Conf-950740--43

BNL-NUREG-61664

CONVOLUTION-DECONVOLUTION IN DIGES

A.J. Philippacopoulos

Department of Advanced Technology
Brookhaven National Laboratory
Upton, NY 11973

Nikolaos Simos

Department of Advanced Technology
Brookhaven National Laboratory
Upton, NY 11973

ABSTRACT

Convolution and deconvolution operations is by all means a very important aspect of SSI analysis since it influences the input to the seismic analysis. This paper documents some of the convolution/deconvolution procedures which have been implemented into the DIGES code. The 1-D propagation of shear and dilatational waves in typical layered configurations involving a stack of layers overlying a rock is treated by DIGES in a similar fashion to that of available codes, e.g. CARES, SHAKE. For certain configurations, however, there is no need to perform such analyses since the corresponding solutions can be obtained in analytic form. Typical cases involve deposits which can be modeled by a uniform halfspace or simple layered halfspaces. For such cases DIGES uses closed-form solutions. These solutions are given for one as well as two dimensional deconvolution. The type of waves considered include P, SV and SH waves. The non-vertical incidence is given special attention since deconvolution can be defined differently depending on the problem of interest. For all wave cases considered, corresponding transfer functions are presented in closed-form. Transient solutions are obtained in the frequency domain. Finally, a variety of forms are considered for representing the free field motion both in terms of deterministic as well as probabilistic representations. These include (a) acceleration time histories, (b) response spectra (c) Fourier spectra and (d) cross-spectral densities.

1.0 INTRODUCTION

1.1 Problem Definition The need to develop acceptable floor response spectra has been an ongoing process. Such spectra are affiliated with seismic loads that the structure is subjected to and they represent the prediction of the response of various elevations within the structure that in turn can be utilized to predict the response of sensitive equipment resting on a particular elevation. These seismic loads are conventionally expressed in the form of *design response spectra* for a number of reasons.

Consequently, the development of computational schemes which can incorporate the information or assessment pertaining to the seismic load and, in conjunction with the dynamic characteristics of the structure, predict the elevation spectral responses has been the focus of earthquake response prediction. The definition of the seismic load, which determines the theoretical basis of the link between excitation and response, has been deduced from both deterministic as well as stochastic models.

On one hand, the deterministic approaches seek to assess the elevation response due to a prescribed ground excitation or dynamic load on the structure itself. On the other hand, the stochastic approaches attempt to define the floor response to an anticipated ground excitation that belongs to a family of earthquakes which in turn is described by *target* response or power spectra.

Within the stochastic processes, however, the statistics that accompany the definition of the ground excitation are *usually* carried over to the floor response with an ensemble of realizations of the stochastic process that defines the ground excitation. This simulation of earthquakes procedure that attempts to *match* the statistics of the target spectrum has been used extensively both by *directly* linking the target response spectrum to an artificial earthquake or by implementing the constraint of the power spectral density function of the ground motion. The latter earthquake simulation process, more sophisticated in nature, matches some of the statistics of the target spectrum with realizations (sample earthquakes) deduced from the power spectrum of the stochastic process.

The direct link between a stochastic characterization of the ground excitation and the stochastic floor response has received less attention. Through this process, the statistical properties of an anticipated family of earthquakes, expressed in its power spectrum, are transferred to the structure of deterministically defined dynamic properties.

1.2 DIGES Profile

The present effort has been undertaken so that an efficient theoretical/computational tool can be devised such that seismic problems of concern to the regulatory agencies can be effectively treated. In this study, the *direct* link between the input excitation and the output response in the stochastic sense is explored. This dimension of the seismic analysis, along with the earthquake simulation procedures and the deterministic seismic and dynamic response of the structure, define the DIGES computational domain.

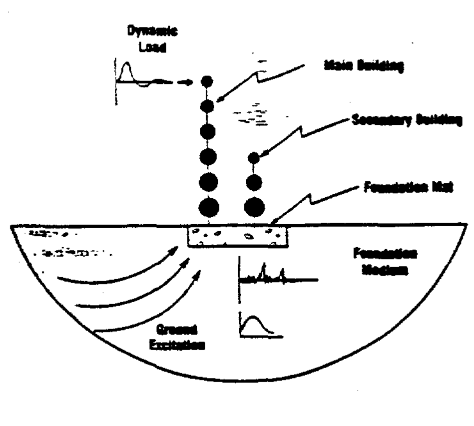

Figure 1. The physical system

Figure 1 depicts the physical system whose response to the action of dynamic loads is sought. Participating in the generic physical system are the superstructure, which is the focus of the resulting response, the foundation and the soil medium the foundation/superstructure is resting on. Further, the different dynamic loads that can excite the physical system are shown. The overall description of DIGES can be seen in Figure 2 where the general capabilities are listed.

