3 Buildings as Sources of Rotational Waves

Mihailo D. Trifunac

Department of Civil Engineering, University of Southern California
Los Angeles, California, USA, 90089-2531
e-mail: trifunac@usc.edu

3.1 Introduction

Most man-made structures are built above the ground. Single-family residential homes can be several tens of meters high, while modern steel skyscrapers can reach heights of several hundred meters. Supported asymmetrically at their base, with their center of gravity near mid-height, these structures undergo rocking motions when excited by earthquakes, strong winds, and man-made transient and steady excitations. Through the rocking compliance, the soil-structure interaction then acts as a mechanism for conversion of the incident wave energy into rotational motions of the foundation, which then radiates this wave energy back into the soil. During earthquake and ambient noise excitations, the incident waves are scattered and diffracted by the foundation-soil interface, and together with the waves generated by soil-structure interaction radiate rotational motions back into the soil. During wind and man-made excitation, a part of the wave energy in the building is converted into rotational excitation of the soil. The early work on the waves created by soil structure-interaction dates back to the 1930s (Sezawa and Kanai 1935, 1936) and 1940s (Biot 2006). Full-scale experiments of soil-structure interaction have provided data to measure and quantify the nature of the motions at the interface between the soil and the building foundations (Luco et al. 1986, Todorovska 2002, Trifunac and Todorovska 2001). The emphasis in these full-scale tests, so far, has been on the response of structures, and on how this response is affected by soil-structure interaction. Some experiments, however, did investigate the nature of the near-field deformation of soil surrounding the building foundation (Luco et al. 1975, 1988, Foutch et al. 1975, Wong et al. 1977a). It has been found that for stiff foundation-structure systems, the soil-foundation interaction can be approximated by a rigid foundation model having only six degrees of freedom. For flexible foundations (Trifunac et al. 1999) and multiple foundations, the soil deformation is far more complex, and the translational and rotational waves in the near field, radiated by the motion of the foundations, require complex three-dimensional analyses. In densely populated metropolitan areas where separation of adjacent buildings is small or negligible and there are building-soil-building interactions, and where long bridges have multiple supports resting on soil, detailed two- and three-dimensional analyses are required (Werner et al. 1979, Wong and Trifunac 1975). Analytical studies of two-dimensional soil-structure interaction (of long buildings on rigid foundations) have shown how the interference of the incident waves, and of the scattered waves from the foundation, can lead to nearly standing wave motions on the ground surface, which, at the nodes, result in strong torsional motions (Trifunac 1972, Trifunac et al. 2001c, Todorovska et al. 1988). Analytical studies of the response of three-dimensional models show amplification of the torsional response of building-foundation-soil systems and the radiation of torsional scattered waves for near-horizontal incidence of SH waves (Lee 1979). Studies of the wave passage effects around rigid embedded foundations have explained amplification of the rocking foundation motions and the more energetic radiation of rotational waves when half wave-lengths of the incident waves are comparable to the foundation width (Todorovska and Trifunac 1990, 1991, 1993). Observational and analytical studies of buildings in an urban setting have examined the site-city interaction (Boutin and Roussillon 2004, Gueguen et al. 2000, 2002, Kham et al. 2006, Tsogka and Wirgin 2003), and have interpreted the prolonged duration of strong ground motion in urban settings (Wirgin and Bard 1996) in terms of the waves delayed by prolonged paths up and down the buildings (Gicev 2005).

Experiments using forced vibration of full-scale structures have been used to investigate the wave motion in the far field radiated due to soil-structure interaction (Luco et al. 1975, Favela 2004). The radiated waves in the far field have been used as monochromatic sources of waves to investigate the relative significance of irregular topography and irregular geometry of sedimentary layers on amplification of surface displacements (Wong et al. 1977b).

In the following, the linear theory of soil-structure interaction will be illustrated, emphasizing the rotational aspects of motion, which result from (1) the presence of an inclusion (foundation) in the half space (Lee and Trifunac 1982), and (2) from the interaction with a structure (Lee 1979). A discussion of the non-linear aspects of this class of problems is beyond the scope of this chapter, but the reader may find introductory examples of analyses and observations in Gicev (2005) and Trifunac et al. (2001a,b).

3.2 Soil-Foundation Interaction – Near Field

The dynamic response of a rigid foundation embedded in an elastic medium to seismic waves can be separated into two parts. The first part corresponds to the determination of the restraining forces due to the rigid body motion of the inclusion. The second part deals with the evaluation of the driving forces due to the scattering of incident waves by the inclusion, which is presumed to be immobile.

