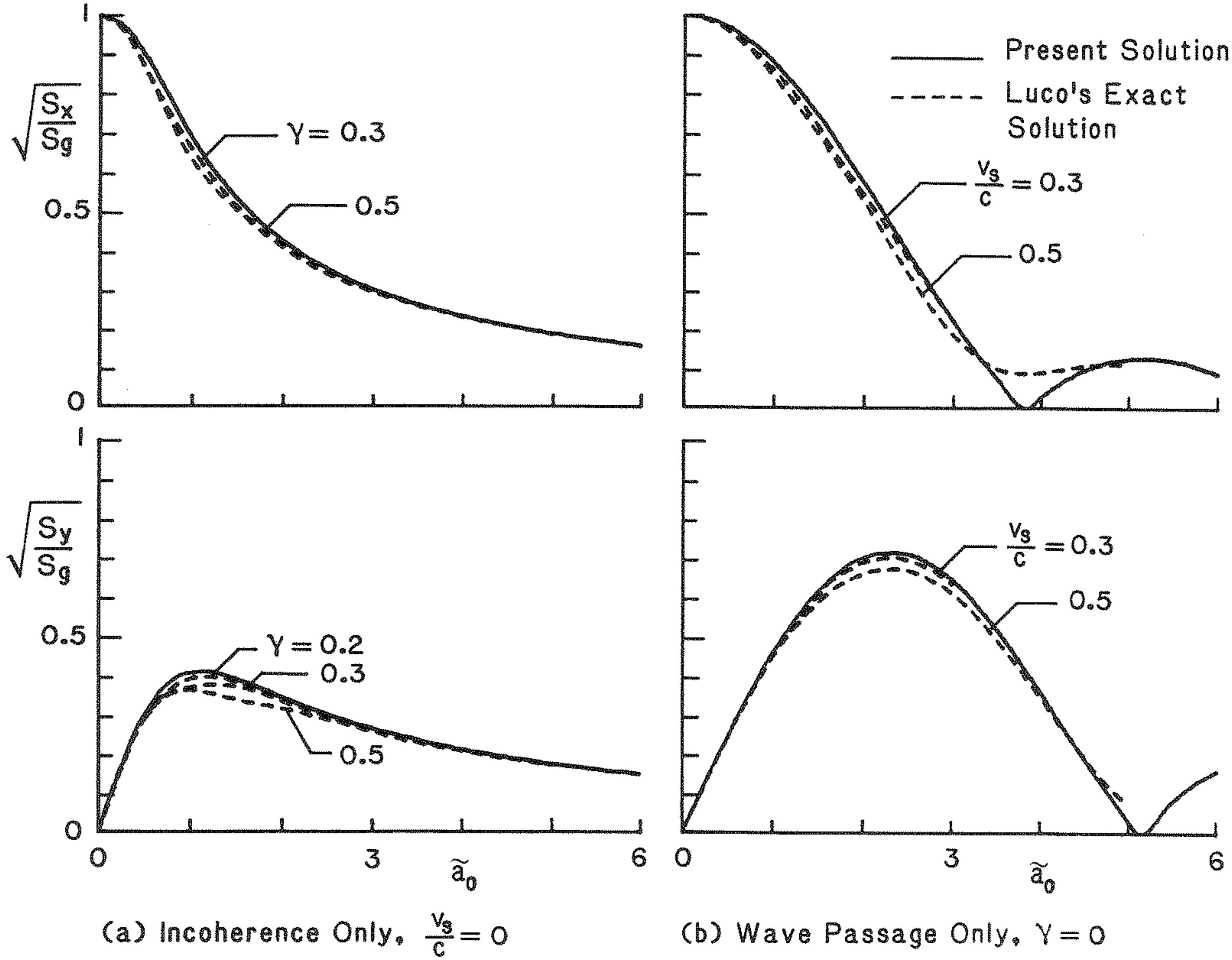


FIG. 3-3 Comparison of Approximate and Exact Magnitudes of Foundation Transfer Functions

H_v = the corresponding function for torsional response, obtained from Eq. 15 by replacing p_x by p_θ and ζ_x by ζ_θ; and vertical bars indicate the modulus of the enclosed quantity. Similarly, the psd function S_w for the total deformation at the most highly stressed point on the periphery of the structure, w = u + v, is given by [12]

$$S_w = S_u + S_v + 2\,|\,\mathrm{Re}(H_u\, H_v^*\, S_{\ddot{x}\ddot{y}})| \tag{16}$$

in which $S_{\ddot{x}\ddot{y}}$ = the cross psd function of the lateral and circumferential components of the foundation input accelerations; Re denotes the real part of the indicated quantity; and a star superscript denotes the complex conjugate of the quantity to which it is attached.

3.3 Characterization of Free-Field Earthquake Ground Motions

The local psd function for the set of acceleration traces considered in the remainder of this paper is taken in the form

$$S_{\ddot{g}} = \begin{cases} \dfrac{f^4}{0.5 + f^4}\left(1 - \dfrac{f^2}{f_o^2}\right) S_o & \text{for } f \leqq f_o \\ 0 & \text{for } f \geqq f_o \end{cases} \tag{17}$$

in which S_o = a constant; $f = \omega/2\pi$ = the exciting frequency in cps; and f_o = the cut-off frequency, taken as 15 cps. Of the same general form as that employed in a related study by Pais and Kausel [22], the function $S_{\ddot{g}}$, along with the associated functions for ground velocity and ground displacement, are plotted in Fig. 3-4, with all peaks normalized to a unit value. As would be expected, the psd function for velocity decays much more rapidly with frequency than that for acceleration, and the corresponding displacement function decays even faster.

Let $\ddot{X}_g$ be the mean of the absolute maximum peaks of the acceleration traces, and $\dot{X}_g$ and X_g be the corresponding means of the velocity and displacement traces. These values were computed from Der Kieureghian's empirical expressions [5] summarized in Appendix B, considering the duration of the intense portion of the excitation to be t_o = 20 sec. The

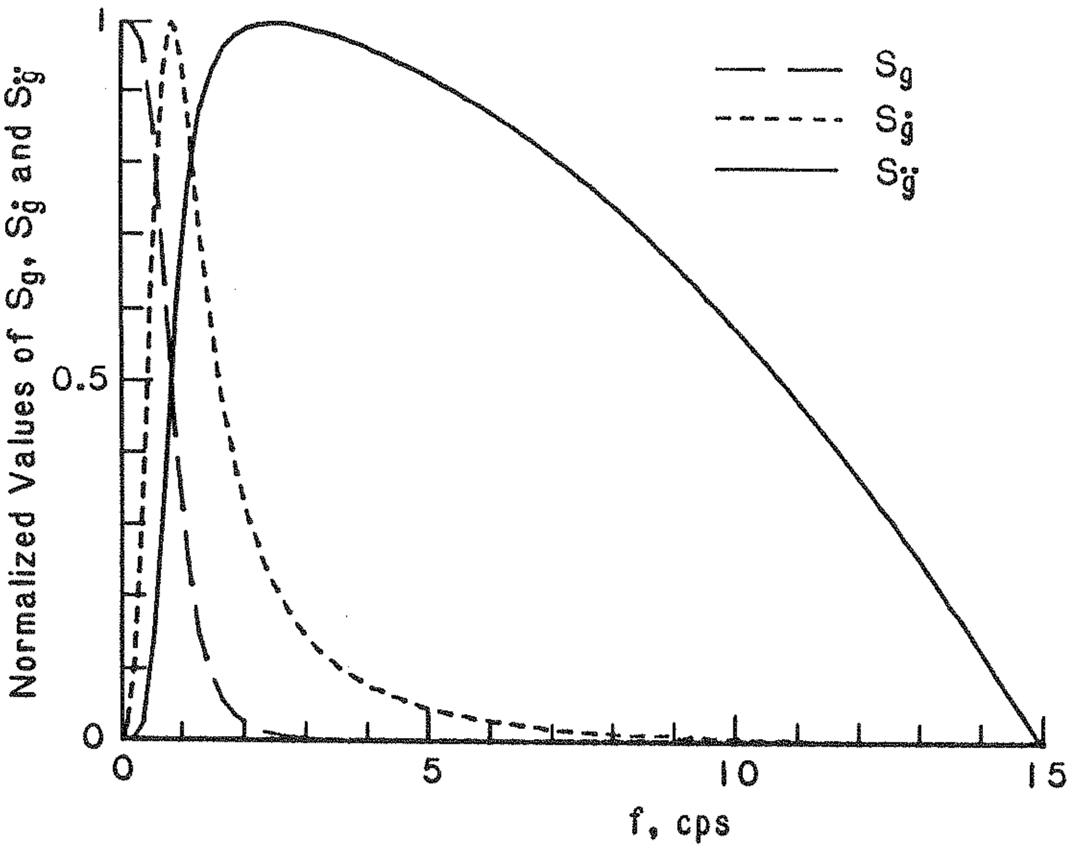


FIG. 3-4 Normalized PSD Functions for Free-Field Ground Motions Considered

resulting values are $\ddot{X}_g = 26.17\sqrt{S_o}$, $\dot{X}_g = 1.417\sqrt{S_o}$ and $X_g = 0.2468\sqrt{S_o}$.