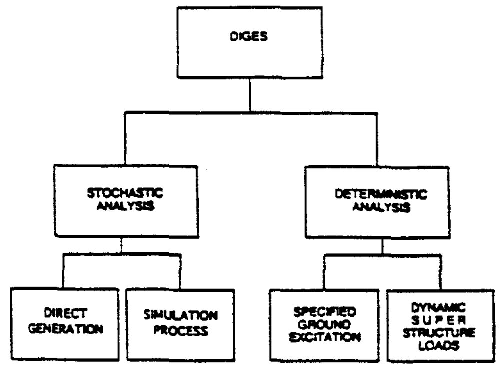

Figure 2. Overall description of DIGES

According to Fig. 2, analyses of both stochastic and deterministic nature can be undertaken. While in the deterministic analysis the consideration of dynamic superstructure loads has been implemented (an important element of dynamic analysis) alongside with the classical treatment of defined ground motion, the stochastic analysis mode incorporates both the earthquake simulation and the direct transferring of stochastic properties.

The stochastic mode, which implements both the simulation and the direct stochastic transferring) seeks to evaluate elevation response spectra induced by ground excitations that can be defined as both target response spectra or cross-spectral densities of the stochastic process describing the excitation.

The *direct* stochastic mode determines the cross spectral density matrix of the response $[\Phi_Y(\omega)]$ for a stochastic process with cross spectral density $\Phi_X(\omega)]$. For a statistic process that defines the free field in terms of *target* response spectra, a *consistent* cross spectral matrix is formed and eventually transferred to the elevation. The *simulation* seeks the elevation response spectra through by utilizing statistical properties of the responses at the same elevation due to an ensemble of ground accelerations whose response spectra *match* the target spectrum over some of its statistic properties.

DISCLAIMER

DISCLAIMER

As shown, both simulation procedures are implemented (one leads to ground motions from a response spectrum through its power spectrum and the other to ground motions directly from the response spectrum).

2. FREE-FIELD EARTHQUAKE

A free-field earthquake may be in the form of **a response spectrum, power spectrum or time varying acceleration.**

Response Spectra to Power Spectra

The response spectrum characterizing the free-field motion $RS_t(\omega, \xi)$ is known for the frequency range of interest. This spectrum could also be called *target* Response Spectrum. Assume that the power spectrum consistent with the target response spectrum is $\Phi_t(\omega, \lambda)$ where $\{\lambda\}$ is a vector of parameters that are specific of the power spectrum. These parameters define the shape of the analytical expression of the *psd* and they are unknown until the consistency between the power and the response spectra is achieved. Over the years, several closed form expressions that can describe the power spectrum of earthquake ground accelerations have been proposed. Some of these PSD forms are listed below:

i. Kanai-Tajimi

$$S_x(\omega, \lambda) = S_0 \frac{1 + 4\zeta_g^2 \left(\frac{\omega}{\omega_g}\right)^2}{\left[1 - \left(\frac{\omega}{\omega_g}\right)^2\right]^2 + 4\zeta_g^2 \left(\frac{\omega}{\omega_g}\right)^2}$$

ii. Ruiz-Penzien

$$S_x(\omega, \lambda) = S_0 \frac{1 + 4\zeta_g^2 (\omega/\omega_g)^2}{\left[1 - (\omega/\omega_g)^2\right]^2 + 4\zeta_g^2 (\omega/\omega_g)^2}$$

$$\frac{(\omega/\omega_p)^4}{\left[1 - (\omega/\omega_p)^2\right]^2 + 4\zeta_p^2 (\omega/\omega_p)^2}$$

iii. Superposition

$$S_x(\omega, \lambda) = S_0 \sum_{k=1}^{2} P_k \frac{(1 + 4\zeta_k^2)\left[1 - e^{-\frac{\omega^4}{\omega_k^4}}\right]}{\left[1 - \left(\frac{\omega}{\omega_k}\right)^2\right]^2 + 4\zeta_k^2 \left(\frac{\omega}{\omega_k}\right)^2}$$

Earthquake Simulation

If the **simulation** option is selected to transfer the free-field earthquake to the structure it is implied that an ensemble of generated earthquakes will be transferred by utilizing the Transfer Function of the system $H(\omega)$ according to the relationship

$$F_y(\omega) = H(\omega) F_x(\omega) \qquad (1)$$

where $F_x(\omega)$ and $F_y(\omega)$ are the Fourier expansions of the input and output respectively.