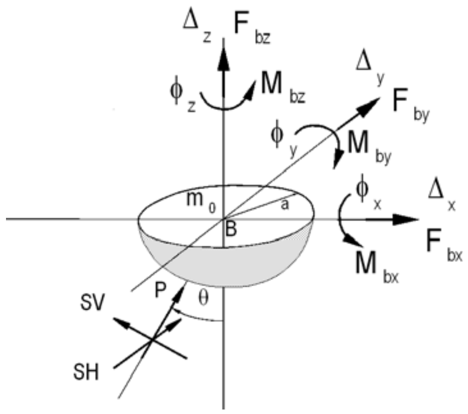

Fig. 3.1 Forces F_{bx}, F_{by}, F_{bz}, M_{bx}, M_{by}, M_{bz} acting on the foundation, and its displacements Δ_x, Δ_y, Δ_z, ϕ_x, ϕ_y, ϕ_z

Consider a foundation embedded in an elastic medium and supporting an elastic superstructure. The steady-state harmonic motion of the foundation having frequency ω can be described by a vector $\{\Delta_x, \Delta_y, \Delta_z, \phi_x, \phi_y, \phi_z\}^T$ (Fig. 3.1), where Δ_x and Δ_y are horizontal translations, Δ_z is vertical translation, ϕ_x and ϕ_y are rotations about horizontal axes, and ϕ_z is torsion about the vertical axis. Using superposition, displacement of the foundation is the sum of two displacements

$$\{U\} = \{U^*\} + \{U_0\}, \tag{3.1}$$

where $\{U^*\}$ is the foundation input motion corresponding to the displacement of the foundation under the action of the incident waves in the absence of external forces, and $\{U_0\}$ is the relative displacement corresponding to the displacement of the foundation under the action of the external forces in the absence of incident wave excitation.

The interaction force $\{F_s(\omega)\}\exp\{-i\omega t\}$ generates the relative displacement $\{U_0\}\exp\{-i\omega t\}$. It corresponds to the force that the foundation exerts on the soil, and it is related to $\{U_0\}$ by $\{F_s\} = [K_s(\omega)]\{U_0\}$, where $[K_s(\omega)]$

is the 6×6 complex stiffness matrix of the embedded foundation. It depends upon the material properties of the soil medium, the characteristics and shape of the foundation, and the frequency of the harmonic motion. It describes the force-displacement relationship between the rigid foundation and the soil medium.

The driving force of the incident waves is equal to

$$\left\{F_s^*\right\} = \left[K_s\right]\left\{U^*\right\}, \tag{3.2}$$

where the input motion $\{U^*\}$ is measured relative to an inertial frame. The "driving force" is the force that the ground exerts on the foundation when the rigid foundation is kept fixed under the action of incident waves. It depends upon the properties of the foundation and the soil and on the nature of excitation.

The displacement $\{U\}$ is then related to the interaction and driving forces via

$$\left[K_s\right]\left\{U\right\} = \left\{F_s\right\} + \left\{F_s^*\right\}.$$

For a rigid foundation having a mass matrix $[M_0]$ and subjected to external force, $\{F_{ext}\}\exp\{-i\omega t\}$, the dynamic equilibrium equation is

$$-\omega^2\left[M_0\right]\left\{U\right\} = -\left\{F_s\right\} + \left\{F_{ext}\right\}. \tag{3.3}$$

$\{F_{ext}\} = \{F_{bx}, F_{by}, F_{bz}, M_{bx}, M_{by}, M_{bz}\}$ is the force the structure exerts on the foundation (Fig. 3.1). Then Eq. (3.3) becomes

$$\left(-\omega^2[M_0] + [K_s]\right)\left\{U\right\} = \left\{F_s^*\right\} + \left\{F_{ext}\right\}. \tag{3.4}$$

The solution of $\{U\}$ requires the determination of the mass matrix, the impedance matrix, the driving forces and the external forces.

Foundation Response. The stiffness and damping coefficients associated with the real and imaginary parts of $[K_S]$ are the functions of dimensionless frequency η (Lee 1979). $\eta = \omega a/\pi\beta$ is the ratio of the diameter of the hemispherical foundation to the wavelength of the transverse (S) waves, λ, in the half space. η is also a dimensionless wave number $k_\beta a/\pi$. The range of η considered in the examples here will be from 0 to 1.

Incident SV wave (Fig. 3.1) excites vertical motion, horizontal translation, and rocking, $\{\Delta_x/a_0,\ 0,\ \Delta_z/a_0,\ a\phi_y/a_0,\ 0\}^T$. For the Poisson's ratio, $\nu = 0.25$, the critical angle of incidence is $\theta_{cr} = 35°16'$. The incidence angles $\theta = 0°$ and 30° are thus below the critical angle, while the angles of incidence $\theta = 60°$ and 85° are beyond the critical angle. The case of grazing incidence ($\theta = 90°$) will result in zero motion in the free field, and thus it is not considered here. In Fig. 3.2, the normalized amplitudes $|a\phi_y/a_0|$, where a is the radius of the foundation, and a_0 is the amplitude of incident waves, are plotted versus the dimensionless frequency η for different angles of incidence and for $m_0/m_S = 0$, 2, and 4; m_0 is the mass of the hemispherical foundation; and m_s is the mass of soil removed by the foundation. For vertical incidence, only the horizontal translation and rocking are excited, and there is no vertical motion. At low frequencies, the displacement amplitudes approach the limit of the free field displacement amplitudes.