3.4 Foundation Input Motion

Before examining the response of the structure, it is desirable to compute the mean peak values of the acceleration, velocity and displacement traces of the horizontal and circumferential components of the foundation input motion. The relevant values for the horizontal component of motion are denoted by $\ddot{X}$, $\dot{X}$ and X, and those for the circumferential component along the periphery of the foundation are denoted by $\ddot{Y}$, $\dot{Y}$ and Y. Computed by Der Kieureghian's approximation from the appropriate psd functions, these values are plotted in Fig. 3-5 normalized with respect to the mean peak values of the corresponding histories of the free-field ground motion.

For the multifrequency, transient excitation considered in this section, the solution is controlled by the effective transit time,

$$\tilde{\tau} = \sqrt{\gamma^2 + \sin^2\alpha_v}\ \tau \qquad (18)$$

in which $\tau = R/v_s$ = the time required for the shear wave to traverse the radius of the foundation; and by the modified incoherence parameter, $\tilde{\gamma}$, defined by Eq. 12.

The following observations may be made and inferences drawn from the data presented in Fig. 3-5:

1. The reduction in the horizontal component of the foundation input motion and the corresponding increase in the rotational component are greatest for acceleration, much smaller for velocity, and almost negligible for displacement. Since the foundation filters the high-frequency wave components more effectively than the low-frequency wave 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.

2. Considering that the response of high-frequency systems is acceleration-sensitive whereas that of low-frequency systems is

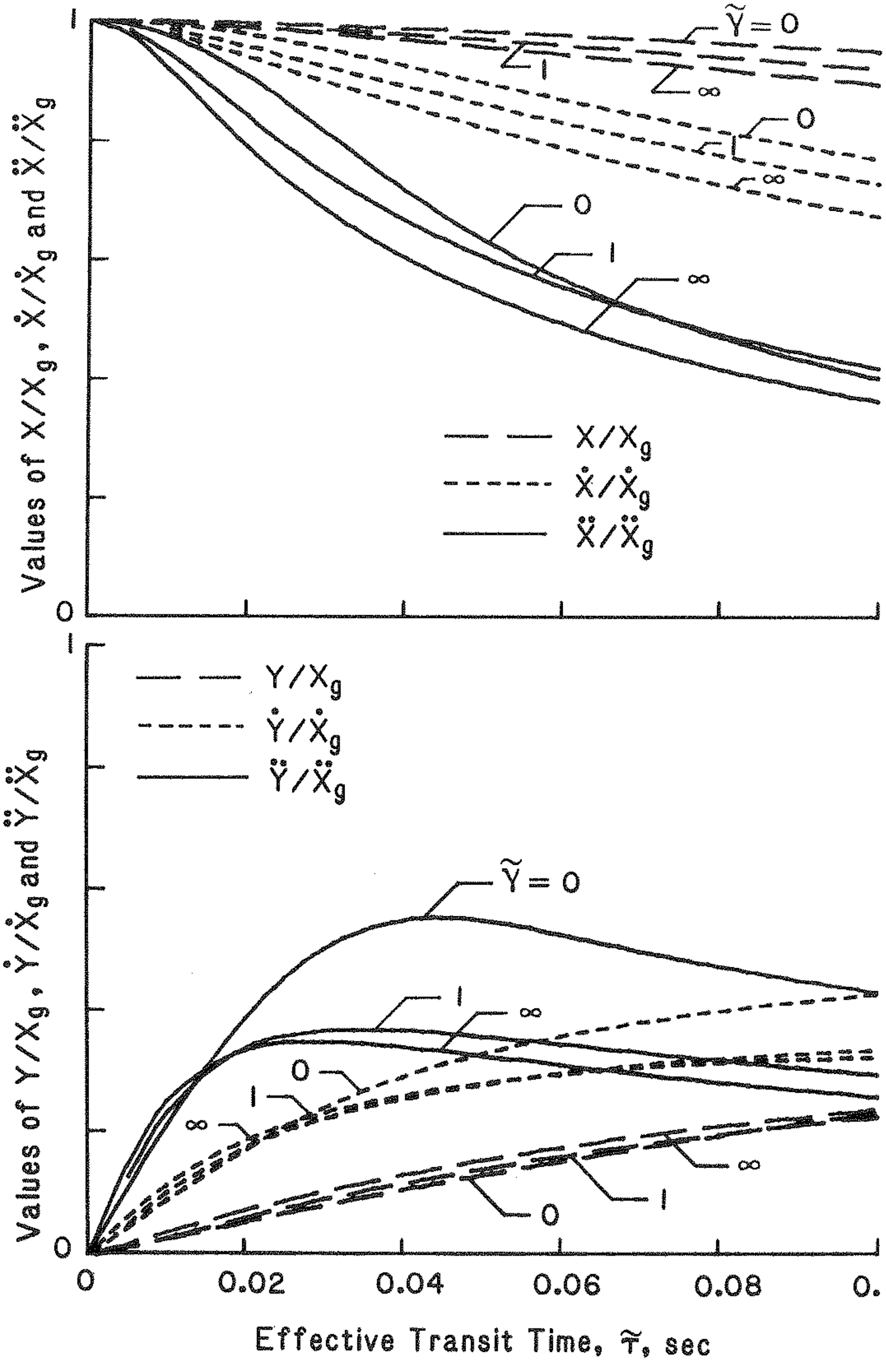


FIG. 3-5 Normalized Mean Peak Values of Lateral and Torsional Components of Foundation Input Accelerations, Velocities and Displacements

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.

3.5 Effects of Ground-Motion Incoherence on Structural Response

Let U_x = the mean of the maximum values of the structural deformations induced by the ensemble of lateral components of the foundation input motions, and U_y = the corresponding mean of the deformations induced at the periphery of the deck by the torsional components. These quantities have been evaluated for vertically propagating incoherent shear waves ($\tilde{\gamma} = \infty$), and the results are displayed in Fig. 3-6 in the form of tripartite response spectra. The solid curves in the upper part of the figure refer to lateral response, and the lower curves refer to torsional response. Several values of the effective transit time parameter, $\tilde{\tau}$, are considered, including the limiting value of $\tilde{\tau} = 0$ for which there is no kinematic interaction. The damping factors for both modes of response are taken as $\zeta_x = \zeta_\theta = 0.02$.