The artificial earthquakes can be generated from either a **power** or a **response** spectrum characterizing the free-field stochastic process.

PSD Based Ground Accel. Simulation

A time history $g(t)$ of an artificial acceleration can be generated from the form

$$g(t) = \zeta \left[2 \sum_{i=1}^{N} \sqrt{\Phi_a(\omega_i)\,\Delta\omega} cos(\omega_i t + \phi_i)\right] \qquad (2)$$

where, $w_i = i\Delta\omega \quad \Delta\omega = \frac{\omega_u}{N}$ while ω_u is a cutoff frequency above which the power spectrum is assumed to vanish, **N** is the number of **uniform** frequency increments, $\{\phi_i\}$ is a vector of random phase angles uniformly distributed between 0 and 2π (different choices of the vector of random phase angles will lead to a different simulated earthquake that has both the mean and the autocorrelation of the stochastic process described by the *PSD* of the stochastic free-field), $\Phi_a(\omega)$ is the power spectral density of the process and $\zeta(t)$ is a modulating function that introduces the nonstationarity in the generated record. It should be noted that the simulated earthquake $g(t)$ is periodic with a period $T_0 = \frac{2\pi}{\Delta\omega}$

Simulation Based on Response Spectra

Simulated earthquakes that belong to the family represented by the target response spectrum can assume the form,

$$g(t) = \zeta(t) \sum_{i=1}^{N} C_i(\omega) \, sin(\omega_i t + \phi_i) \qquad (3)$$

where $C_i(\omega)$ is the amplitudes of the i_{th} contributing sinusoid while ϕ_i and $\zeta(t)$ are the same as above.

When the complete ensemble of generated earthquakes has been transferred to the structure, the response of the system at any d.o.f. can be then seen as a single Response Spectrum which is deduced from the average of the ensemble of response spectra each deterministic process will provide,

$$\overline{RS}_y(\omega_i, \xi) = \frac{\sum_{j=1}^{n} RS_y(\omega_i, \xi)}{n} \qquad (4)$$

along with the statistical properties of the ensemble of amplifications at every specified frequency ω_i, $[m+\sigma] * RS(\omega_i)$ where σ is the number of standard deviations from the mean.

3. FOUNDATION INPUT MOTIONS

The foundation input motion represents the response of the *rigid* and *massless* foundation to the free-field motion. Generally, the response of the foundation depends on its geometry, the properties of the interacting soil and the nature of the impinging seismic waves. A *scattering* transfer function $H(\omega)$ links the free-field motion U_G^0 with the foundation input motion U_G

$$U_G = H(\omega) U_G^0 \qquad (5)$$

Described below are three different approaches that are extensively utilized in determining the foundation input motion:

a. Free-field applied directly

This case represents early stages of seismic analyses of building foundation systems according to which the criteria motion was directly applied at the bottom of the soil springs and it reflects primarily cases involving surface foundations.

$$H(\omega) = \begin{bmatrix} [I] \\ --- \\ [0] \end{bmatrix} \qquad (6)$$

where [I] and [0] are [3 x 3] unit and null matrices respectively.

b. Convolution/Deconvolution

In this case the foundation input motion is the free-field motion at some depth, depending on the characteristics of the embedded foundation. The free-field motion at a given depth is obtained through convolution or deconvolution depending on whether or not the criteria motion is treated as an outcrop motion or a surface (or near surface for very soft top layers) motion.

$$H(\omega) = \begin{bmatrix} [H_T(\omega)] \\ --- \\ [0] \end{bmatrix} \qquad (7)$$

Convolution in Uniform Soil Deposits

Inclined SH-wave

Figure 3 depicts the incidence of an inclined SH wave propagating in a uniform soil deposit. The displacement vector associated with such wave is of the form

$$u(r, \omega, t) = Ae^{ik(r \bullet p - c_s t)} d \qquad (8)$$

where

$p = sind\theta i_1 + cos\theta i_2$ = propagation vector

$d = i_3$ = direction of particle motion

$r = x_1 i_1 + x_2 i_2$ = position vector

A, k, c_s are the amplitude, wavenumber and phase velocity respectively

The requirement that the surface is free of traction yields that the reflected wave is in phase with the incident wave. The total displacement is then

$$u_3(x_1, x_2; \omega, t) = 2A cos a_\theta e^{ik(x_1 sin\theta - c_s t)} \qquad (9)$$

$$a_\theta = \frac{\omega x_2}{c_s} cos\theta$$

Thus, the transfer function between the displacement (or acceleration) at a depth $x_2 = -h$ and its counterpart at the surface is expressed in the form

$$H(\omega) = \frac{u_3(x_1, -h; \omega, t)}{u_3(x_1, 0; \omega, t)} = cos a_h \qquad (10)$$

$$a_h = \frac{\omega h}{c_s} cos\theta$$

Inclined P-waves

From the incidence of an inclined P-wave (seen in Fig. 4) and the condition that the surface is traction-free, the displacement vectors associated with the various waves have as follows:

