SEISMIC DESIGN METHOD FOR ARBITRARY PROPAGATING WAVES

M. M. ETTOUNEY, J. A. BRENNAN, A. A. AGUERO
Burns and Roe, Inc., 185 Crossways Park Drive, Woodbury, New York 11797, U.S.A.

Abstract

This paper presents a complete analysis and design method which accounts for the multi-incident angle (wave number) case. The proposed method is presented in two steps. The first step is to generalize the concept of the design response spectra so that it can represent the seismic accelerations due to incident waves with multi-wave numbers. This was done by introducing the concept of the three-dimensional design spectrum (3-DS). The 3-DS is based on assuming that the spectrum is a surface instead of a line, as in the conventional case. This surface is a function of the frequency as well as the wave number of the seismic excitation. A simple and logical relation between the 3-DS and the conventional spectrum is presented and it is shown that the conventional spectrum is a limiting case of the 3-DS.

The second step is to present an accurate and simple method of analyzing a given plane strain soil-structure system for the 3-DS input motion. The solution is based on a semi-analytical approach which solves the elastic wave propagation equation of the layered soil due to a surface harmonic impulse. The rigid body motion of the footing is then enforced and the dynamic soil flexibility coefficients are determined. The transfer functions between the free field and the building are obtained for any required wave number (incident angles) at the free field. The method is then repeated for as many wave numbers (incident angles) as the 3-DS require. The final frequency spectra, building accelerations and building response spectra could be obtained then by a simple numerical integration technique.

Some practical dynamic soil-structure interaction cases were studied using both the 3-DS approach and the conventional method of analysis. Comparisons between the structural responses in both cases show considerable differences, expecially in the case of structural rocking response where higher amplitudes were obtained from the 3-DS approach.

1. Introduction

Seismic analysis of massive structures due to nonvertically propagating seismic waves have been studied lately by many authors, [1, 2, 3, 4, 10]. It was found [10] that the seismic response of the massive structures is very sensitive to the assumed angle of incidence of the seismic waves. This paper presents a complete design method which accounts for the variability of the angle of incidence, and uses the Newmark design spectra 18 as a basis of the time dependent input motion. This design method is conservative and is simple enough to be used in the design of massive and sensitive structures.

2. Theoretical Background

2.1 Analysis of Structure for General Wave Motion

Consider the plane soil-structure system shown in Figure 1. The soil is a multi-layered system resting on an elastic half-space (or rigid rock). The soil layers are assumed to be composed of linear hysteretic material with complex Lame's constants $\lambda(1+2i\beta)$ and $G(1+2i\beta)$, for each layer where β is the percentage of critical damping in the soil layer [5, 6, 7, 8]. The two dimensional wave equations for the system shown in Figure 1 are:

$$\nabla^2\psi = \frac{1}{C^2{}_p}\ \psi_{,tt} \qquad \text{and}$$
$$\nabla^2\phi = \frac{1}{C^2{}_s}\ \phi_{,tt} \qquad (1)$$

where ψ and ϕ are potential functions and C_p and C_s are the dilatational wave velocity and the shear wave velocity of the soil layer, respectively.

The solutions of equations (1) for harmonic motion with frequency (w) were derived [7, 9] as:

$$\psi = A\ \exp\ \left(i(wt-kx+knz)\right) \quad + B\ \exp\ \left(i(wt-kx-knz)\right)$$
$$\phi = C\ \exp\ \left(i(wt-kx+khz)\right) \quad + D\ \exp\ \left(i(wt-kx-khz)\right) \qquad (2)$$

where $n^2 = (C/C_p)^2-1$

$h^2 = (C/C_s)^2-1$

C = phase velocity

k = wave number

and A, B, C and D are constants representing the amplitudes of elastic plane waves of different kinds, which are propagating in different directions [7, 9, 10].

Gazetas [7], used equations (2) to obtain the compliance functions of massless footings. Ettouney, Brennan and Brunetti [10] presented a general solution of the equations as:

$$\left[\underset{\sim}{K}_d - w^2\underset{\sim}{M}\right] \underset{\sim}{V}^*(\alpha) = \underset{\sim}{P}^* - \underset{\sim}{P}_o{}^*(\alpha) \qquad (3)$$

where

$\underset{\sim}{M}$ = mass matrix of the rigid structure

$\underset{\sim}{K}_d$ = soil stiffness matrix

The definition of the displacement vector $\underset{\sim}{V}^*(\alpha)$ and the force vector $\underset{\sim}{P}^*$ are shown in Figure 2. The expressions for the matrices $\underset{\sim}{K}_d$, $\underset{\sim}{M}$ and the vector $\underset{\sim}{P}^*{}_o$ can be found in reference [10]. Equation (3) defines the structural compliance V* as a result of any applied forces on the structure which is presented by $\underset{\sim}{P}^*$, and the seismic waves, which generate the vector $\underset{\sim}{P}^*{}_o$. This vector is a function of the assumed incident angle of the plane seismic wave (α). It was shown [10] that the structural compliances $\underset{\sim}{V}^*$ are very sensitive to the assumed angle of incidence of the seismic wave.