The left-hand diagonal scale in the upper part of Fig. 3-6 represents U_x normalized with respect to the mean peak value of the free-field displacement, X_g; the vertical scale represents the corresponding pseudo-velocity, $V_x = p_x U_x$, normalized with respect to $\dot{X}_g$; and the right-hand diagonal scale represents the corresponding pseudo-acceleration, $A_x = p_x V_x$, normalized with respect to $\ddot{X}_g$. In an analogous manner, the three scales in the lower part of the figure represent the deformation ratio, U_y/X_g; the pseudo-velocity ratio, $V_y/\dot{X}_g$, in which $V_y = p_\theta U_y$; and the pseudo-acceleration 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 motions, the lateral component of the response of high-frequency systems in Fig. 3-6 is affected materially by ground incoherence, and this effect is particularly large in the practically important region of the response

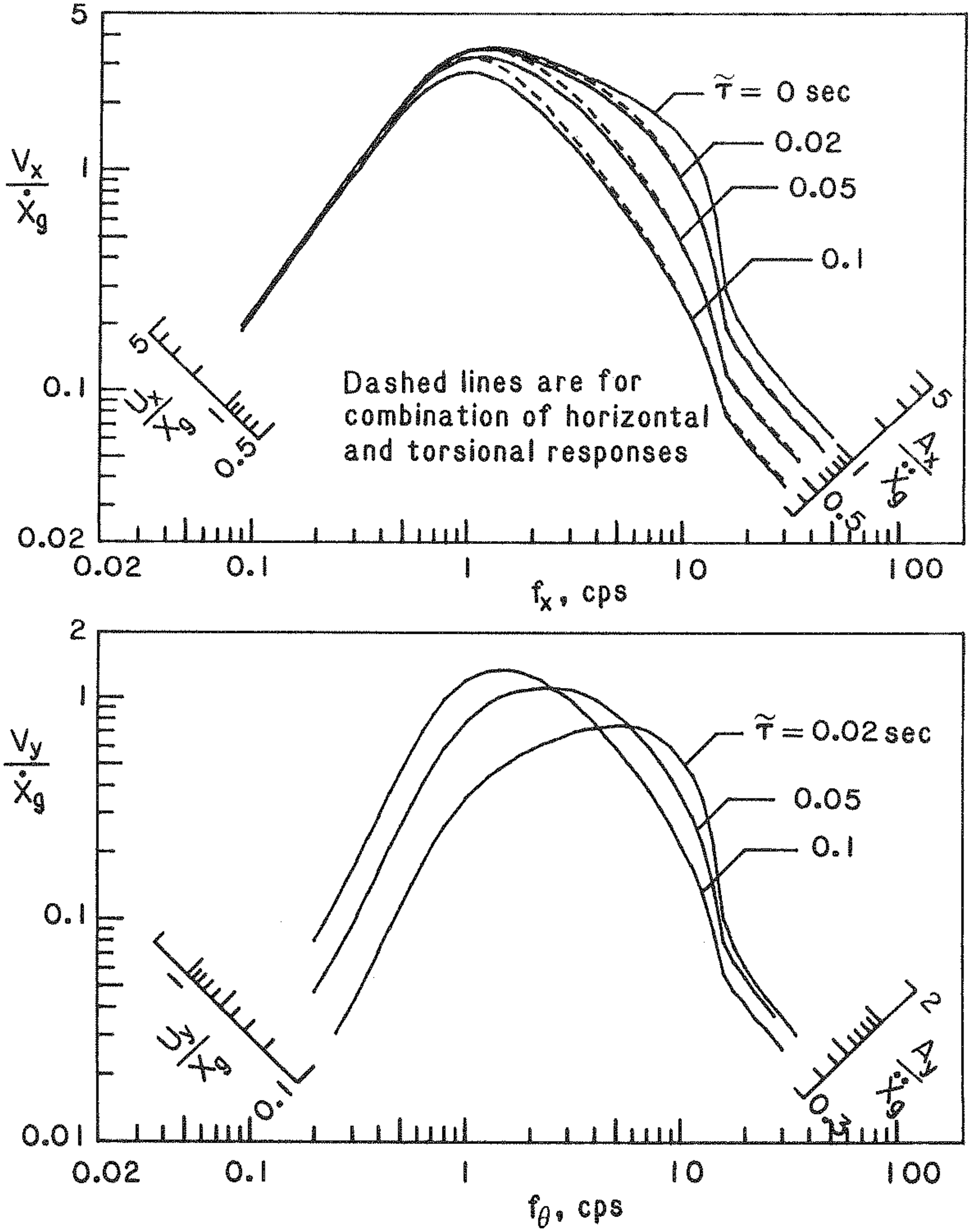


FIG. 3-6 Effects of Ground Motion Incoherence on Maximum Deformations of Structures with $\zeta_x = \zeta_\theta = 0.02$ for Vertically Propagating Waves

spectrum within which the pseudo-acceleration attains its maximum value. For $\gamma = 0.2$ and $\tilde{\tau} = 0.02$ sec. (a value corresponding to, say, R = 100 ft. and v_s = 1000 ft/sec), the maximum value of A_x is 78 percent of that obtained for a fully coherent, uniform free-field ground motion; for $\tilde{\tau}$ = 0.05 sec., the corresponding ratio is 55 percent. The reductions are significantly less pronounced for medium-frequency systems and practically negligible for low-frequency systems. For systems of very high-frequency, for which A_x may be considered to be equal to the mean peak value of the foundation input acceleration, the percentage reductions are, of course, identical to those indicated in Fig. 3-5 for the foundation input acceleration.

The general trends of the response spectra for the torsional deformation in Fig. 3-6 are consistent with those of the corresponding curves for the foundation input motion presented in Fig. 3-5. Specifically, in the low-frequency, displacement-sensitive region, the response increases with increasing values of the effective transit time, $\tilde{\tau}$, whereas in the high-frequency, acceleration-sensitive region, the response values for $\tilde{\tau} = 0.02$ sec. are higher than those for the higher values of $\tilde{\tau}$ considered. Furthermore, the percentage changes in response are comparable to those for the controlling values of the foundation input motion.

The component of the response contributed by the rotation of the foundation is generally small, and the combined effect of lateral and torsional responses is generally only slightly greater than that due solely to lateral response. The mean maximum values of the total deformation for the most highly stressed column along the periphery of the structure were evaluated considering $p_\theta/p_x = 1.5$, and the results are shown by the dashed lines in Fig. 3-6. These results were computed by Der Kieureghian's approximation making use of Eq. 16 for the psd function of the combined motion.

3.5.1 Comparison of Incoherence and Wave Passage Effects. Some of the response spectra for the incoherent ground motions presented in Fig. 3-6 are compared in Fig. 3-7 with those computed considering only wave passage effects, and combinations of wave passage and incoherence