Incident **P-wave**:

$$[u_1\ u_2\ u_3]^T = A_1 [sin\theta\ cos\theta\ 0]^T e^{ik(r \bullet p_1 - c_p t)}$$

Reflected **P-wave**:

$$[u_1\ u_2\ u_3]^T = A_2\,[sin\theta\ \ -cos\theta\ 0]^T\, e^{ik(r\bullet p_2 - c_p t)}$$

Reflected **SV-wave**:

$$[u_1\ u_2\ u_3]^T = A_3\,[cos\theta_0\ sin\theta_0\ 0]^T\, e^{ik_0(r\bullet p_3 - c_s t)}$$

c_p, c_s : **P** and **SV** wave velocity respectively

A_1, A_2, A_3 : Wave amplitudes

p_1, p_2, p_3 : propagation vectors of incident **P**, reflected **P** and reflected **SV**-wave

while, $sin\theta_0 = \frac{sin\theta}{s}$ $k_0 = sk$ and $s = \frac{c_p}{c_s}$

The amplitudes A_1, A_2 and A_3 satisfy the relation

$$\begin{bmatrix} \lambda + 2\mu cos^2\theta & -s\mu\sin 2\theta_0 \\ -\mu sin2\theta & -s\mu cos2\theta_0 \end{bmatrix} \begin{bmatrix} q_1 \\ q_2 \end{bmatrix} = - \begin{bmatrix} \lambda + 2\mu cos^2\theta \\ \mu sin2\theta \end{bmatrix} \quad (11)$$

where $q_1 = \frac{A_2}{A_1}$ $q_2 = \frac{A_3}{A_1}$ and λ, μ are the Lame constants of the halfspace.

The total displacement due to the incident and reflected waves is simply the superposition of the three displacement vectors. Based on the above relations various transfer functions of interest can be deduced. Specifically, the transfer function between the horizontal and the vertical displacement at the free-surface will be of the form

$$\frac{u_{1\|x_2=0}}{u_{2\|x_2=0}} = \frac{(1+q_1)\,sin\theta + q_2 cos\theta_0}{(1-q_1)\,cos\theta + q_2 sin\theta_0} \quad (12)$$

The transfer function between the vertical displacement at a depth **h** from the free surface and the vertical displacement at the surface is of the form

$$\frac{u_{2\|x_2=-h}}{u_{2\|x_2=0}} = \frac{\left(e^{-ia_L} - q_1 e^{ia_L}\right) cos\theta + q_2 e^{ia_s} sin\theta_0}{(1-q_1)\,cos\theta + q_2 sin\theta_0} \quad (13)$$

Similarly, the transfer function between the horizontal displacement at a depth **h** and the horizontal displacement at the surface takes the form

$$\frac{u_{1\|x_2=-h}}{u_{1\|x_2=0}} = \frac{\left(e^{-ia_L} + q_1 e^{ia_L}\right) sin\theta + q_2 e^{ia_s} cos\theta_0}{(1+q_1)\,sin\theta + q_2 cos\theta_0} \quad (14)$$

where $a_L = \frac{\omega h}{c_L} cos\theta$ and $a_s = \frac{\omega h}{c_s} cos\theta_0$ represent dimensionless frequencies for **P** and **SV** waves respectively.

For vertical incidence ($\theta = 0$) the transfer function matrix reduces to a single relationship between the vertical displacement at a depth **h** and the vertical displacement at the free surface, specifically,

$$\frac{u_{2\|x_2=-h}}{u_{2\|x_2=0}} = cos a_L^0 \qquad a_L^0 = \frac{\omega h}{c_L}$$

Inclined SH-wave in Soil Deposit Overlying a Rock Formation

In such formation, Figure 5, the halfspace represents the rock underlying the soil. Thus, the transfer functions will represent relations between the base rock (outcropping) motion and the motion in the soil deposit. The displacement vectors induced by the various waves involved have as follows:

