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State-of-the Art Report
DESIGN APPROACHES FOR SOIL-STRUCTURE INTERACTION

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SUMMARY

After defining the meaning of the terms kinematic and inertial interaction, information and concepts are presented with the aid of which the effects of these actions may be approximated readily in design.

INTRODUCTION

It is generally recognized that the motion that is experienced by the foundation of a structure during an earthquake may be substantially different from the free-field ground motion, which is the motion that the ground would experience at its interface with the foundation in the absence of the structure. Two factors are responsible for this difference: (1) The inability of a rigid foundation to conform to the generally non-uniform, spatially varying, free-field ground motion; and (2) the interaction or coupling between the vibrating structure, its foundation, and supporting soils.

Several factors contribute to the spatial variability of the free-field ground motion. 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 that 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.

The seismic response of a structure is frequently evaluated considering the motion of its base to be equal to the stipulated free-field ground motion at a convenient reference or control point, normally taken at the ground surface. No provision is made in this approach for either the spatial variability of the free-field ground motion or for the properties of the supporting medium. The exact analysis requires that the structure be considered to be part of a larger system which includes the foundation and the supporting medium, and that due cognizance be taken of the spatial variability of the ground motion and of the properties of the soils involved.

Such an analysis is implemented in two steps: First, the motion of the foundation is evaluated considering both the foundation and the superimposed structure to be massless. Referred to as the foundation input motion (FIM), the resulting motion generally includes torsional and rocking components in addition to translational components. Next, the response of the actual foundation-structure system with mass to the FIM is evaluated using the actual properties of the supporting medium and providing for the dynamic interaction between its elements. The flexibility of the supporting medium has a two-fold effect: (1) It increases the number of degrees of freedom of the system and lowers its effective stiffness; and (2) it makes it possible for part of the vibrational energy of the structure to be dissipated in the supporting medium by radiation of waves and by hysteretic action in the soil itself. These forms of energy dissipation have no counterpart in a rigidly supported structure.

The difference in the responses of the superstructure computed for the FIM and the free-field control point motion (CPM) represents the kinematic interaction (KI) effect, whereas the difference of the responses computed with and without regard for the flexibility of the supporting medium is known as the inertial interaction (II) effect. The total soil-structure interaction (SSI) is given by the sum of the KI and II effects. More specifically, if $R_o(\text{FIM})$ = the response to the FIM computed considering the supporting medium to be rigid, and $R_o(\text{CPM})$ = the corresponding response to the CPM, then

$$KI = R_o(\text{FIM}) - R_o(\text{CPM}) \tag{1}$$

Similarly, if R(FIM) = the response to the FIM computed with due regard for the flexibility of the supporting medium, then

$$II = R(\text{FIM}) - R_o(\text{FIM}) \tag{2}$$

and $$SSI = KI + II = R(\text{FIM}) - R_o(\text{CPM}) \tag{3}$$

The objectives of this presentation are: (1) To highlight the nature and relative importance of the kinematic and inertial interaction effects; and (2) to present information and concepts with which these effects may be estimated and provided for readily in design. Consideration will first be given to the inertial interaction effects, which have been examined previously in Refs. 1 to 3.

SYSTEM CONSIDERED

The concepts involved will be identified by reference to a simple linear structure of mass m and height h, which is supported through a foundation of mass m_o at the surface of a homogeneous elastic halfspace. The circular natural frequencies of lateral and torsional modes of vibration of the fixed-base structure 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 β_x and β_θ. 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_o 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 control point motion is defined at the center of the foundation-soil interface, and it is considered to be a uni-directional, horizontal excitation.

INERTIAL INTERACTION EFFECTS

First, the effects of a vertically propagating plane seismic wave are examined. The FIM in this case is equal to the CPM, and only inertial interaction effects are present. Under the influence of such an excitation, the foundation of the structure displaces horizontally by an amount x(t) which is generally different from $x_g(t)$, and rocks or rotates about a horizontal axis by an amount $\psi(t)$. The configuration of the system can then be defined by x(t), $\psi(t)$ and the structural deformation, u(t). The rocking component of foundation motion would be expected to be particularly prominent for tall structures and flexible soils. For a rigidly supported structure, $\psi(t) = 0$ and $x(t) = x_g(t)$.