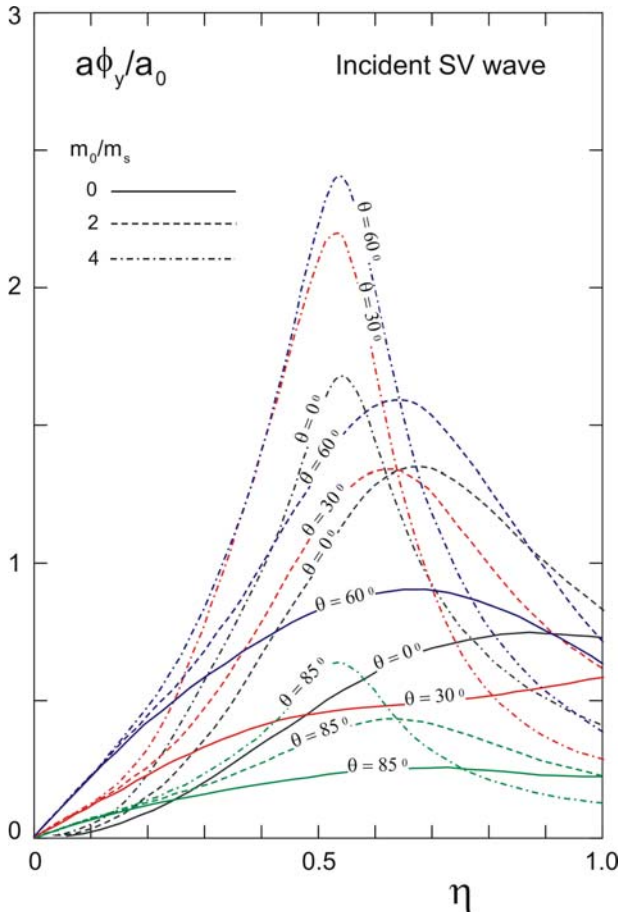

Fig. 3.2 Rocking ϕ_y of the foundation, excited by plane SV waves, with incident angles $\theta = 0°$, 30°, 60° and 85°, and dimensionless frequency η

The amplitudes gradually decrease with increasing η because of the embedment, which introduces coupling and rocking. The increase of the m_0/m_S factor influences the displacement amplitudes in a way that can be viewed through an analogy with a single-degree-of-freedom system (Lee 1979). It is seen that the foundation converts a significant part of the incident wave energy into rotational motions and that this conversion is most efficient for incident wavelengths about twice the diameter of the foundation.

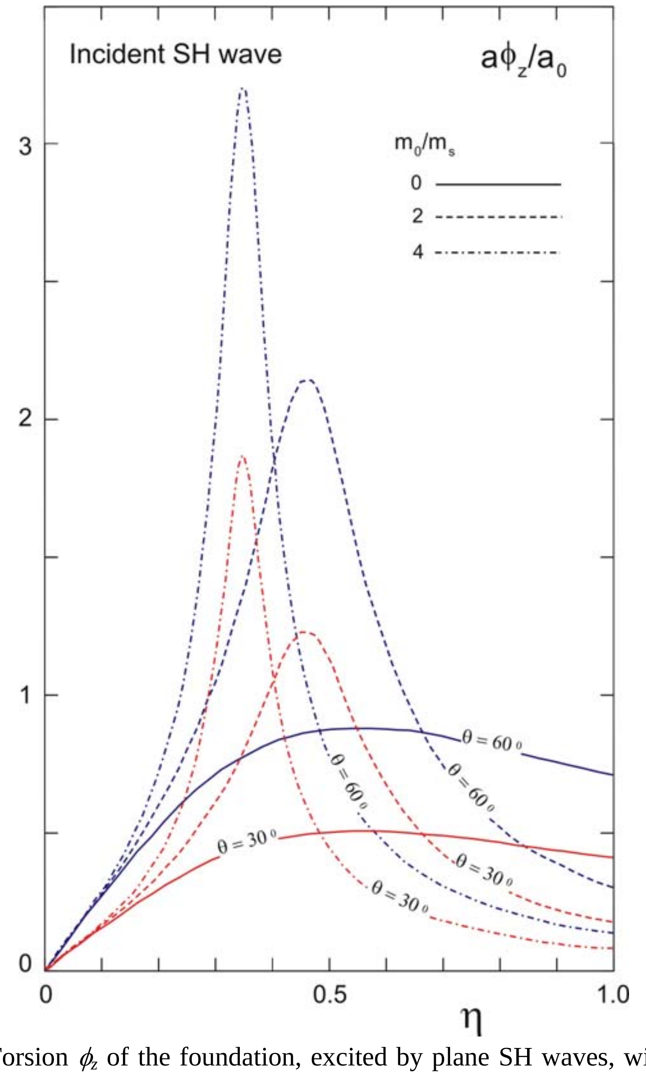

Fig. 3.3 Torsion ϕ_z of the foundation, excited by plane SH waves, with incident angles $\theta = 30°$, 60° and 85°, and dimensionless frequency η