Incident **SH**-wave:

$$u_3^{(1)} = A_1 e^{ik(r\bullet p_1 - c_{S,R} t)}$$

Reflected **SH** at interface:

$$u_3^{(2)} = A_2 e^{ik(r\bullet p_2 - c_{S,R} t)}$$

Refracted **SH** across interface:

$$u_3^{(3)} = A_3 e^{ik(r'\bullet p_1 - c_{S,S} t)}$$

Reflected **SH**-wave at surface:

$$u_3^{(4)} = A_3 e^{ik_0(r'\bullet p_1 - c_{S,S} t)}$$

where,

$c_{S,R}, c_{S,S}$: **SH**-wave velocities in the rock and soil respectively

r, r' : position vectors WRT coordinate systems

$p_1, .., p_4$: propagation vectors

$A_1, .., A_4$: wave amplitudes

k, k_0 : wave numbers

μ_s, μ_R : shear moduli of soil and rock respectively

Continuity of total stresses τ_{23} and total displacements u_3 across the interface yields,

$$\begin{bmatrix} A_1 \\ A_2 \end{bmatrix} = \frac{A_3}{2} \begin{bmatrix} 1-q & 1+q \\ 1+q & 1-q \end{bmatrix} \begin{bmatrix} e^{ia_H} \\ e^{-ia_H} \end{bmatrix} \qquad (15)$$

where,

$$q = \frac{s_{S,R} cos\theta_0 \mu_S}{c_{S,S} cos\theta \mu_R} \; a_H = \frac{\omega H}{c_{S,S}} \; sin\theta_0 = \frac{c_{S,S}}{c_{S,R}} sin\theta$$

Based on the above relations the following transfer functions can be deduced:

$$H_1(\omega) = \frac{motion\ at\ depth\ x_2' = -h}{motion\ at\ surface} = \frac{1}{2}\left(e^{ia_H} + e^{-ia_H}\right)$$

$$H_2(\omega) = \frac{motion\ at\ depth\ x_2' = -h}{motion\ at\ interface} = \frac{e^{ia_h} + e^{-ia_h}}{e^{ia_H} + e^{-ia_H}}$$

$$H_3(\omega) = \frac{motion\ at\ depth\ x_2' = -h}{at\ interface\ (no\ top\ soil)} = \frac{e^{ia_h} + e^{-ia_h}}{(1-q)\,e^{ia_H} + (1+q)\,e^{-ia_H}}$$

Considering that the rock is sufficiently stiff, the latter transfer function represents the relation between the motion at depth **h** and the outcropping motion. One should further note that the following relations hold between the transfer functions above:

$$H_1^0(\omega)\, H_2^H(\omega) = H_2^0(\omega)\, H_1^H(\omega) = 1$$

$$H_3^0(\omega)\, H_3^H(\omega) \neq 1$$

The three amplitudes A_1, A_2 and A_3 are related with two equations. The third equation required to completely define them depends on the selected input to the system. For example, if the outcropping motion $U_g e^{-i\omega t}$ is known then $A_1 = \frac{1}{2} U_g$ and the system is completely defined.

c: Kinematic Interaction

In the case of foundation input motion which incorporates kinematic interaction effects due to the scattering of the seismic waves by the rigid foundation, $H(\omega)$ is a [6 x 3] frequency dependent matrix containing the scattering coefficients which depend on the types of seismic waves considered, the properties of the underlying medium and the geometry of the foundation itself.

$$H(\omega) = \begin{bmatrix} H_{transl}(\omega) \\ --- \\ H_{rotnl}(\omega) \end{bmatrix} \qquad (16)$$

When kinematic interaction is considered, the $3x3$ H_{rotnl} submatrix in $H(\omega)$ is no longer a null matrix. Specifically, it contains scattering coefficients relating the rocking and torsional motion of the foundation due to the horizontal and vertical components of the free-field motion U_G^0.

The dependency of the foundation input motion on the geometry of the massless foundation and the interface condition with the soil, on the properties and the stratigraphy of the underlying soil and on the nature of waves leads to a complex problem. While in the generic complex configuration techniques in boundary integral or finite element methods need to be employed, for surface foundations with simple geometries (circular or rectangular) that rest on uniform halfspaces and are subjected to the action of plane waves analytical closed-form expressions of the scattering coefficients have been deduced by various researchers (Ref. 7, 8, 11).