2.2 General Design Spectra

The concept of design spectra is based on defining a response spectra [11, 12, 13] at the free field, then evaluating the structural response [14, 15] using a field equation with an assumption of vertically propagating waves ($\alpha = 0$). This implies an assumption that the input design spectra is a resultant only of vertically propagating waves. To investigate this point further, assume that the free field motion that generates a design spectra is

$$U(t,x) = \sum_{j=1} \sum_{\ell=1} F_{\ell j} \quad \exp \left(i(w_\ell t - k_j x)\right) \qquad (4)$$

where w_ℓ and k_j are the frequency and the wave numbers, respectively. This equation shows that the ground motion is a function of both the frequency of excitation and the wave number. It was shown that [16, 17] the contribution of the components of ground motion with finite wave numbers could be very large. This shows that the design spectra is actually a surface in the k-w domain, rather than a line in the w-domain. However, it is necessary to have a record of the ground motion showing the space as well as the time behavior (U(t,x)) in order to construct the required three dimensional design spectra. Only adequate time behavior of the ground motion is available. Space behavior is available also [17] but the recordings are not large enough to make a general statistical conclusion.

To avoid this apparent impass in a conservative way, a semi-probabilistic approach is considered. This approach assumes that the angle of incidence of the seismic wave (α) is a normally distributed random variable. The mean of this distribution is zero. These should be logical assumptions since seismic behavior is natural phenomena and most natural phenomenon are normally distributed. However, evaluation of the standard deviation of this distribution remains a major problem since there are not enough measurements. The solution of this problem is to solve the system for several assumed standard deviations and envelope these solutions to obtain an upper bound solution.

2.3 Expected Value of Structural Compliance

If the probability of the occurrence of the angle of incidence of the seismic wave in the free field is assumed to be $P(\alpha)$, then the expected value of the structural compliance is

$$E\left[\underset{\sim}{V}\right] = \int P(\alpha)\ \underset{\sim}{V}^*(\alpha)\ d\alpha \qquad (5)$$

Equation (5) could be used to evaluate the expected value of the structural response for an assumed standard deviation of the incident angle (α). For a standard deviation of zero, the expected value becomes

$$E\left[\underset{\sim}{V}\right] = \underset{\sim}{V}^*(o)$$

which is the conventional expression used for vertically propagating seismic waves.

Equation (5) should be evaluated several times with different assumed standard deviations. Enveloping the resulting set of expected values, we get an upper bound of the structural response.

3. Example

To demonstrate the use of this method, a practical problem will be considered. Figure 3 shows a one-layer soil-structure interaction system, resting on a rigid rock. This system is excited by horizontal and vertical motions at the free field simultaneously. The response spectrum of the equal horizontal and vertical free field accelerations is shown in Figure 4. This system was analyzed for five different standard deviations of α, namely 0., 5.0, 10.0, 15.0 and 20.0 degrees. Figure 5 shows the expected response spectra of the horizontal motion of the structure for all these cases. The expected motions for non-zero standard deviations are higher than that of the zero standard deviation case (vertically propagating waves). This is due to the coupling terms between the horizontal and vertical motions which have a large contribution in the non-zero standard deviation cases, while they have no contribution for the case of the vertically propagating waves. Also, the shifting in the natural frequencies of the system causes higher responses in the high frequency range. Figure 6 shows the final design envelope of the spectra resulting from consideration of all the standard deviation cases and the conventional design spectrum resulting from the vertically propagating waves assumption. It is clear that the envelope spectrum is more conservative than that of the conventional design spectrum.

4. Conclusions

A simple and complete method of seismic analysis and design of massive buildings is presented. This method accounts in a conservative way for the randomness of the angle of incidence of the seismic waves. The method is semi-probabilistic and is based on logical assumptions.

An example problem was presented and the suggested method of analysis resulted in higher structural responses when compared to the conventional method.

5. Acknowledgments

Dr. B. Bedrosian of Burns and Roe, Inc. made many helpful suggestions. This work was made possible by the use of the facilities of the Civil Engineering Division, Burns and Roe, Inc., Woodbury, N. Y.

6. References

[1] Wong, H. L., and Luco, J. E., "Dynamic Response of Rectangular Foundations to Obliquely Incident Seismic Waves," Earthquake Engineering and Structural Dynamics, Vol. 6, 1978, pp. 3-16.