Incident P wave excites vertical motion, horizontal translation, and rocking of the foundation $\{\Delta_X/a_0,\ 0\ ,\ \Delta_Z/a_0,\ 0\ ,\ a\phi_y/a_0,\ 0\}^T$. For vertical incidence, only the vertical motion is excited. The embedment introduces coupling of the horizontal translation and rocking, and, for a general incidence angle, a significant component of rocking is present. The normalized rocking amplitudes $|a\phi_y/a_0|$ are similar to those for excitation by SV waves (Fig. 3.2) but are smaller (Lee 1979).

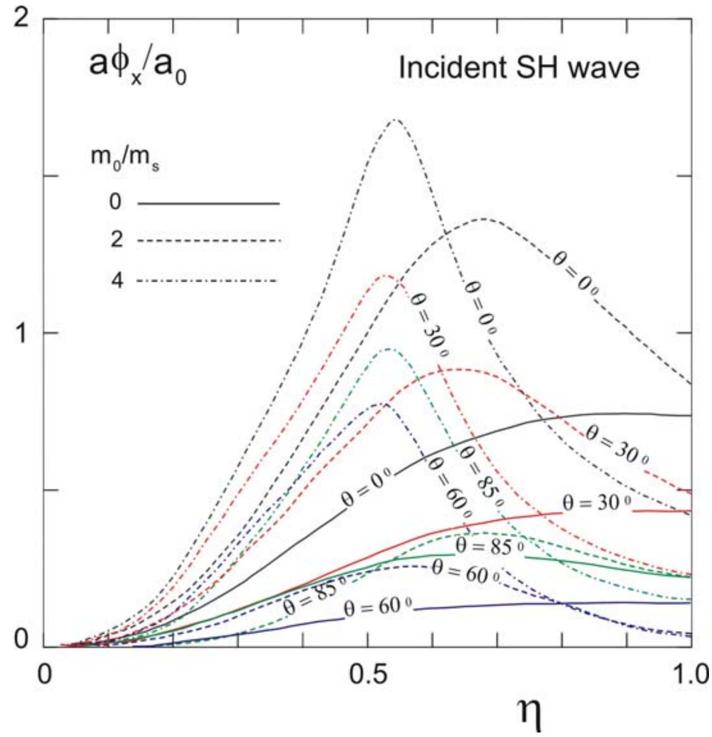

Fig. 3.4 Normalized rocking ϕ_x of the foundation, excited by plane SH waves, with incident angles $\theta = 0°$, 30°, 60° and 85°, and dimensionless frequency η

Incident SH wave excites torsion, horizontal translation, and rocking of the foundation. Unlike for incident P and SV waves, no vertical motion is excited: $\{0,\ \Delta_y/a_0,\ 0\ ,\ a\phi_x/a_0,\ 0,\ a\phi_z/a_0\}^T$. In the absence of the foundation, the free field surface displacement amplitude for an incident SH wave of unit amplitude is two for the horizontal y-component and zero for the x- and z-components, for all angles of incidence. The presence of the foundation changes this simplicity. In Figs. 3.3 and 3.4, the amplitudes $|a\phi_z/a_0|$ and $|a\phi_x/a_0|$ are plotted versus the dimensionless frequency η for different angles of incidence and for $m_0/m_S = 0$, 2, and 4. For vertical incidence, only the horizontal translation and rocking are excited, and there is no torsion ($a\phi_z a_0$). At low frequency, the y-component displacement amplitudes approach the free field displacement amplitude for all angles of incidence. As for incident P and SV waves, the motions decrease with increasing frequency because of wave scattering by the embedded foundation. Coupling and a significant component of rocking are introduced by the embedment. The increase in the m_0/m_S ratio illustrates again the analogy with single-degree-of-freedom system.

Hill's Equation. From (3.4), the matrix equation of motion of the foundation, with external forces present, is

$$[M_0]\{\ddot{U}\} + [K_s]\{U\} = \{F_s^*\} + \{F_{ext}\}. \tag{3.5}$$

After the mass matrix $[M_0]$, the stiffness matrix $[K_s]$, and the force $\{F_s^*\}$ have all been evaluated, those can be used to determine the foundation displacement $\{U\}$. For soil-structure interaction problems, in the presence of the structure, $\{F_{ext}\}$ is the force that the structure exerts on the foundation.