Specifically, for a *rectangular* foundation $(2ax2b)$ resting on a half space and subjected to an incident nonvertical wave with propagation in the x_1 direction and vertical angle of incidence θ_V (angle between the normal to the propagation and the horizontal) which leads to *apparent* surface velocity $C_a = \frac{velocity\ of\ propagation}{cos\theta_V}$ (velocity of propagation = **P**, **S** or **Rayleigh** depending on the nature of waves), the scattering functions are in the form,

$$R_{x_1x_1} = R_{x_2x_2} = R_{x_3x_3} = \frac{sin\beta}{\beta}$$

$$R_{x_1x_3} = R_{x_2x_1} = 0$$

$$R_{x_3x_2} = \frac{3i}{a\beta}\left(\frac{sin\beta}{\beta} - cos\beta\right)$$

$$R_{x_2x_3} = \frac{-3ai}{(a^2+b^2)\beta}\left(\frac{sin\beta}{\beta} - cos\beta\right)$$

where, a = dimension normal to propagation and $\beta = \frac{\omega a}{C_a}$. Similarly, for a circular foundation of radius **r**:

$$R_{x_1x_1} = R_{x_2x_2} = R_{x_3x_3} = \frac{4}{\pi\beta}\int_0^{\beta} \Gamma cos\mu d\mu$$

$$R_{x_1x_3} = R_{x_2x_1} = 0$$

$$R_{x_3x_2} = \frac{-16i}{\pi r\beta^2}\int_0^{\beta} \Gamma cos\mu d\mu$$

$$R_{x_2x_3} = \frac{8i}{\pi r\beta^2}\int_0^{\beta} \Gamma \mu sin\mu d\mu$$

where, $\mu = \frac{\omega x_3}{C_a}$ and $\beta = \frac{\omega r}{C_a}$ and $\Gamma = \sqrt{1-\mu^2}$. For the circular case Luco (9) has deduced the first approximation in the form

$$R_{x_1x_1} = R_{x_2x_2} = R_{x_3x_3} \approx \frac{sin\beta}{\beta} \approx 1 - \frac{\beta^2}{6}$$

$$R_{x_3x_2} = 2R_{x_2x_3} \approx -\frac{3i}{\beta}\left(\frac{sin\beta}{\beta} - cos\beta\right) \approx -i\beta$$

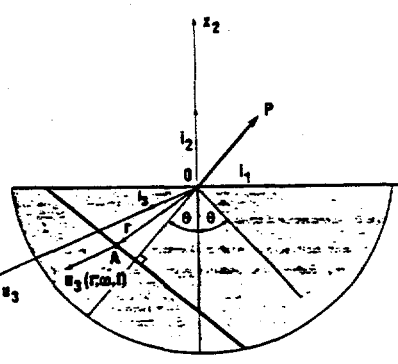

Fig. 3. Uniform Soil: SH-Wave Incidence

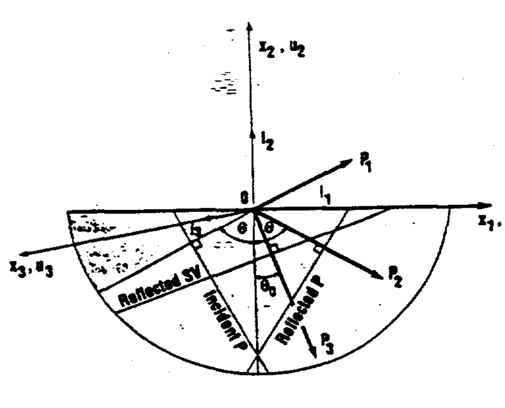

Fig. 4. Uniform Soil: P-Wave Incidence

It should be noted that when the interface condition is not *relaxed* there exists an additional rocking scattering function $R_{x_1x_3} = 0$ which for surface foundations is small and for practical purposes can be ignored.

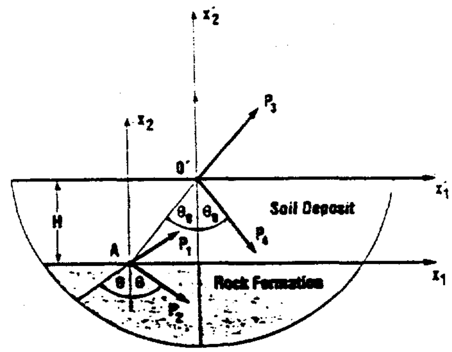

Fig. 5. Soil Overlying Rock: SH-waves

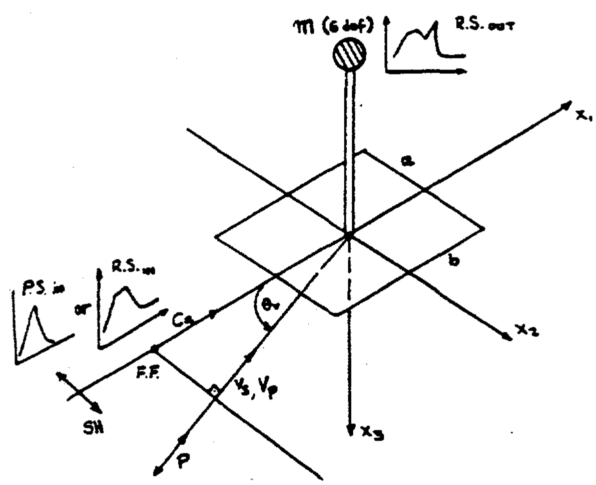

Fig. 6. Description of the SSI problem

4. Numerical Examples and Summary

In the previous section a scattering matrix $H(\omega)$ has been formed that allows the transferring of the free-field motion to the foundation. In addressing the applicability of the three different approaches the following points are raised:

- The *direct* application of the free-field motion on the foundation results in only translational components of the foundation motion regardless the soil stratigraphy, wave-type and the position of the foundation depthwise.
- The *convolution/deconvolution* approach does incorporate the stratigraphy by including the outcrop motion, the wave-type and the position of the foundation mat but it fails to introduce any torsional components on the foundation.
- The *kinematic interaction* approach incorporates the wave-type, the foundation geometry, the soil deposit stratigraphy (analytic closed-form solutions are possible only for uniform halfspace), and allows the torsional and rocking effects to appear in the lower half of the scattering matrix. For the simple model of a surface foundation resting on a halfspace and subjected to a nonvertically incident SH-wave, the differences between the convolution approach and the kinematic interaction extend even to the upper part of the scattering matrix. Apparently, the translational components of the free-field motion are altered for the kinematic interaction by the matrix coefficient $\frac{\sin\beta}{\beta}$.

In order to address the basic SSI problem which is defined as the transferring of a free-field motion (expressed in the form of a response or power spectrum) to a location on the structure, a simple structural model has been adopted (stick model on the foundation with a 6-dof mass) and subjected to various nonvertically incident waves, Fig. 6. Assuming that the free-field motion is a cross-spectral density matrix $\Phi_x(\omega)$, the relation

$$\Phi_y(\omega) = H(\omega)\,\Phi_x(\omega)\,H^*(\omega)^T \qquad (17)$$

leads to the cross-spectral density of the response $\Phi_y(\omega)$. The diagonal elements of $\Phi_x(\omega)$ are the power spectra of the particle motion in the free-field and the off-diagonal represent the cross-correlation of the motion in the three directions. For a free-field motion represented by a response spectrum, say Reg. 1.60, a compatible power spectrum is generated and transferred to the structure through Eqn (17). The response power spectrum is finally converted to a compatible response spectrum.

Incident SH-wave

The response spectra at the mass point of Fig. 6 are evaluated for incident SH-waves at different angles θ_v ($\theta_v = 90^o$ represent vertical incidence and $\theta_v = 0^o$ indicates surface SH waves). The input is a Reg. 1.60 response spectrum (single direction of particle motion) while the output response spectra include a translational component along x_2 (Fig. 7) and a torsional component $R_{x_3x_3}$ due to the kinematic interaction (Fig. 8).

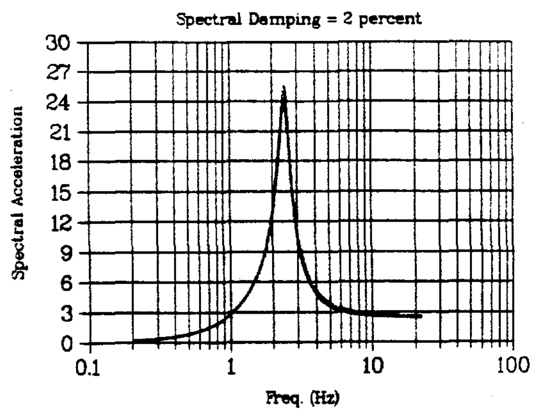

Fig. 7. Translational RS at Mass Point

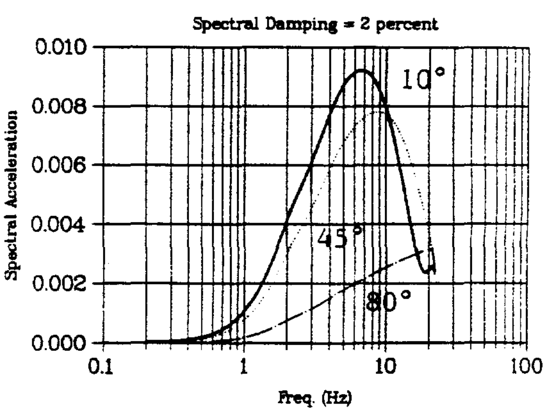

Fig. 8. Torsional RS at Mass Point

Incident P-wave

The same model is subjected to a P-wave impinging at different angles. Along with the vertical (Fig. 9) and horizontal component (Fig. 10), a rocking response spectrum about axis x_2 (Fig. 11) is evaluated.