The factors complicating the analysis of this system and the available methods for evaluating its response have been identified in Ref. 1 and will not be repeated here. Emphasis will instead be placed on simple approximate procedures that are convenient for design applications. Two such procedures have been proposed. The first involves modifying the stipulated free-field ground motion and evaluating the response of the structure to the modified motion of the foundation, whereas the second involves modifying the dynamic properties of the structure, considering it to be rigidly supported, and evaluating the response of the modified structure to the prescribed free-field ground motion. The second approach, which permits the direct use of response spectra for the specified free-field CPM, is the more convenient and will be used exclusively here.

Principal Effects The interaction effects in the latter approach are expressed approximately by an increase in the fixed-base natural period of the structure, and by a change (generally an increase) in the associated damping. The increase in period results from the flexibility of the supporting medium, whereas the increase in damping results from the capacity of the medium to dissipate energy by radiation of waves and by hysteretic action.

If T represents the natural period of the structure in its fixed-base condition, and $\tilde{T}$ represents the period of the modified structure which approximates the flexibly supported system, it can be shown (e.g., Ref. 1) that

$$\tilde{T} = T \sqrt{1 + \frac{k}{K_x}\left(1 + \frac{K_x h^2}{K_\psi}\right)} \tag{4}$$

in which K_x = the lateral translational stiffness of the foundation, and K_ψ = the corresponding rocking stiffness. Strictly speaking, these stiffnesses should be evaluated for a harmonic excitation of a period $\tilde{T}$. However, reasonable approximations are obtained by use of their static values, or better still, of the values corresponding to the fixed-base natural period of the system, T.

It can further be shown that if β represents the percentage of critical damping for the fixed-base structure, and $\tilde{\beta}$ represents the corresponding damping of the modified structure that approximates the interacting system, the two quantities are interrelated approximately by

$$\tilde{\beta} = \beta_o + \frac{\beta}{(\tilde{T}/T)^3} \tag{5}$$

in which β_o represents the contribution of the foundation damping, including radiation and soil material damping. Note that β_o and β are not directly additive, and that the effectiveness of the structural damping is reduced by soil-structure interaction, the reduction being substantial when $\tilde{T}/T$ is large. In fact, unless the reduced contribution of structural damping is compensated by the foundation damping, the overall damping of the interacting system will be less than that of the rigidly supported structure.

The three most important parameters that affect the value of β_o are: the period ratio, $\tilde{T}/T$, which is a measure of the relative flexibilities of the foundation medium and structure; the ratio of the height of the structure to the radius of the foundation, h/R; and the hysteretic capacity of the soil itself, defined by the factor

$$\tan \delta = \frac{1}{2\pi} \frac{\Delta W_s}{W_s} \tag{6}$$

in which ΔW_s represents the area of the hysteresis loop in the stress-strain diagram for a soil specimen undergoing harmonic shearing deformation, and W_s represents the strain energy stored in a linearly elastic material subjected to the same maximum values of stress and strain (i.e., the area of the triangle in the stress-strain diagram between the origin and the point of the maximum induced stress and strain). This factor depends on the magnitude of the imposed peak strain, increasing with increasing intensity of excitation or level of straining.

The variation of β_o with $\tilde{T}/T$ is shown in Fig. 1 for two values of $\tan\delta$. The dashed lines, which refer to systems supported on a purely elastic medium, represent the effect of radiation damping only, whereas the solid lines, which refer to a viscoelastic medium with $\tan\delta = 0.10$, represent the combined effect of radiation and hysteretic soil action. It can be seen that the contribution of the foundation damping may be quite substantial for relatively short, stubby structures, and that the effect of hysteretic soil action may be particularly significant for tall structures for which the radiational effects are generally quite small.

The particular data presented in Fig. 1 are for systems with negligible foundation mass and a structural mass equal to 15 percent of the mass of the structure when filled with soil. The latter value is representative of that for building structures. Additional data are available in Ref. 1.