The first model illustrated here is shown in Fig. 3.5. It considers the cases involving incident P and SV waves. The structure is represented by an equivalent single-degree-of-freedom system, with a concentrated mass m_b at a height h above the foundation. It has a radius of gyration r_b and a moment of inertia $I_b = m_b r_b^2$ about 0. The degree-of-freedom is chosen to correspond to the rocking ψ_r. This rotation is restrained by a spring with rocking stiffness K_r and by a dashpot with rocking damping C_r (both not shown in Fig. 3.5). The gravitational force $m_b g$ is considered. Taking moments about B results in the equation of motion

$$\begin{aligned} \ddot{\phi}_y + \ddot{\psi}_r + 2\omega_r \varsigma_r \dot{\psi}_r + \omega_r^2 \psi_r = (1/\varepsilon)\Big\{ &-\big(\ddot{\Delta}_x / a\big)\cos\big(\phi_y + \psi_r\big) \\ &+ \big(\omega_r^2 \varepsilon_g + \ddot{\Delta}_z / a\big)\sin\big(\phi_y + \psi_r\big)\Big\}, \end{aligned} \tag{3.6}$$

where $\varepsilon = h(1+(r_b/h)^2)/a$, $\omega_r^2 = K_r / [m(h^2 + r_b^2)]$; the natural frequency of rocking squared, ζ_r is a fraction of the critical damping in $2\omega_r \zeta_r = C_r / [m(h^2 + r_b^2)]$; and $\varepsilon_g = 2/\omega_r^2 a$. Equation (3.6) is a differential equation coupling the rocking of the foundation and the structure with the horizontal and vertical motions of the foundation. It is a nonlinear equation, whose solution will require numerical analysis. In this work we will consider only the case when $\phi_y + \psi_r$ is small. Then

$$\ddot{\psi}_r + 2\omega_r\zeta_r\dot{\psi}_r + \left\{\omega_r^2\left(1-\varepsilon_g / \varepsilon\right) - \ddot{\Delta}_z / \varepsilon a\right\}\psi_r = \\ = -\ddot{\phi}_y + \left(1/\varepsilon\right)\left\{-\ddot{\Delta}_x / a + \left(\omega_r^2\varepsilon_g + \ddot{\Delta}_z / a\right)\phi_y\right\}. \quad (3.7)$$

For steady-state excitation by the incident P and SV waves, with frequency ω, Δ_x, ϕ_y, and Δ_z, and therefore the coefficients of (3.7), will be periodic. Equation (3.7) is then a special form of Hill's equation. Analysis of the stability of this equation can be found in the report by Lee (1979).

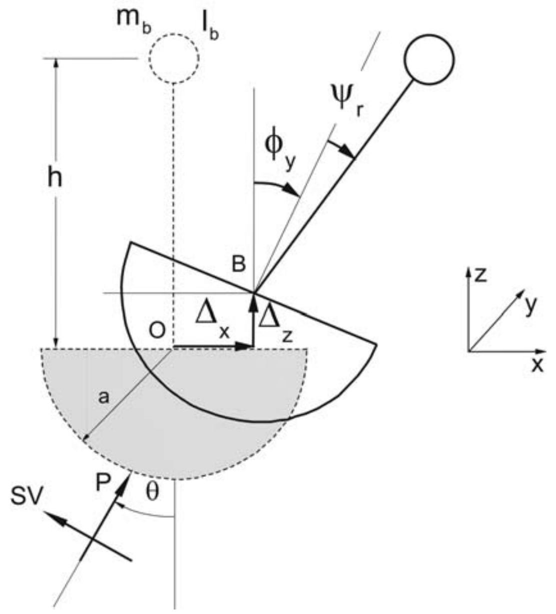

Fig. 3.5 In-plane rocking of foundation-structure model excited by plane P or SV waves

Solution of the Interaction Equation. For steady-state harmonic incident P and SV waves with excitation frequency ω, the vertical component of the foundation displacement, Δ_z, will also be harmonic with the same frequency ω. Considering only the period $T = 2\pi/\omega$, the most general solutions will take the form of Fourier series

$$\Delta_x = \sum \Delta_n e^{-in\omega t}, \quad \phi_y = \sum \phi_n e^{-in\omega t}, \quad \psi_r = \sum \psi_n e^{-in\omega t}, \quad (3.8)$$

where the summation is from $n = -\infty$ to $+\infty$. Δ_n, ϕ_n, and ψ_n are Fourier coefficients to be determined (Lee 1979). For the examples that follow, the model parameters have been chosen to take on the following values:

$m_0/m_s = 1$, $m_b/m_s = 3$, $h/a = 1$, $r_b/h = 0.5$, $\zeta_r = 0.02$, $\varepsilon_g = 0.1$, $a_0/a = 0.05$, and the dimensionless fixed-base frequency of the structure $\eta_r = \omega_r a/\pi\beta = 0.1$, 0.3, 0.5, and 0.8. $m_0/m_s = 1.0$ corresponds to the case of a rigid hemispherical foundation of the same density as the elastic medium of the half space.