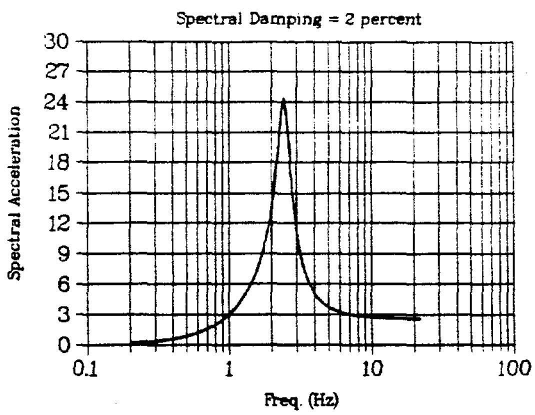

Fig. 9. Vertical RS (inclined P-wave)

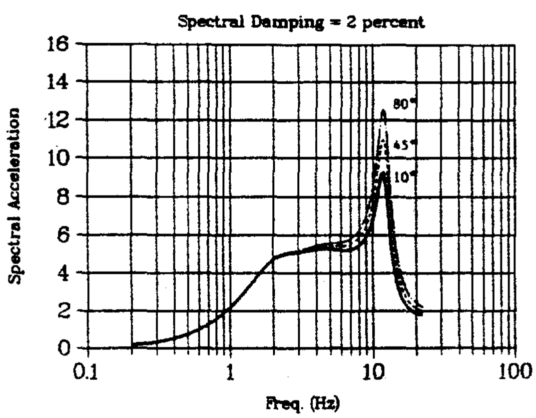

Fig. 10. Horiz. RS (inclined P-wave)

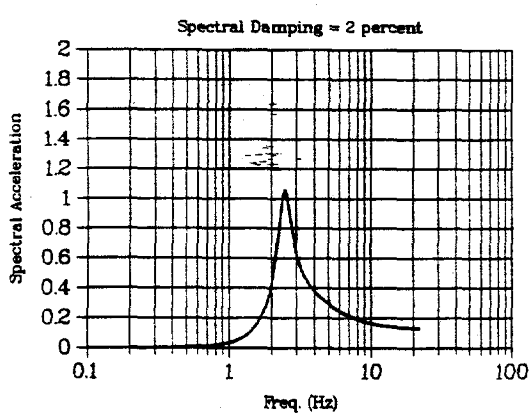

Fig. 11. Rocking RS (inclined P-wave)

References

1. R.A. Parmelee, *Building-Foundation Interaction Effects*, J. Eng. Mech. Div. ASCE, 93, pp. 131-152, 1967.
2. J. E. Luco, *Linear Soil-Structure Interaction* - Seismic Safety Margins Research Program, UCRL-15272, PSA No.7249809, 1980.
3. Y.K. Lin, *Probabilistic Theory of Structural Dynamics*, Krieger, 1976.
4. R.W. Clough, J. Penzien, *Dynamics of Structures*, McGraw Hill,1975.
5. D. Gasparini, E. Vanmercke, *Simulated Earthquake Motions Compatible with Prescribed Response Spectra*, MIT/Dept. of Civil Engnrg, Report No. R76-4, 1976.
6. P.D. Spanos, L.M. Vargas Loli, *A Statistical Approach to Generation of Design Spectrum Compatible Earthquake Time Histories*, Soil Dynamics and Earthquake Engineering, Vol.4, No.1, 1985.
7. H.L. Wong, J.E. Luco, *Tables of Impedance Functions and Input Motions for Rectangular Foundations*, Univ. of Southern California Report, Report No. CE 78-15, 1978.
8. A. Pais, E. Kausel, *Stochastic Response of Foundations*, MIT/Dep. of Civil Engrg. Report, Research Report R85-6, 1985.
9. J.E. Luco, A. Mita, *Response of a Circular Foundation on a Uniform Half-Space to Elastic Waves*, Earthquake Engrg. and Struct. Dynamics, Vol. 15, pp.105-118, 1987.
10. M. Shinozuka, G. Deodatis, *Ground Acceleration Time History Generation Under Specified PSD Function and Calculation of Response Spectra*, BNL Technical Report A-3962-1-11/89, 1989.
11. B.Li, F.G.A Al-Bermani and S. Kitipornchai, *Maximum Response of Assymetric Structures Subject to a Multicomponent Earthquake*, Earthquake Engineering and Structural Dynamics, Vol.22, pp. 1047-1066, 1993.
12. N. Simos, A.J. Philippacopoulos, *Theoretical Basis of Diges*, BNL Technical Report, 1993.

Acknowledgments

This work was performed under the auspices of the NRC.