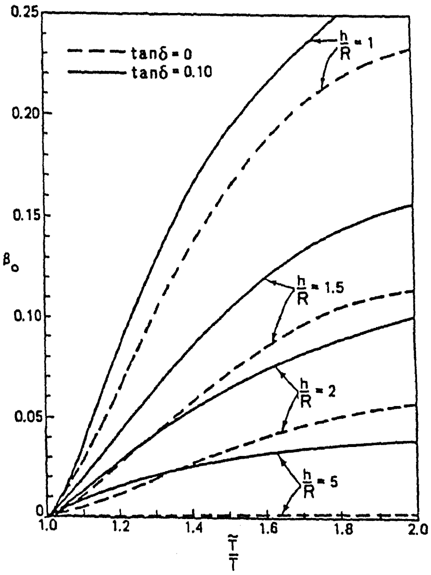


Fig. 1 Foundation Damping Factor

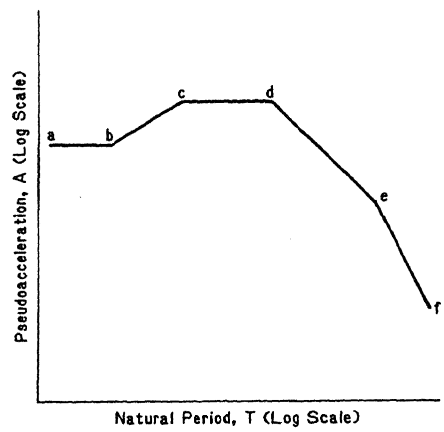


Fig. 2 Design Response Spectrum

Summary of Procedure The steps involved may be summarized as follows: First, compute the fixed-base natural period of the structure, T, and by application of Eq. 4, evaluate the modified natural period, $\tilde{T}$. Next, estimate the structural damping factor, β, and the value of $\tan \delta$ which would be appropriate for the anticipated strains in the supporting medium, and by application of Eq. 5 and of plots such as those presented in Fig. 1, determine the effective damping factor, $\tilde{\beta}$. From the response spectrum for single-degree-of-freedom systems subjected to the stipulated free-field ground motion, evaluate then the pseudo-acceleration, $\tilde{A}$, corresponding to $\tilde{T}$ and $\tilde{\beta}$. The maximum value of the base shear for the interacting system, $\tilde{Q}$, is finally determined from

$$\tilde{Q} = m\tilde{A} \tag{7}$$

and the maximum displacement of the structure relative to the moving ground, $\tilde{\Delta}$, is determined from

$$\tilde{\Delta} = \frac{\tilde{Q}}{k} + \frac{\tilde{Q}\,h^2}{K_\psi} \tag{8}$$

in which the term on the extreme right represents the contribution of the foundation rotation.

For building structures that must be analyzed as multi-degree-of-freedom systems in their fixed-base condition, a reasonable approximation to the maximum response of a structure may be obtained by assuming that soil-structure interaction affects only the response component contributed by the fundamental mode of vibration. This component is computed by the procedure outlined, by interpreting m and h to be the effective mass and effective height of the structure when vibrating in its fundamental mode. The contributions of the higher modes are then computed disregarding the interaction effects. The rationale for this approach is explained in Ref. 1.

The concepts summarized herein have provided the basis of the design provisions for soil-structure interaction for building structures that have been recommended by the Applied Technology Council (Refs. 2,3,4) and have been adopted recently by the Building Seismic Safety Council (Ref. 5) in connection with the National Earthquake Hazard Reduction Program (NEHRP).

Nature of Effects A recurring question is whether SSI increases or decreases the maximum response of a structure. The answer is a function of the response quantity under examination, and of the characteristics of the ground motion and the system itself. More specifically, SSI may increase, decrease, or have no effect on the peak response of a system depending on the characteristics of the relevant response spectrum, and the regions of the spectrum to which the fundamental natural periods of the fixed-base and the interacting systems fall.

The various possibilities are illustrated in the following paragraphs by reference to the base shear induced by a free-field ground motion the pseudo-acceleration spectrum of which is represented by the piecewise linear diagram shown in Fig. 2.

1. If both T and $\tilde{T}$ fall in the extremely small period range of the spectrum (to the left of point b), SSI will have no effect on the response, as the pseudo-acceleration value in this case is unaffected by changes in either period or damping.

2. If T falls to the right of point c, SSI will reduce the maximum response, the reduction being a function of the values of T, $\tilde{T}$, β and $\tilde{\beta}$. An increase in damping under these conditions decreases the pseudo-acceleration, whereas an increase in period either does not change it or further decreases it.