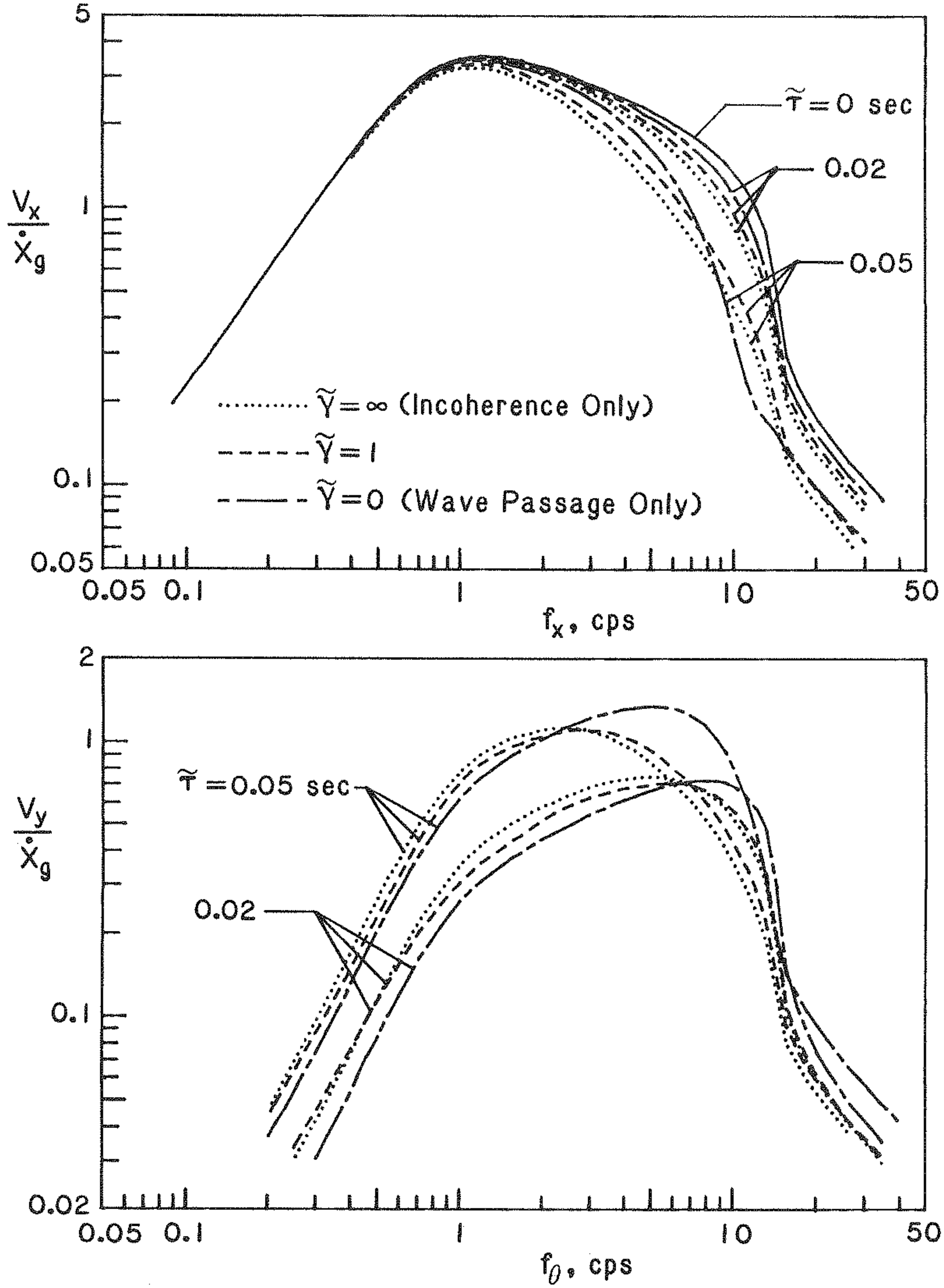


FIG. 3-7 Effects of Ground Motion Incoherence and Wave Passage on Maximum Deformations of Structures with $\zeta_x = \zeta_\theta = 0.02$

represented by a value of $\tilde{\gamma} = 1$. It should be clear that the results are not particularly sensitive to the choice of the parameter $\tilde{\gamma}$, and that this insensitivity is fully compatible with that observed in Fig. 3-5 for the peak values of the foundation input motions. Indeed, the ratio of the low-frequency limiting values of U_x for $\tilde{\gamma} = 0$ and $\tilde{\gamma} = \infty$ in Fig. 3-7 is almost identical to that of the peak values of the lateral component of the foundation input displacements in Fig. 3-5, and the ratio of the corresponding values of U_y is almost identical to the displacement ratio of the torsional component of the foundation input motion. Similarly, the ratios of the high-frequency limits of A_x and A_y in Fig. 3-7 are identical to those obtained from Fig. 3-5 for the mean peak values of the lateral and torsional components of the foundation input accelerations. It follows that, to the degree of approximation represented by the differences in the results displayed in Fig. 3-7, the effects of ground motion incoherence may be replaced by those of wave passage and vice versa. This possibility has also been suggested by Luco and Wong [16] from examination of the relevant foundation transfer functions. In implementing this replacement, it is important that the value of $\tilde{\tau}$ be the same in the two cases.

SECTION 4
INERTIAL INTERACTION EFFECTS

The inertial interaction effects are now evaluated by a simple modification of the procedure used in previous studies in which the effects of kinematic interaction were neglected (e.g., Refs. 26, 28). For each mode of excitation, it is only necessary to replace the free-field motion by the appropriate component of the foundation input motion.

The following steps are involved in the analysis: First, the harmonic response of the system is evaluated making use of the appropriate complex-valued foundation impedance functions. Next, the psd functions of the torsional and lateral components of structural response are determined. The desired mean peak values of the responses are finally computed from Der Kiureghian's approximation. Additional details are given in Appendix C.

The foundation impedances for the torsional mode of vibration were computed from the approximate closed-form expressions of Veletsos and Nair [29], and those for the horizontal and rocking motions were computed from the corresponding expressions of Veletsos and Verbic [30]. The cross coupling terms between horizontal and rocking motions were presumed to be negligible.

The principal parameters that influence the response of the system are the characteristics of the free-field ground motion; the fixed-base natural frequencies of the structure, f_x and f_θ, and the associated damping factors, ζ_x and ζ_θ; the height to base radius ratio, h/R; the mass density ratio for the structure, defined conveniently as $\delta = m/(\pi\rho R^2 h)$, in which the denominator represents the total mass of the structure when filled with the supporting soil; and the wave transit times, τ and $\tilde{\tau}$. It is important to note that whereas the kinematic interaction effects are defined completely by $\tilde{\tau}$, the evaluation of the inertial interaction effects requires the separate specification of the parameters γ and τ. Other parameters affecting the response of the system are Poisson's ratio for the supporting medium, ν; the mass ratio of the foundation and superstructure, 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. For the solutions presented herein, $\zeta_x = \zeta_\phi = 0.02$; $\delta = 0.15$; $\nu = 1/3$; and m_f (and hence I_f and J_f) are considered to be negligible.

4.1 Results for Vertically Propagating Incoherent Waves

Fig. 4-1 shows response spectra for lateral and torsional response obtained for vertically propagating incoherent waves, taking $\gamma = 0.4$ and $\tau = R/v_s = 0.05$ sec. Three sets of solutions are presented: (a) making no provision for soil-structure interaction, i.e., considering the foundation motion to be equal to the free-field ground motion; (b) providing only for the kinematic interaction effects, i.e., using as base excitation the foundation input motions; and (c) providing for both kinematic and inertial interaction effects; i.e., analyzing the structure-foundation-soil system exactly as a coupled system. In the analysis of the inertial interaction effects, two values of h/R are used: a unit value, corresponding to short stubby structures, and a value of 3, corresponding to taller, more slender structures. Solutions (a) and (b) are independent of h/R, whereas solutions (b) are valid for all combinations of γ and τ for which $\gamma\tau = \tilde{\tau} = 0.02$ sec.

Previous studies of soil-structure interaction involving only inertial interaction effects [3,6,26,27] have shown that these effects may be evaluated to a high degree of approximation using the free-field ground motion as the foundation input motion and merely modifying the relevant natural frequency and damping of the structure. The modified frequency and damping are taken such that, for each mode of vibration, the magnitude and location of the resonant peak of the relevant harmonic response are identical for the actual and replacement systems. For structures for which the kinematic interaction effects are important, this approach would require that the response of the structure be evaluated for the horizontal and torsional components of the foundation input motion rather than for the free-field ground motion.