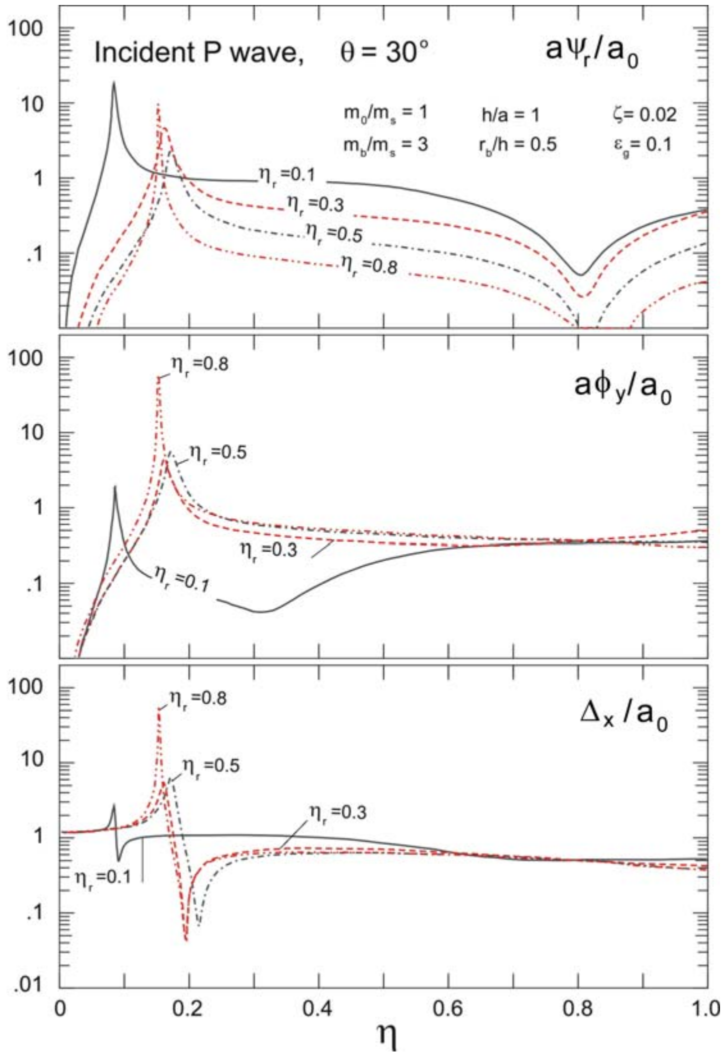

Fig. 3.6 Normalized rocking angles ψ_r and ϕ_y and translation Δ_x versus dimensionless excitation frequency η, for normalized model frequencies $\eta_r = 0.1$, 0.3, 0.5 and 0.8, and for incident P wave with $\theta = 30°$

The amplitudes of the coefficients Δ_1/a_0, $a\phi_1/a_0$, and $a\psi_1/a_0$ of the first harmonic components ($n = 1$) corresponding to the harmonic frequency of excitation, $e^{-i\omega t}$, are the dominating contributors to response. For $n > 1$, the coefficients Δ_n/a_0, $a\phi_n/a_0$, and $a\psi_n/a_0$, among other parameters, are dependent upon the ratio a_0/a, which is chosen to be 0.05. Figure 3.6 illustrates the response amplitudes of the first harmonic components ($n = 1$; the amplitudes for $n = 2$ are much smaller and may be neglected) of the Fourier series of Δ_x/a_0, $a\phi_y/a_0$, and $a\psi_r/a_0$, representing foundation translation, foundation rocking, and relative rocking response.

In the analysis and design of earthquake-resistant structures, it is necessary to estimate the maximum amplitudes of the relative responses at the top of the structure, which are then used to calculate the strain and the maximum stresses in the structure. In the absence of soil-structure interaction, the structure's relative rocking response, ψ_r, would be maximum at the fixed-base natural frequency, ω_r, and the relative response would approach infinity, as the fraction of critical damping, ζ_r, approaches zero. Interaction of the foundation with half space introduces "damping" into the relative response and results in a reduction of the "natural frequency" of the complete system. This reduction is more pronounced for larger values of fixed-base natural frequencies (Todorovska and Trifunac 1991).

At each of the frequencies where the relative responses experience maxima, the foundation rocking has an associated peak. At the fixed-base natural frequencies of long buildings (Trifunac 1972), the corresponding component of the external force will experience a maximum, and the corresponding displacement component has a "node" in the half space. Similar nodes are observed in the foundation rocking component of the three-dimensional models. However, because of the coupling of foundation rocking with the foundation horizontal translation, these nodes do not occur at exactly the fixed-base natural frequencies of the structure.

As the values of m_b/m_s and h/a increase, the rocking components of the foundation and the structure become more prominent–so prominent that the peak relative responses occur at frequencies characteristic of the natural rocking frequencies of the total system. For large m_b/m_s and h/a, the rocking stiffness of the structure, $K_r = m_b\omega_r^2(h^2 + r_b^2)$, becomes large. As the structure becomes stiffer, its relative rocking response becomes small, and the structural response, ψ_r, contributes a negligible amount to $\phi_y + \psi_r$. The total system then behaves like a rigid, partially embedded mass, $m = m_0 + m_b$, vibrating on an elastic half space (Biot 2006, Lee et al. 1982).