3. If T falls in the intermediate period range (between points b and c in Fig. 2), or if T falls to the left of point b and $\tilde{T}$ falls to the right of this point, SSI may increase or decrease the response depending on the values of $\tilde{T}/T$ and $\tilde{\beta}/\beta$. An increase in period in this case increases the response, whereas an increase in damping has the opposite effect.

The pseudo-acceleration response spectrum for fixed-base systems in the ATC-NEHRP design provisions is a non-increasing function of the fundamental natural period of the system. As a result, consideration of soil-structure interaction reduces the design values of the lateral forces, shears and overturning moments below the levels applicable to a rigid-base condition, and it is conservative in this case to neglect the interaction effects. Because of the influence of foundation rocking, however, the horizontal displacement of the structure relative to the moving base may increase due to interaction, and this will, in turn, increase both the required spacing between buildings and the secondary design forces associated with the P-Δ effects. The latter increases are generally small and have only a minor influence on the final design.

KINEMATIC INTERACTION EFFECTS

Effects of Wave Passage The structure in this section is presumed to be excited by a horizontally polarized plane shear wave propagating in a direction that makes an angle α with the vertical. In addition to a horizontal displacement component, $x_i(t)$, the FIM in this case includes a torsional component, $\theta_i(t)$, about a vertical axis. These displacements refer to the foundation motion of the massless foundation-structure system, and should not be confused with those of the actual system with mass, $x(t)$ and $\theta(t)$. For convenience, the torsional component of the FIM will be expressed by the circumferential displacement along the periphery of the foundation, $y_i(t) = R\,\theta_i(t)$. The absolute maximum values of these displacements will be denoted by x_i, θ_i and y_i, and of the associated velocities and accelerations will be identified by one and two dot superscripts, respectively.

If use is made of the averaging technique of Iguchi (Ref. 6) and Scanlan (Ref. 7), in which the restraining action of the supporting medium is effectively represented by a series of Winkler springs, it can be demonstrated that the interrelationship of the FIM and the free-field CPM is defined by the time τ^* required for the wave front to traverse the foundation radius. Referred to as the effective transit time, this quantity is given by

$$\tau^* = \frac{R}{c} = \frac{R}{v_s} \sin\alpha = \tau \sin\alpha \tag{9}$$

in which v_s = the velocity of shear wave propagation in the medium, $c = v_s/\sin\alpha$ = the apparent horizontal velocity of wave propagation, and $\tau = R/V_s$. For a harmonic motion of circular frequency ω, the parameters ω and τ^* appear in the solution as a product, and the foundation input motion in this case is defined by the dimensionless frequency parameter

$$a_o^* = \omega\tau^* = a_o \sin\alpha \tag{10}$$

in which $a_o = \omega R/v_s$ = the well known dimensionless frequency parameter used in studies of foundation dynamics. For $\tau^* = a_o^* = 0$, all points of the foundation are excited simultaneously, the FIM = CPM, and there is no kinematic interaction.

Foundation Input Motion. The effect of a_o^* on the amplitudes of FIM for harmonically excited systems is displayed in part (a) of Fig. 3. The results are normalized with respect to the amplitude of the free-field CPM. As would be expected, the horizontal component of the FIM generally decreases with increasing

a^*_o, the reduction being particularly significant for values of a^*_o of the order of 2 or greater. The torsional component of the FIM, on the other hand, first increases and then decreases.

For the harmonic motion considered, the temporal variations of its displacement, velocity and acceleration traces are identical, and the displacement ratios in Fig. 3(a) may also be interpreted as velocity or acceleration ratios. By contrast, for an actual earthquake ground motion, for which the frequency contents of the acceleration, velocity and displacement traces are different, the peak displacements of the FIM will be related to the peak displacement of the CPM differently from the corresponding velocity and acceleration values. This is demonstrated in part (b) of Fig. 3 which shows the results obtained for the CPM corresponding to the first 6.24 sec. of the N-S component of the 1940 El Centro, California earthquake. The acceleration, velocity and displacement traces of this record are available in Ref. 8.