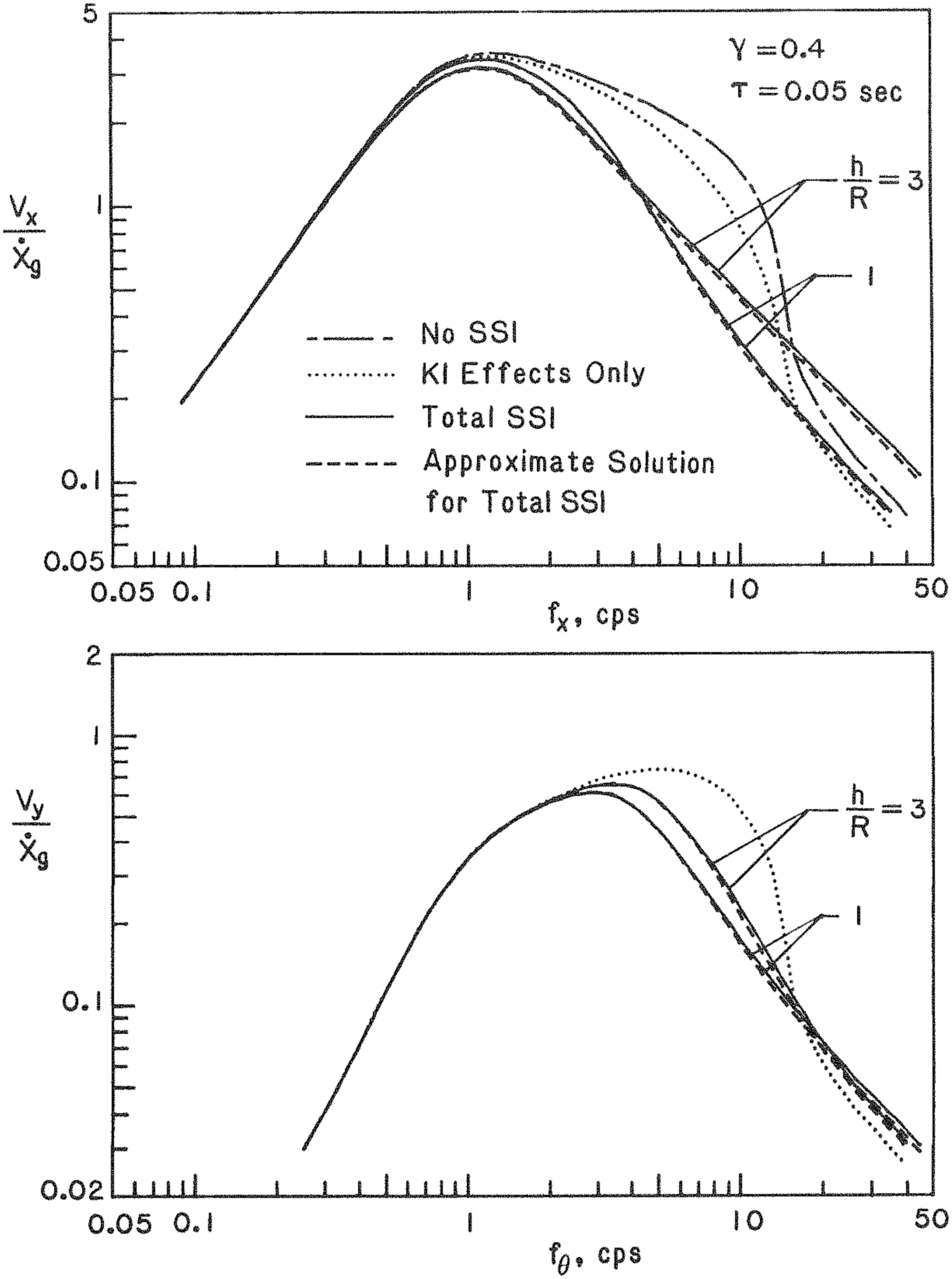


FIG. 4-1 Comparison of Effects of Kinematic and Inertial Interaction on Maximum Deformations of Structures with $\zeta_x = \zeta_\theta = 0.02$

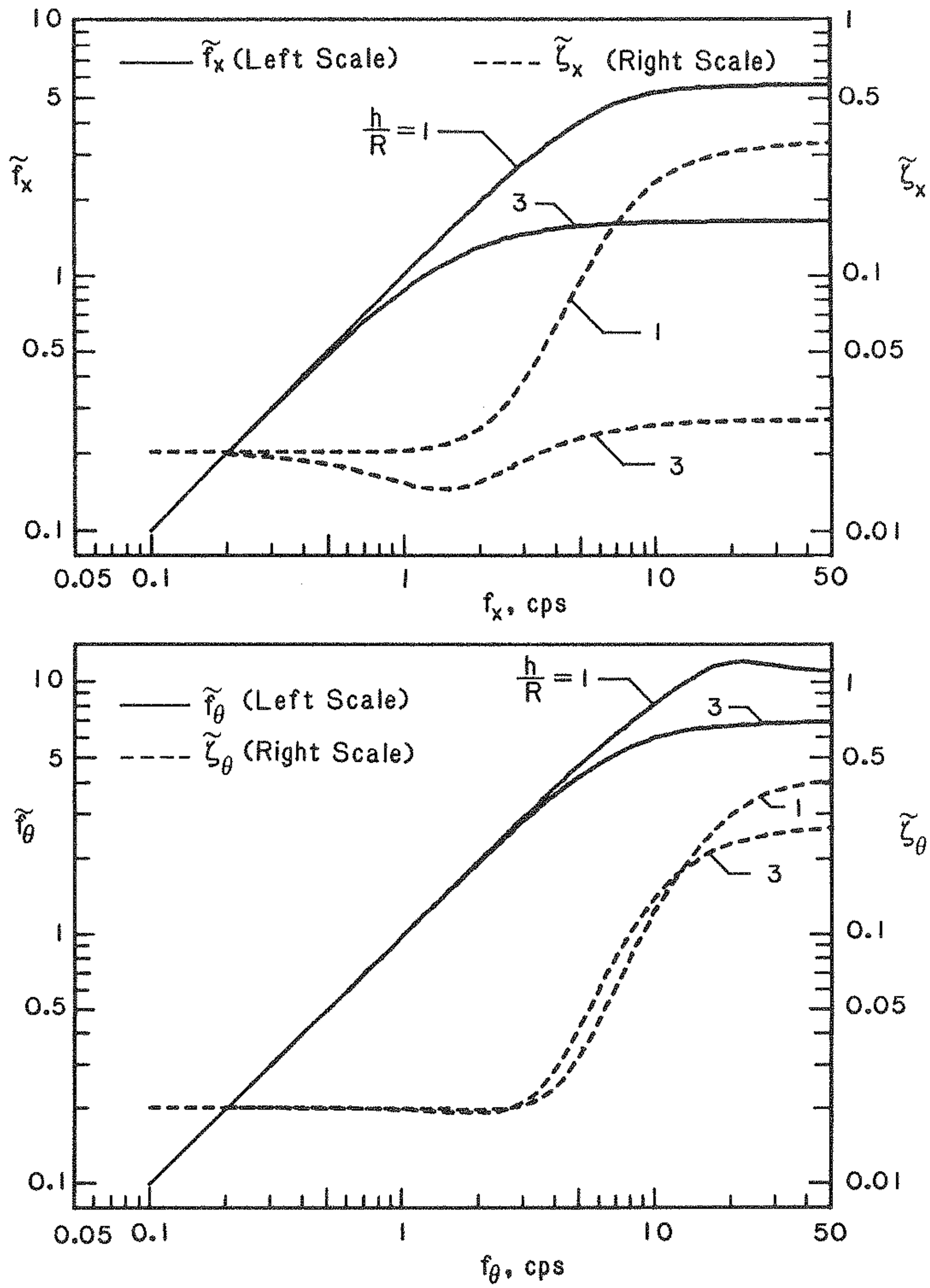


FIG. 4-2 Natural Frequencies and Damping of Modified Systems in Approximate Analysis of Inertial Interaction Effects

The mean maximum values of the responses obtained by this approximate procedure are shown by the dashed lines in Fig. 4-1, and the values of the modified natural frequencies and damping factors are identified in Fig. 4-2. Denoted with a tilda superscript, the modified frequencies are, of course, lower than the corresponding fixed-base frequencies, and the modified damping factors are higher than the value of $\zeta_x = \zeta_\theta = 0.02$ assumed for the fixed-base structure.

The following trends should be observed in these figures:

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 more important than the KI effects.

3. Unlike kinematic interaction which generally reduces the lateral response, inertial interaction may increase the corresponding response of tall, slender structures in the high frequency region of the response spectrum. Such structures, however, typically fall in the middle-frequency region of the spectrum, for which the interaction effects are relatively small.

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

5. Provided the base excitation for the structure is taken equal to the foundation input motion rather than the free-field ground motion, the concept of modifying the fixed-base natural frequencies and associated damping values of the system provides a simple and highly reliable practical means for assessing the II effects.

It may be surprising that the values of $\tilde{f}_\theta$ and $\tilde{\zeta}_\theta$ in Fig. 4-2 are functions of the ratio h/R. This is due to the fact that with the value of the mass ratio, δ, fixed in these solutions, the polar mass moment of inertia of the system, J, is different for different values of h/R.

SECTION 5
CONCLUSIONS

1. The information and concepts presented herein provide valuable insight into the nature of kinematic and inertial interaction effects for simple structures subjected to earthquakes, and into the effects and relative importance of the numerous parameters involved.

2. In the approximate method of analysis employed, the kinematic interaction effects are defined completely by the effective transit time, $\tilde{\tau}$, and the modified incoherence parameter, $\tilde{\gamma}$.