[2] Luco, J. E., "Dynamic Response of Rectangular Foundations for Rayleigh Wave Excitation," Proceedings, 6th World Conference on Earthquake Engineering, New Delhi, India, 1977.

[3] Newmark, N. M., "Torsion of Symmetrical Buildings," Proceedings, 4th World Conference on Earthquake Engineering, Santiago, Chile, 1969.

[4] Dominguez, J., and Roesset, J. M., "Response of Embedded Foundations to Travelling Waves," Department of Civil Engineering Research Report, R78-24, Massachusetts Institute of Technology, Cambridge, Massachusetts, 1978.

[5] Kausel, E., "Forced Vibrations of Circular Foundations on Layered Media," Department of Civil Engineering Research Report, R74-11, Massachusetts Institute of Technology, Cambridge, Massachusetts, 1974.

[6] Chang-Liang, V., "Dynamic Response of Structures in Layered Soils," Department of Civil Engineering Research Report, R74-10, Massachusetts Institute of Technology, Cambridge, Massachusetts, 1974.

[7] Gazetas, G. C., and Roesset, J. M., "Forced Vibrations of Strip Footings on Layered Soils," National Structural Engineering Conference, American Society of Civil Engineers, University of Wisconsin, Madison, Wisconsin, August 22-25, 1976.

[8] Lysmer, J., et al, "A Finite Dynamic Model for Infinite Media," Journal of the Engineering Mechanics Division, American Society of Civil Engineers, Vol. 95, No. EM4, 1969.

[9] Jones, J. T., "Soil Amplification of SV and P Waves," Thesis presented to Massachusetts Institute of Technology, at Cambridge, Massachusetts, in 1970, in partial fulfullment of the requirements for the Degree of Master of Science.

[10] Ettouney, M. M., Brennan, J.A., and Brunetti, J. V., "Analysis of Massive Structures Supported by Soil Layers Subjected to a Propagating Elastic Wave with Arbitrary Direction," Proceedings, Conference on Structural Analysis, Design and Construction in Nuclear Power Plants, Vol. 2, pp. 539-552, Porto Alegre, Brazil, April 1978.

[11] Newmark, N. M., and Rosenblueth, E., Fundamentals of Earthquake Engineering, Prentice-Hall, Inc., Englewood Cliffs, N. J., 1971.

[12] Wiegle, R. L., Earthquake Engineering, Prentice-Hall, Inc., Englewood Cliffs, N. J., 1970.

[13] Okamoto, S., Introduction to Earthquake Engineering, John Wiley and Sons, Inc., N. Y., 1973.

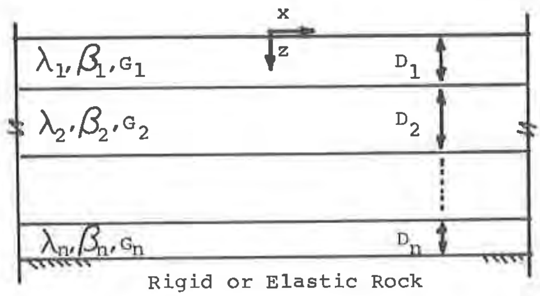
[14] Lysmer, J., et al, "FLUSH - A Computer Program for Approximate 3-D Analysis of Soil-Structure Interaction Problems," Earthquake Engineering Research Center, University of California at Berkeley, Report No. 73-9, 1973.

[15] Schnable, P. B., Lysmer, J. and Seed, H. B., "SHAKE - A Computer Program for Earthquake Response ...," Earthquake Engineering Research Center, University of California at Berkeley, Report No. 72-12, 1972.

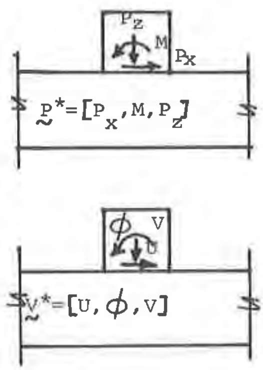
[16] Trifunac, M. D., "Response Envelope Spectrum and Interpretation of Strong Earthquake Motion," Bulletin of the Seismological Society of America, Vol. 61, No. 2, April 1971, pp. 343-356.

[17] Hanks, T. C., "Strong Ground Motion Following the San Fernando, California, Earthquake: Ground Displacements," Bulletin of the Seismological Society of America, Vol. 65, 1975, pp. 193-225.

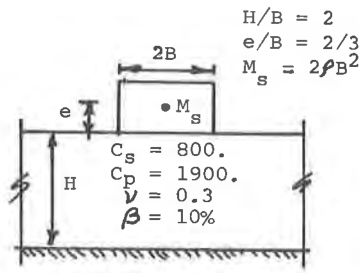
[18] Newmark, N. M., Blume, J. A. and Kapur, K. K., "Seismic Design Spectra for Nuclear Power Plants," Journal of the Power Division, American Society of Civil Engineers, No. PO2, 1973.