The model of the structure for the case of an incident SH wave (Fig. 3.7) is similar to the one for the incident P and SV waves. Its displacement vector

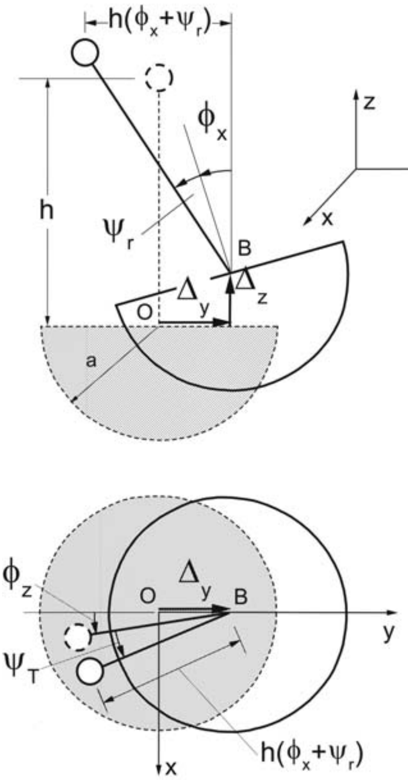

Fig. 3.7 Rocking (ϕ_x and ψ_r), torsion (ϕ_z and ψ_T) and translation Δ_y of foundation-structure model, excited by incident SH wave

is $\{0, \Delta_y, \Delta_z, \phi_x, 0, \phi_z\}^T$. The structure can rock about the *x*-axis (Fig. 3.7, top) and twist about the *z*-axis (Fig. 3.7, bottom). Let ψ_r be the relative angle of rocking of the mass. For torsion, let ψ_T be the relative angle of twist of the mass; I_T its moment of inertia about the *z* axis; K_T the torsional stiffness; C_T the torsional damping; ω_T the torsional natural frequency, $\omega_T^2 = K_T / I_T$; and ζ_T the fraction of critical damping for torsion, where $2\omega_T \varsigma_T = C_T / I_T$. Writing the rocking moment equilibrium equations about B (Fig. 3.7, top), torsional moment equilibrium about a vertical axis through B (Fig. 3.7, bottom), and assuming that $\phi_x + \psi_r$ and $\phi_z + \psi_r$ are small, gives

$$\begin{aligned} \ddot{\psi}_r + 2\omega_r \varsigma_r \dot{\psi}_r + \left\{ \omega_r^2 \left(1 - \varepsilon_g / \varepsilon\right) - \ddot{\Delta} / \varepsilon a \right\} \psi_r = \\ = -\ddot{\phi}_x + \left(1/\varepsilon\right) \left\{ \ddot{\Delta}_y / a + \left(\omega_r^2 \varepsilon_g + \ddot{\Delta}_z / a\right) \phi_x \right\} \end{aligned} \tag{3.9}$$

and

$$\ddot{\psi}_{\mathrm{T}} + 2\omega_{\mathrm{T}}\varsigma_{\mathrm{T}}\dot{\psi}_{\mathrm{T}} + \omega_{\mathrm{T}}^{2}\psi_{\mathrm{T}} = -\ddot{\phi}_{z}\,. \tag{3.10}$$

It is seen that for small relative response and small rocking, the torsional twist is uncoupled from the horizontal motions. Figure 3.8 illustrates the amplitudes of Δ_y/a_0, $a\phi_x/a_0$, and $a\psi_r/a_0$, for incident SH waves when $\theta = 30°$.