3. Even for vertically propagating waves, kinematic interaction may reduce significantly the critical responses of high-frequency systems. These reductions are generally smaller than, but of approximately the same order of magnitude as, those due to inertial interaction.

4. Reliable estimates of the effects of kinematic interaction on the peak values of structural response may be obtained from knowledge of the corresponding values of the acceleration, velocity and displacement traces of the foundation input motion. The latter quantities may be computed from analyses of the response of the massless foundation to the free-field ground motion.

5. Insofar as the mean maximum values of the responses are concerned, the kinematic interaction effects due to ground motion incoherence are similar to those due to wave passage, and the two effects may be interrelated.

6. An excellent approximation to the inertial interaction effects may be obtained by a previously recommended simple procedure [3,6,26,27] using as base excitation the foundation input motion rather than the free-field motion. The inertial interaction effects in this approach are expressed by changes in the natural frequency of vibration and the associated damping of the structure for the mode of vibration considered.

SECTION 6

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APPENDIX A
DERIVATION OF EQUATIONS 9

For incoherence effects only, the integrands in Eqs. 5a and 5b are symmetric about $\xi_1 = \xi_2$. This symmetry may be provided for by multiplying these expressions by 2 and changing the upper limit of integration of ξ_2 from unity to ξ_1. On using the identity

$$I_n(z) = \frac{1}{\pi}\int_0^{\pi} e^{z\cos\theta}(\cos n\theta)d\theta \tag{19}$$

given as Eq. 9.6.19 in Abromowitz and Stegun [2], the specialized form of Eqs. 5a and 5b are integrated with respect to the circumferential co-ordinates to yield

$$\frac{S_x}{S_g} = 8\int_0^1\int_0^{\xi_1} \xi_1\xi_2 \exp\left[-b_o^2(\xi_1^2+\xi_2^2)\right] I_o(2b_o^2\xi_1\xi_2)\, d\xi_1\, d\xi_2 \tag{20a}$$

$$\frac{S_y}{S_g} = 16\int_0^1\int_0^{\xi_1} (\xi_1\xi_2)^2 \exp\left[-b_o^2(\xi_1^2+\xi_2^2)\right] I_1(2b_o^2\xi_1\xi_2)\, d\xi_1\, d\xi_2 \tag{20b}$$

The dummy variable ξ_2 in these equations is then expressed as $\xi_2 = s\xi_1$, and the resulting expressions are integrated with respect to s by making use of the identity (See Eq. 6.631.8 in Ref. 7)

$$\int_0^1 s^{n+1} \exp^{-\alpha s^2} I_n(2\alpha s)ds = \frac{1}{4\alpha}\left[e^{\alpha} - e^{-\alpha}\sum_{y=-n}^{n} I_y(2\alpha)\right] \tag{21}$$

to yield

$$\frac{S_x}{S_g} = \frac{1}{b_o^2}\int_o^1 2\xi_1\left[1 - \exp(-2b_o^2\xi_1^2)\, I_o(2b_o^2\xi_1^2)\right] d\xi_1 \tag{22a}$$

$$\frac{S_y}{S_g} = \frac{4}{b_o^2}\int_o^1 \xi_1^3\left\{1 - \exp(-2b_o^2\xi_1^2)\left[I_o(2b_o^2\xi_1^2) + 2I_1(2b_o^2\xi_1^2)\right]\right\} d\xi_1 \tag{22b}$$

Finally, on letting $a = 2b_o^2\xi_1^2$ and making use of the identities

$$\int_0^z e^{-a}\, a^n I_n(a)\, da = \frac{e^{-z}\, z^{n+2}}{2n+1}\, [I_n(z) + I_{n+1}(z)] \tag{23}$$

$$\int_0^z a^n\, I_{n-1}(a)\, da = z^n I_n(z) \tag{24}$$

$$I_o(z) - I_2(z) = \frac{2}{z}\, I_1(z) \tag{25}$$

given as Eqs. 11.3.12, 11.3.25 and 9.6.26 in Abromowitz and Stegun [2], Eqs. 22a and 22b are integrated to yield Eqs. 9a and 9b. Equation 9c follows from the fact that the integrand of Eq. 5c is antisymmetric in this case.

For small values of b_o, application of Taylor's series expansion to Eqs. 9a and 9b yields

$$S_x = [\, 1 - b_o^2 + \frac{5}{6}\, b_o^4 + \ldots \,]\, S_g \tag{26a}$$

$$S_y = [\frac{1}{2}\, b_o^2 - \frac{2}{3}\, b_o^4 + \ldots]\, S_g \tag{26b}$$

APPENDIX B

EVALUATION OF PEAK VALUES OF INPUT AND RESPONSE

Let z(t) be a stationary, ergodic random Gaussian process with zero mean and limited duration, t_o, and let Z be the ensemble mean of its peak values. Further, let $G(\omega)$ be the one-sided power spectral density of the process, and λ_o, λ_1 and λ_2 be its first three moments, defined by

$$\lambda_n = \int_0^\infty \omega^n G(\omega)\, d\omega \qquad n = 0,1,2 \tag{27}$$

The value of Z in Der Kiureghian's approach is evaluated conservatively from

$$Z = \left[\sqrt{2\, \ell n(\mu_e t_o)} + \frac{0.5772}{\sqrt{2\, \ell n(\mu_e t_o)}}\right]\sqrt{\lambda_o} \tag{28}$$

in which

$$\mu_e t_o = \begin{cases} 2.1 \text{ or } 2q\mu t_o \text{ if greater than } 2.1 & \text{for } q \leq 0.1 \\ (1.63\, q^{0.45} - 0.38)\, \mu t_o & \text{for } 0.1 \leq q \leq 0.69 \\ \mu t_o & \text{for } q \geq 0.69 \end{cases} \tag{29}$$

$\mu = \sqrt{(\lambda_2/\lambda_o)}/\pi$ = the mean zero-crossing rate of the process; and

$q = \sqrt{1 - \lambda_1^2/(\lambda_o\lambda_2)}$ = Vanmarcke's bandwidth parameter [25].

APPENDIX C

HARMONIC RESPONSE OF SYSTEMS WITH INERTIAL INTERACTION

C.1 Torsionally Excited System

Let $\chi = \chi(t)$ and $\chi_f = \chi_f(t)$ be the torsional components of the foundation input displacement and the actual foundation displacement, respectively, and $\psi = \psi(t)$ be the resulting torsional deformation of the structure. The equations of motion for the system may then be written as

$$\ddot{\psi} + 2\zeta_\theta p_\theta \dot{\psi} + p_\theta^2 \psi = - \ddot{\chi}_f \qquad (30)$$

and $$J(\ddot{\psi} + \ddot{\chi}_f) + J_f \ddot{\chi}_f + Q_\theta(t) = 0 \qquad (31)$$

in which a dot superscript denotes differentiation with respect to time; J and J_f are the polar mass moments of inertia of the structure and foundation, respectively; and $Q_\theta(t)$ = the instantaneous value of the torque at the foundation-soil interface. Eq. 30 expresses the dynamic equilibrium of the forces acting on the structural mass, whereas Eq. 31 expresses the fact that the sum of the torsional moments due to the inertia of the structure and the foundation equals the torque acting at the foundation-soil interface.

For the harmonic response considered

$$\chi(t) = X e^{i\omega t} \qquad (32a)$$

$$\chi_f(t) = X_f e^{i\omega t} \qquad (32b)$$

$$\psi(t) = \Psi e^{i\omega t} \qquad (32c)$$

and $$Q_\theta(t) = K_\theta (X_f - X) e^{i\omega t} \qquad (33)$$

in which X, X_f and Ψ are the complex-valued amplitudes of χ, χ_f and ψ, respectively; and K_θ = the complex-valued torsional impedance of the massless foundation.