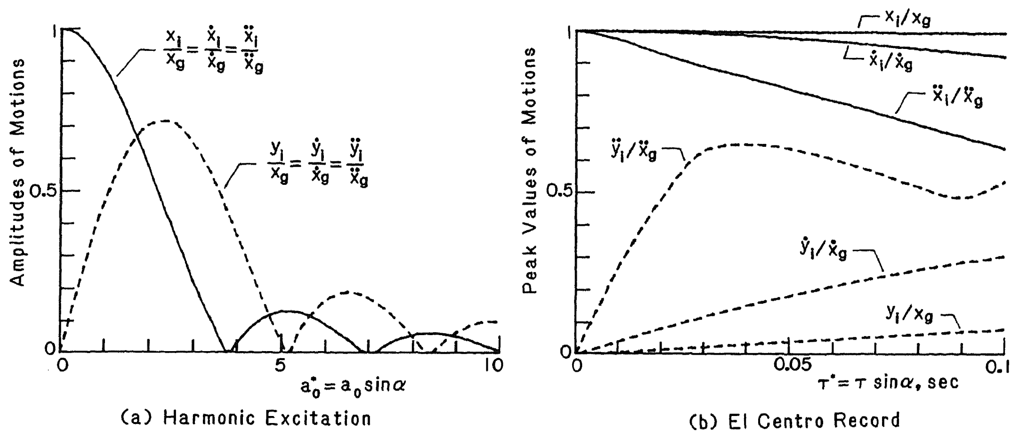


Fig. 3 Peak Foundation Input Motions for Obliquely Incident Plane Shear Waves

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

1. The reduction in the horizontal component of the foundation input motion and the corresponding increase in the torsional 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 high-frequency systems are acceleration-sensitive whereas low-frequency systems are displacement-sensitive, it should be clear that the effects of kinematic interaction on the lateral component of response 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 next section.

Structural Response. Let u_x be the absolute maximum value of the structural deformation induced by the lateral component of the FIM, and u_y be the corresponding deformation induced at the periphery of the deck by the torsional component. Fig. 4 presents response spectra for these deformations considering damping factors $\beta_x = \beta_\theta = 0.02$. The results are displayed in logarithmic scales, with the abscissa representing the natural frequency of the system for the particular mode of vibration considered, and the ordinates representing the pseudo-velocity values, $V_x = p_x u_x$ and $V_y = p_\theta u_y$, normalized with respect to the peak velocity of the CPM, $\dot{x}_g$. Three different values of τ^* are considered, including the limiting value of $\tau^* = 0$ for which there is no kinematic interaction.

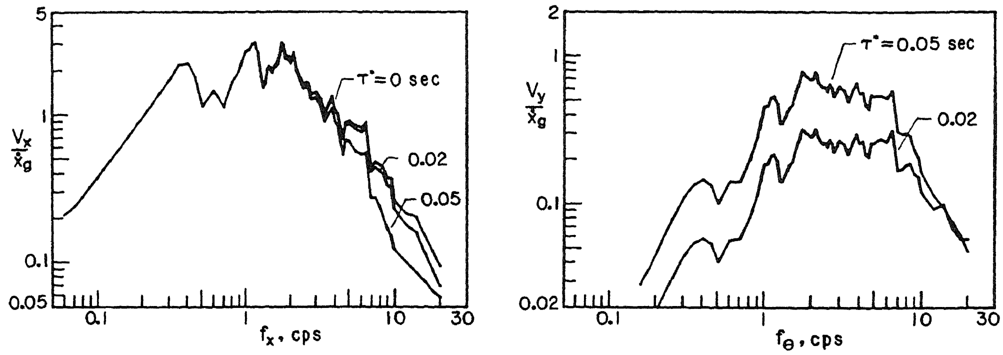


Fig. 4 Effect of Wave Passage on Maximum Lateral and Torsional Deformations

As already anticipated, the effect of wave-passage on the lateral component of the response is greatest for high-frequency systems, significantly smaller for medium frequency systems, and negligible for low-frequency systems. Furthermore, both the magnitudes and the trends of the response spectra for torsional response are consistent with those expected from the torsional components of the FIM presented in Fig. 3(b).