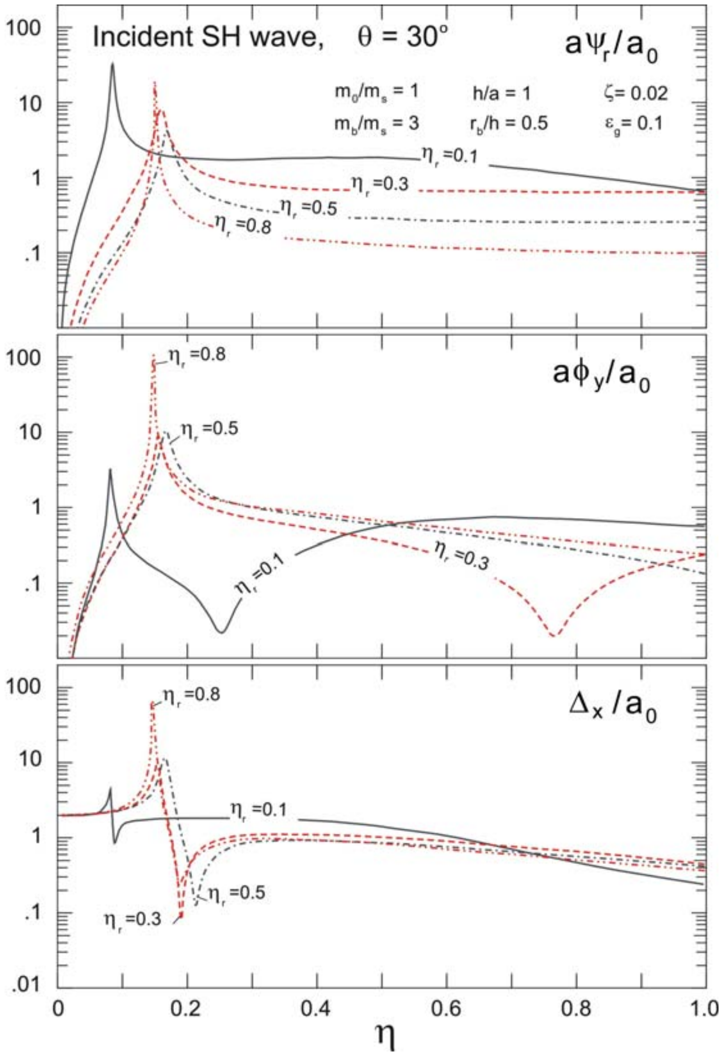

Fig. 3.8 Normalized rocking angles ϕ_x and ψ_r, and out-of-plane foundation motion Δ_y versus dimensionless excitation frequency η, for normalized model frequencies $\eta_r = 0.1$, 0.3, 0.5 and 0.8, and for incident SH wave with $\theta = 30°$

3.3 Soil-Foundation Interaction – Far Field

For distances greater than about 10*a* (Fig. 3.1) the motions of the half space, generated by building vibration, can be evaluated in terms of a point source consisting of two horizontal forces, F_{bx}, F_{by}, two rocking moments, M_{by}, M_{bx}, and a torsional moment, M_{bz}. A building oscillating in the *x*-*z* plane, for example, will produce strong Rayleigh waves in the *x* direction, while Love waves will be generated in the *y* direction (Bycroft 1956, Cherry 1962, Favela 2004). During forced vibration tests of a building, sufficiently large displacements of the ground can be generated, so that the recording and analysis can be made with standard recording devices at distances approaching 10 km (Luco et al. 1975). With specialized recording and data analyses, surface waves generated by building rocking can be detected at distances approaching 1.000 km (Favela 2004). The recorded motions contain valuable information about the properties of the medium of propagation and can serve as a full-scale laboratory for testing and verification of the wave propagation models of sedimentary and soil layers (Wong et al. 1977b), and of the influence of the surface topography on the amplitudes of surface motions (Wong et al. 1976).

3.4 Summary

The above single-degree-of-freedom system model of buildings erected on flexible soil shows how, through soil-structure interaction, a portion of the energy of incident waves is converted into the rotational motions. The eccentric location of the foundations, below the center of mass of the structures they support, makes the foundations the sources of rotational motions in the soil. Through the soil-structure interaction, the translational energy of the in-plane body P- and the SV-waves is converted into the foundation rocking (in the same plane), while the energy of the incident SH waves becomes the source of out-of-plane rocking and torsion about the vertical axis. It can be shown that the additional rotational excitation associated with the passage of Rayleigh (rocking) and Love (torsion) surface waves further amplifies the rotational motions of the foundations.

The amplitudes of the rotational motions of the foundations depend most upon the natural frequency of the structure above and on the wavelength of the incident waves below. The largest rotations of the foundation occur for relatively stiff buildings (when $\eta_r = \omega_r a/\pi\beta$ is large; e.g., 0.8 in Figs. 3.6 and 3.8) excited by incident waves with wavelengths that are approximately double the characteristic width of the foundation (when η is near one half; see Figs. 3.2, 3.3, and 3.4).

For multiple foundations of long structures, like bridges, for example, all of the above will hold for individual foundations, but additional rotational motions in the soil will also result from differential motions of the supports and from the wave passage effects (Trifunac and Todorovska 1997, Trifunac and Gicev 2006). These additional rotations will tend to be associated with longer wave-lengths, comparable to the inter-foundation distances of the multiple foundation systems, and can be viewed as resulting from a couple, or from a chain of couples, whose component forces lie in a vertical (for in-plane excitation) or horizontal (for out-of-plane excitation) plane.

In an urban setting, a distribution of buildings will act as an extended surface source area, consisting of a large number of closely spaced sources of translational and rotational motions, which will cause the warping of the half space surface in the near field and a seemingly random distribution of strong, high-frequency surface waves in the far field. For a distribution of buildings 1–50 stories high, the waves generated by the movement of their foundations will be in the range 0.1–10 Hz.

In this chapter, the basic source of rotational motions has been illustrated in terms of a spherical rigid foundation supporting a single-degree-of-freedom oscillator as a model of a “simple building”. Mutatis mutandis, many of the above-described phenomena can be generalized to interpret the changes in the free-field wave motions resulting from a broad spectrum of other eccentrically supported “oscillators” ranging from individual trees to large and small geological formations like those in Monument Valley in Utah, down to Meteora in Greece or Sigiria in Sri Lanka, for example.

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