Equations 30 and 31 are solved in three steps as follows: First, on substituting Eqs. 32b and 32c and its derivatives into Eq. 30, the deformation amplitude of the structure, Ψ, is expressed in terms of the amplitude of the torsional component of the foundation acceleration, $\ddot{X}_f$, as

$$\Psi = H_v \ddot{X}_f \tag{34}$$

in which $\ddot{X}_f = -\omega^2 X_f$, and H_v = the transfer function for torsional response, given by

$$H_v = -\frac{1}{p_\theta^2} \frac{1}{1 - (\omega/p_\theta)^2 + i2\zeta_\theta(\omega/p_\theta)} \tag{35}$$

Next, Eq. 34, along with Eqs. 32b, 32c and 33, are substituted into Eq. 31, and the resulting expression is solved for $\ddot{X}_f$. This step yields

$$\ddot{X}_f = T_v \ddot{X} \tag{36}$$

in which

$$T_v = \frac{\kappa_\theta}{\kappa_\theta - [(TR)_\theta + (J_f/J)a_o^2]} \tag{37}$$

$\kappa_\theta = K_\theta R^2/(J v_s^2)$; and $(TR)_\theta$ = the torsional transmissibility of the system, given by

$$(TR)_\theta = -(p_\theta^2 + i2\xi_\theta \omega p_\theta)H_v = \frac{1 + i2\zeta_\theta(\omega/p_\theta)}{1 - (\omega/p_\theta)^2 + i2\zeta_\theta(\omega/p_\theta)} \tag{38}$$

The expression for $\ddot{X}_f$ defined by Eq. 36 is finally substituted into Eq. 34 to yield

$$\Psi = H_v T_v \ddot{X} \tag{39}$$

from which it follows that the psd function of the deformation of the structure along its periphery, S_v, is given by

$$S_v = |H_v|^2 |T_v|^2 S_{\ddot{y}} \tag{40}$$

The factor $|T_v|^2$ in the latter expression represents the effect of inertial interaction.

For the circular foundation considered, K_θ is defined by [29]

$$K_\theta = \frac{16}{3} GR^3 (\alpha_\theta + i a_o \beta_\theta) \tag{41}$$

in which α_θ and β_θ are dimensionless functions of the frequency parameter $a_o = \omega R/v_s$. On making use of the expression $v_s = \sqrt{G/\rho}$, the dimensionless stiffness factor, κ_θ, in Eq. 37 may also be written as

$$\kappa_\theta = \frac{16}{3} \frac{\rho R^5}{J} (\alpha_\theta + i a_o \beta_\theta) \tag{42}$$

C.2 Laterally Excited System

Let $x = x(t)$ and $x_f = x_f(t)$ be the lateral components of the foundation input displacement and actual foundation displacement, respectively, and $\phi(t)$ be the angular rocking displacement of the foundation. Further, let $u = u(t)$ be the resulting lateral deformation of the structure. The equations of motion of the system may then be expressed as

$$\ddot{u} + 2\zeta_x p_x \dot{u} + p_x^2 u = -(\ddot{x}_f + h\ddot{\phi}) \tag{43}$$

$$m_f \ddot{x}_f + m(\ddot{x}_f + h\ddot{\phi} + \ddot{u}) + Q_x(t) = 0 \tag{44}$$

$$I_T \ddot{\phi} + mh(\ddot{x}_f + h\ddot{\phi} + \ddot{u}) + Q_\phi(t) = 0 \tag{45}$$

in which I_T = the total mass moment of inertia of the structure and its foundation about a horizontal axis through the centroid of the foundation, and $Q_x(t)$ and $Q_\phi(t)$ are the horizontal shear and overturning moment at the foundation-soil interface. Equation 43 expresses the dynamic equilibrium of the forces acting on the structural mass, whereas Eqs. 44 and 45 express the equality of the interface forces to the total horizontal force and the base moment induced by the inertia forces acting on the structure and its foundation.

For the harmonic response considered,

$$x(t) = X_o \, e^{i\omega t} \tag{46a}$$

$$x_f(t) = X_f \, e^{i\omega t} \tag{46b}$$

$$\phi(t) = \Phi \, e^{i\omega t} \tag{46c}$$

$$u(t) = U \, e^{i\omega t} \tag{46d}$$

and $$\begin{Bmatrix} Q_x(t) \\ Q_\phi(t) \end{Bmatrix} = \begin{bmatrix} & K_f & \end{bmatrix} \begin{Bmatrix} X_f - X_o \\ \Phi \end{Bmatrix} e^{i\omega t} \tag{47}$$

in which $[K_f]$ = a 2x2 complex-valued impedance matrix for the massless foundation.

The solution of Eqs. 43 through 45 may be obtained in three steps in a manner analogous to that described for the torsionally excited system. First, Eq. 43 is solved for U in terms of $\ddot{X}_f + h\ddot{\Phi}$, in which $\ddot{X}_f = -\omega^2 X_f$ and $\ddot{\Phi} = -\omega^2 \Phi$. Second, the quantity U is eliminated from Eqs. 44 and 45 utilizing the result of the first step, and the resulting equations are solved for $\ddot{X}_f$ and $\ddot{\Phi}$ by making use of Eq. 47. Finally, the expressions for $\ddot{X}_f$ and $\ddot{\Phi}$ are back substituted in the expression for U obtained in the first step.

Implementation of the first step leads to

$$U = H_u(\ddot{X}_f + h\,\ddot{\Phi}) \tag{48}$$

in which H_u is defined by Eq. 15; and implementation of the second step leads to the following system of algebraic equations in $\ddot{X}_f$ and $\ddot{\Phi}$

$$\begin{bmatrix} m(TR)_x + m_f & mh(TR)_x \\ mh(TR)_x & I_T + mh^2(TR)_x \end{bmatrix} \begin{Bmatrix} \ddot{X}_f \\ \ddot{\Phi} \end{Bmatrix} - \frac{1}{\omega^2} \begin{bmatrix} & K_f & \end{bmatrix} \begin{Bmatrix} \ddot{X}_f \\ \ddot{\Phi} \end{Bmatrix} = \frac{1}{\omega^2} \begin{bmatrix} & K_f & \end{bmatrix} \begin{Bmatrix} \ddot{X}_o \\ 0 \end{Bmatrix} \tag{49}$$

in which $(TR)_x$ = the transmissibility factor for lateral response, defined by

$$(TR)_x = -(p_x^2 + i2\zeta_x \omega p_x)H_u = \frac{1 + i2\zeta_x(\omega/p_x)}{1 - (\omega/p_x)^2 + i2\zeta_x(\omega/p_x)} \quad (50)$$

On solving Eqs. 49 and substituting the resulting values of $\ddot{X}_f$ and $\ddot{\Phi}$ into Eq. 48, one obtains the desired U.

For the solutions presented in this report, the off-diagonal terms of $[K_f]$ are presumed to be negligible, and the diagonal terms are denoted by K_x and K_ϕ. On letting $\varepsilon_i = I_T/mh^2$, $\varepsilon_m = m_f/m$, $\kappa_x = K_x R^2/(mv_s^2)$ and $\kappa_\phi = K_\phi R^2/(mh^2 v_s^2)$, the solution for U may be expressed in the form

$$U = H_u T_u \ddot{X}_o \quad (51)$$

in which $\ddot{X}_o = -\omega^2 X_o$; and T_u = the dimensionless factor that provides for the inertial interaction effects. The latter factor is given by

$$T_u = \frac{B_3}{B_1 a_o^4 - B_2 a_o^2 + B_3} \quad (52)$$

in which

$$B_1 = (\varepsilon_i + \varepsilon_m)(TR)_x + \varepsilon_i \varepsilon_m \quad (53a)$$

$$B_2 = (\kappa_x + \kappa_\phi)(TR)_x + \varepsilon_m \kappa_\phi \quad (53b)$$

$$B_3 = \kappa_x(\kappa_\phi - \varepsilon_i a_o^2) \quad (53c)$$

For the circular foundations considered, the expressions for K_x and K_ϕ are given by [31]

$$K_x = \frac{8GR}{(2 - \nu)} (\alpha_x + i a_o \beta_x) \quad (54)$$

$$K_\phi = \frac{8GR^3}{3(1 - \nu)} (\alpha_\phi + i a_o \beta_\phi) \quad (55)$$

in which α_x, β_x, α_ϕ and β_ϕ are dimensionless factors that depend on Poisson's ratio for the halfspace material, ν, and the dimensionless frequency parameter, a_o. On making use of the expression $v_s = \sqrt{G/\rho}$, the stiffness factors κ_x and κ_ϕ in Eqs. 53 may be written as

$$\kappa_x = \frac{8}{2-\nu} \frac{\rho R^3}{m} (\alpha_x + i a_o \beta_x) \tag{56}$$

$$\kappa_\phi = \frac{8}{3(1-\nu)} \frac{\rho R^5}{mh^2} (\alpha_\phi + i a_o \beta_\phi) \tag{57}$$