Effects of Ground Motion Incoherence The free-field ground motion in this section is considered to be due to horizontally polarized, vertically propagating, incoherent shear wave trains. Purely horizontal but differing from point to point of the foundation-soil interface, the motion in this case cannot at the present state of knowledge be specified in deterministic terms. It is, instead, specified stochastically in terms of a local power spectral density (psd) function, $S_g(\omega)$, and a cross psd function, $S(\vec{r}_1, \vec{r}_2, \omega)$, in which ω = the circular frequency of the harmonic component of the motion under consideration; and $\vec{r}_1$ and $\vec{r}_2$ are position vectors for two arbitrary points on the foundation-soil interface. The cross psd function may be expressed as

$$S(\vec{r}_1, \vec{r}_2, \omega) = \Gamma(|\vec{r}_1 - \vec{r}_2|, \omega)\, S_g(\omega) \tag{11}$$

in which Γ, referred to as the incoherence function, is a dimensionless, decreasing function of ω and of the distance between points, $|\vec{r}_1 - \vec{r}_2|$. Several different expressions have been proposed for Γ, and there is no general agreement on the form that may be the most appropriate for realistic earthquakes. In this study, it is taken in the form recommended by Luco and Mita (Ref. 9) as

$$\Gamma(|\vec{r}_1 - \vec{r}_2|, \omega) = \exp\left[-\left(\frac{\gamma\omega|\vec{r}_1 - \vec{r}_2|}{v_s}\right)^2\right] \tag{12}$$

in which γ is a dimensionless factor with a value between zero and 0.5.

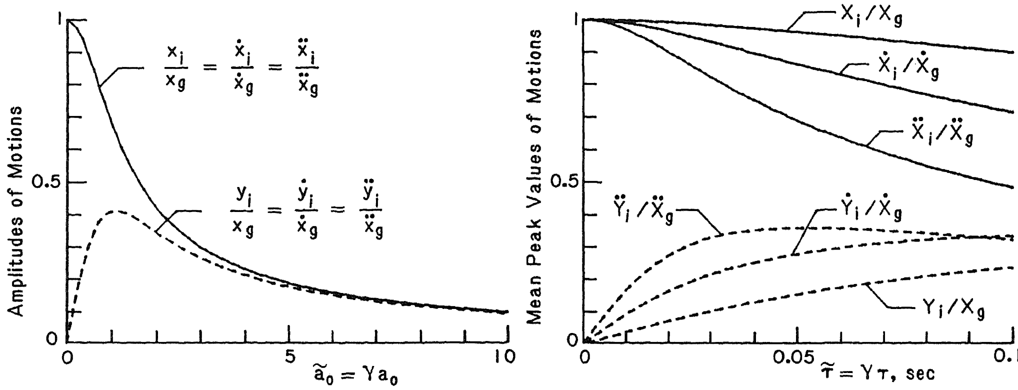
For a free-field harmonic motion of circular frequency ω and spatially varying amplitudes and phases, the interrelationship between the CPM, and FIM is defined by the dimensionless frequency parameter

$$\tilde{a}_o = \gamma a_o = \gamma \frac{\omega R}{v_s} \tag{13}$$

whereas for a transient motion of arbitrary temporal variation, it is defined by the dimensional effective transit time

$$\tilde{\tau} = \gamma \tau = \gamma \frac{R}{v_s} \tag{14}$$

These statements hold true under the same simplifying assumption as that employed in the study of wave propagation effects.



(a) Harmonic Excitation (b) Stochastic Excitation

Fig. 5 Peak Foundation Input Motions for Vertically Incident Incoherent Waves

In part (a) of Fig. 5 are shown the normalized amplitudes of the horizontal and torsional components of the FIM for harmonic free-field excitations, and in part (b) are given corresponding data for an ensemble of transient control point motions for which the psd function of the acceleration histories is of the form considered in Refs. 10 and 11. The cut-off frequency of the latter function is taken as 8 cps in the present study. The quantities X_i and Y_i in Fig. 5(b) represent the ensemble means of the absolute maximum displacement values of the horizontal and circumferential components of the FIM, and X_g represents the corresponding value of the CPM. One dot and two dot superscripts identify the mean peak values of the corresponding velocity and displacement histories. These results were computed by use of Der Kieureghian's empirical relations (Ref. 12) considering the duration of the intense portion of the excitation to be 20 sec.

The plots in Fig. 5 are similar to those presented in Fig. 3 for an obliquely incident plane wave, and the conclusions and inferences drawn from the previous plots also apply in this case. This is confirmed by the solid curves in Fig. 6, which represent response spectra for the ensemble means of the absolute maximum values of the lateral deformations, U_x, and the circumferential deformations, U_y, induced along the periphery of the structure by the torsional component of foundation input motion. The damping values in these solutions are taken as $\beta_x = \beta_\theta = 0.02$. As before, the results are expressed in terms of normalized pseudovelocity values, in which $\hat{V}_x = p_x U_x$ and $\hat{V}_y = p_\theta U_y$. The reductions in the lateral component of the response due to the incoherence of the ground motion and the general trends of the spectra for circumferential response are indeed fully compatible with the peak values of the FIM presented in Fig. 5(b).

Also shown in Fig. 6(a) by dashed lines are the mean peak values of the combination of lateral and torsional deformations along the periphery of structures with $f_\theta = 1.5 f_x$. The contribution of the torsional component of response is clearly small in this case.

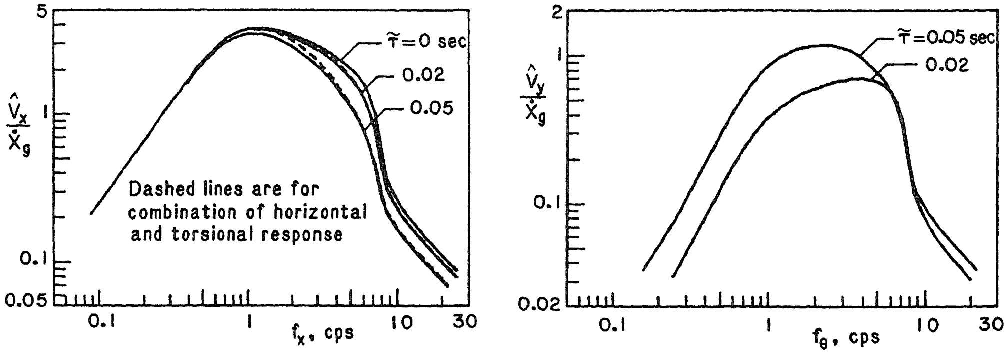


Fig. 6 Effect of Ground Motion Incoherence on Mean Peak Values of Lateral and Torsional Deformations

Design Implication of Results The principal step in the evaluation of the kinematic interaction effects is the computation of the foundation input motion for the massless structure-foundation system. For both plane and incoherent wave fields, these motions may be computed by Fourier transform techniques, making use of the foundation transfer functions presented in Figs. 3(a) and 5(a). Of particular importance are the peak (or mean peak) values of the acceleration, velocity and displacement histories of the motions. With these values established, the effects of kinematic interaction on the peak structural response may be approximated by application of well established rules relating the characteristics of response spectra to the peak acceleration, velocity and displacement values of the excitation. The lateral and torsional effects should be evaluated separately and then combined by the RMS rule, or an appropriate variant of it, giving due regard to the relative values of f_x and f_θ.

TOTAL SOIL-STRUCTURE INTERACTION EFFECTS

As a measure of the relative importance of kinematic and inertial interaction effects, in Figs. 7 and 8 are shown response spectra for the lateral and circumferential responses obtained for systems with a height to base radius ratio, h/R = 2. The plots in Fig. 7 refer to the obliquely incident plane wave