The psd function for the deformation of the interacting system, S_u, is then given by

$$S_u = |H_u|^2 |T_u|^2 S_{\ddot{x}} \tag{58}$$

APPENDIX D

NOTATION

The following symbols are used:

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

$\tilde{a}_o$ = modified frequency parameter for combined wave passage and incoherence effects, defined by Eq. (11);

A = foundation contact area;

A_x = $p_x^2 U_x$ = pseudo-acceleration value of the mean maximum deformation induced by the lateral component of the foundation input motion;

A_y = $p_\theta^2 U_y$ = pseudo-acceleration value of the mean maximum deformation induced along the perimeter of the structure by the torsional component of the foundation input motion;

B_1, B_2, B_3 = dimensionless parameters in expression for T_u, defined by Eqs. 53;

b_o = γa_o = modified frequency parameter for incoherence only;

c = $v_s/(\sin\alpha_v)$ = apparent horizontal velocity of wave front;

c_o = $(v_s/c)a_o$ = $(\sin\alpha_v)a_o$ = modified frequency parameter for wave passage effect only;

d_1, d_2 = components of $\vec{r}_1$ and $\vec{r}_2$ in the direction of propagation of the seismic wave front;

f_o = cut-off frequency of excitation;

$f_x, \tilde{f}_x$ = natural frequencies of rigidly and elastically supported structures in lateral mode of vibration, in cps;

$f_\theta, \tilde{f}_\theta$ = natural frequencies of rigidly and elastically supported structures in torsional mode of vibration, in cps;

$G(\omega)$ = one side-power spectral density function for a stationary Gaussian random process;

h = height of structure;

H_u, H_v = transfer functions relating the lateral and torsional deformations of the structure to the corresponding components of the foundation input acceleration;

i = $\sqrt{-1}$;

I_o, I_1, I_2 = modified Bessel functions of the first kind of order zero, one and two, respectively;

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

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

J_1, J_2 = Bessel functions of first kind of order one and two, respectively;

J, J_f = polar mass moments of inertia of structure and foundation about a vertical centroidal axis;

K_x, K_ϕ, K_θ = complex-valued foundation impedances of massless foundations in lateral, rocking and torsional modes of vibration, respectively;

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

psd = power spectral density;

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;

q = Vanmarcke's bandwidth parameter;

Q_x, Q_ϕ, Q_θ = lateral force, overturning moment and torsional moment at foundation-soil interface;

$\vec{r}_1, \vec{r}_2$ = positon 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_o = constant in expression for psd function of the free-field ground acceleration;

$S_g, S_{\dot{g}}, S_{\ddot{g}}$ = local psd functions for the displacement, velocity and acceleration histories of the free-field ground motion;

$S_x, S_{\dot{x}}, S_{\ddot{x}}$ = psd functions for the displacement, velocity and acceleration histories of the lateral component of foundation input motion;

$S_y, S_{\dot{y}}, S_{\ddot{y}}$ = psd functions for the displacement, velocity and acceleration histories of the motion along the perimeter of the foundation induced by the torsional component of foundation input motion;

$S_{xy}, S_{\ddot{x}\ddot{y}}$ = cross psd functions for the horizontal and torsional components of the foundation input displacement and foundation input acceleration, respectively;

S_u = psd function for the structural deformation induced by the lateral component of foundation input motion;

S_v = psd function for the deformation $v = \psi R$ induced at the periphery of the structure by the torsional component of foundation input motion;

S_w = psd function for the total deformation at the most highly stressed point on the periphery of the structure;

t_o = duration of strong motion portion of earthquake;

T_u, T_v = dimensionless transfer factors that provide for the effects of inertial interaction for laterally and torsionally excited systems;

$(TR)_x$ = transmissibility of laterally excited system defined by Eq. 50;

$(TR)_\theta$ = transmissibility of torsionally excited system defined by Eq. 37;

U_x = mean value of maximum structural deformations induced by the lateral component of foundation input motion;

U_y = mean value of maximum deformations induced along the perimeter of the structure by the torsional component of foundation input motion;

v = ψR = structural deformation induced along the perimeter of the structure by the torsional component of foundation input motion;

v_s = shear wave velocity for soil medium;

V_x = $p_x U_x$ = pseudo-velocity value corresponding to U_x;

V_y = $p_y U_y$ = pseudo-velocity value corresponding to U_y;

w = $u + v$ = total deformation at the most highly stressed point at the periphery of the structure;

x = lateral component of foundation input displacement;

x_f = lateral component of actual foundation displacement;

$X, \dot{X}, \ddot{X}$ = mean maximum values of the horizontal components of the displacement, velocity and acceleration histories of the foundation input motion;

$X_g, \dot{X}_g, \ddot{X}_g$ = mean maximum values of the displacement, velocity and acceleration histories of the free-field, control-point ground motion;

X_o = amplitude of x for harmonic motion;

$Y, \dot{Y}, \ddot{Y}$ = mean maximum values of the displacement, velocity and acceleration at the periphery of the foundation induced by the torsional component of foundation input motion;

α_v = angle of incidence of seismic waves, measured from vertical axis;

$\alpha_x, \alpha_\phi, \alpha_\theta$ = dimensionless stiffness coefficients in expressions for foundation impedances K_x, K_ϕ and K_θ;

$\beta_x, \beta_\phi, \beta_\theta$ = dimensionless damping coefficients in expressions for foundation impedances K_x, K_ϕ and K_θ;

γ = dimensionless incoherence parameter;

$\tilde{\gamma}$ = $\gamma c/v_s$ = modified incoherence parameter;

Γ = spatial coherence function for free-field ground motion;

δ = mass density ratio for the structure;

Δ_1, Δ_2 = dimensionless distance parameters, defined by Eqs. 6;

ε_i = I_T/mh^2 = dimensionless measure of mass moment of inertia of structure-foundation system about a horizontal centroidal axis;

ε_m = m_f/m = mass ratio of foundation and structure;

ζ_x, ζ_θ = percentages of critical structural damping for fixed-base structure in lateral and torsional modes of vibration, respectively;

$\tilde{\zeta}_x, \tilde{\zeta}_\theta$ = effective structural damping factors for elastically supported system in lateral and torsional modes of vibration, respectively;

θ_1, θ_2 = circumferential co-ordinates of points $\vec{r}_1$ and $\vec{r}_2$, respectively;

κ_x = $K_x R^2/(m v_s^2)$ = dimensionless measure of lateral foundation impedance;

κ_ϕ = $K_\theta R^2/(J v_s^2)$ = dimensionless measure of torsional foundation impedance;

κ_ϕ = $K_\phi R^2/(m h^2 v_s^2)$ = dimensionless measure of rocking foundation impedance;

λ_n = nth moment of one-sided power spectral density, given by Eq. 27;

μ = mean rate of zero crossings for stationary process;

μ_e = effective mean rate of zero crossings;

ν = Poisson's ratio of soil medium;

ξ_1, ξ_2 = normalized radial coordinates of points $\vec{r}_1$ and $\vec{r}_2$, respectively;

ρ = mass density of soil medium;

τ = R/v_s = transit time;

$\tilde{\tau}$ = $\sqrt{\gamma^2 + \sin^2\alpha_v}\;\tau$ = effective transit time.

ϕ = actual rocking displacement of foundation;

Φ = complex-valued amplitude of ϕ for harmonic motion;

χ = torsional component of foundation input motion;

X = complex-valued amplitude of χ for harmonic motion;

χ_f = torsional component of actual foundation of displacement;

X_f = complex-valued amplitude of χ_f for harmonic motion;

ψ = torsional deformation of structure;

Ψ = complex-valued amplitude of ψ for harmonic motion;

ω = circular frequency of excitation and resulting motion.

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