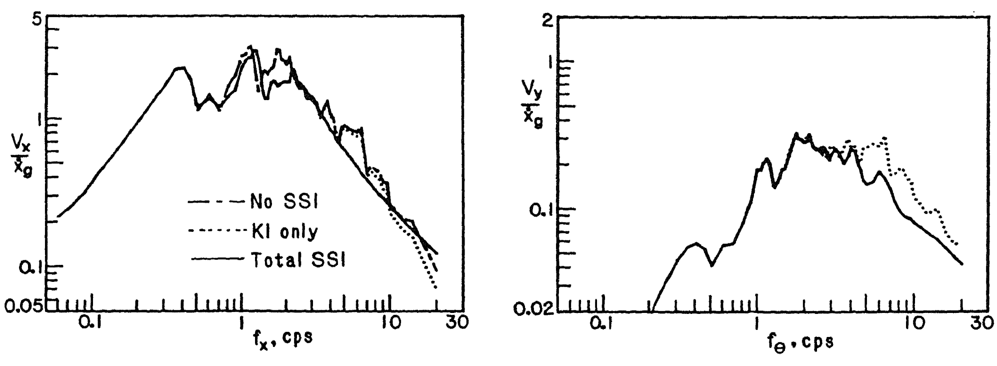


Fig. 7 Response Spectra for Interacting Structures with h/R = 2; Obliquely Incident El Centro Record, $\sin\alpha = 0.4$, $\tau = 0.05$ sec

considered in Figs. 3 and 4, whereas those in Fig. 8 are for the vertically propagating incoherent wave field considered in Figs. 5 and 6. The principal parameters for these solutions are identified on the figure headings, and the remaining parameters are the same as those for the solutions presented in Ref. 11.

Three sets of solutions are displayed in each case: (1) making no provision for soil-structure interaction, i.e., considering the foundation motion to be the same as the free-field control point motion; (2) providing only for the kinematic interaction effects, i.e., using as base excitation the foundation input motion and analyzing the superstructure without regard for the flexibility of the supporting medium; and (3) providing for both kinematic and inertial interaction effects, i.e., analyzing the structure-foundation soil system exactly as a coupled system. Also included in Fig. 8 are the results obtained by the simplified analysis presented in the early part of the paper. The base excitation in these solutions was taken equal to the foundation input motion and the natural frequency and damping of the system for each mode of vibration were modified appropriately. The details of the analysis are given in Ref. 11.

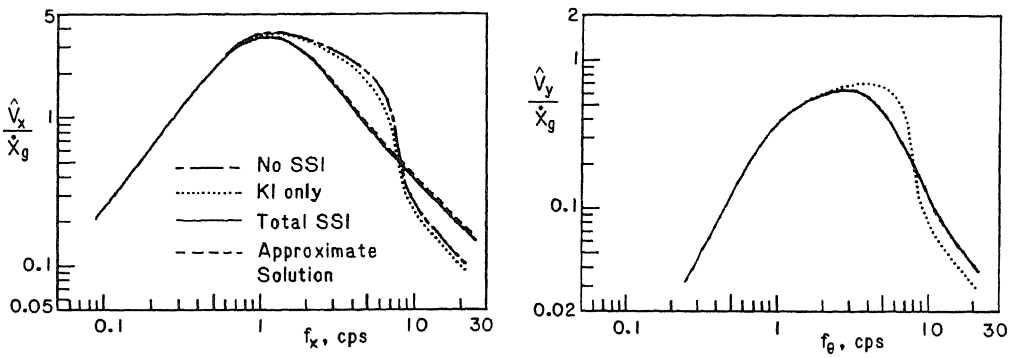


Fig. 8 Response Spectra for Interacting Structures with $h/R = 2$; Vertically Incident Incoherent Waves, $\gamma = 0.4$, $\tau = 0.05$ sec

From these data and additional solutions reported in Ref. 11, the following conclusions are drawn:

1. Like kinematic interaction, inertial interaction influences most the responses of systems in the medium- and high-frequency spectral regions.

2. The II effects are generally more important that the KI effects.

3. Unlike kinematic interaction which generally reduces the lateral response, inertial interaction may reduce or increase the corresponding response.

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

5. The concept of modifying the fixed-base natural frequencies are associated damping values of the system provides a simple and highly reliable means for assessing the inertial interaction effects.

CONCLUSION

The information and concepts presented herein provide simple practical means for assessing the effects of soil-structure interaction in the seismic design of structures. Although presented for relatively simple structures and somewhat idealized conditions, the concepts involved are applicable to more complex systems as well. As a matter of fact, these concepts have been used effectively in studies of a variety of structures, including buildings, nuclear containment structures, offshore structures, and liquid-containing tanks. However, there continues to be a need for much additional research. Topics requiring further study include the behavior of structures with embedded foundations for which the kinematic effects will be more important than for surface-supported foundations, the behavior of pile-supported structures, and the interaction effects for structures responding in the inelastic range of deformation.

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For additional references, see Ref. 11.