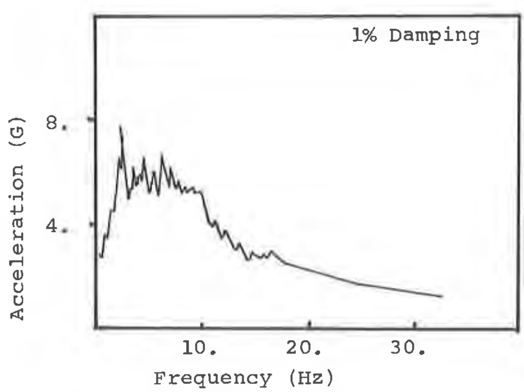
1 Layered Soil Media



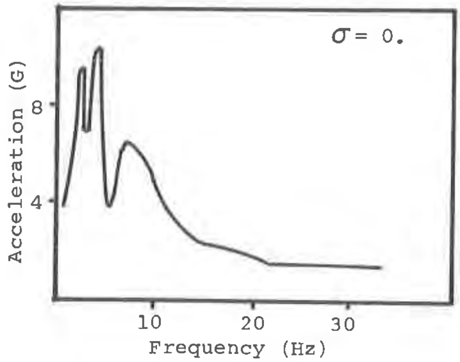
2 Coordinate System



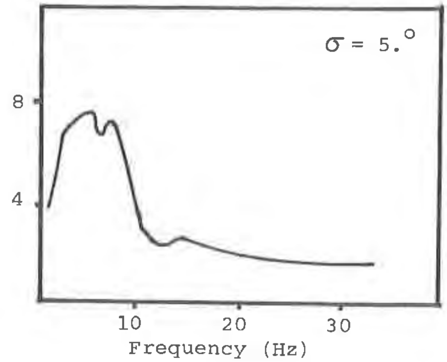
3 Example Problem



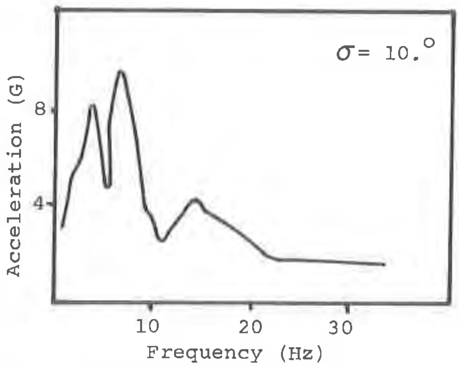
4 Input Horizontal and Vertical Acceleration Spectrum (1% Damping)



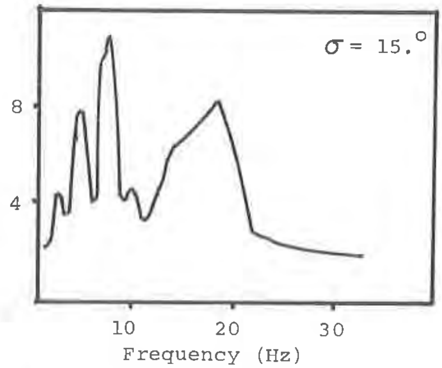
5.a Horizontal Spectrum (1% Damping
σ = 0.°)



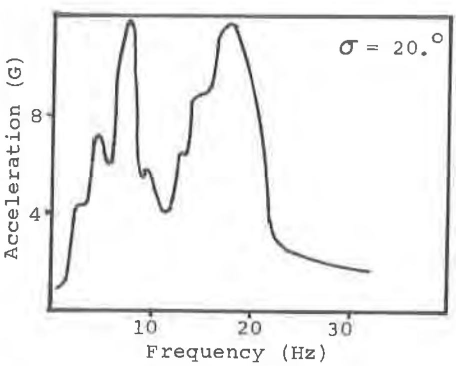
5.b Horizontal Spectrum (1% Damping
σ = 5.°)



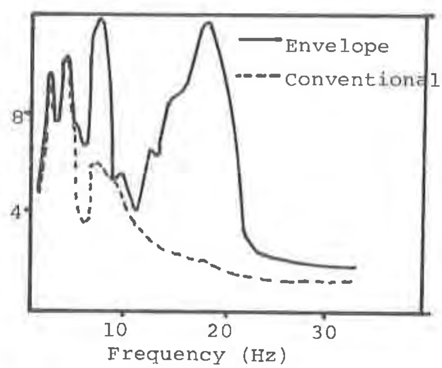
5.c Horizontal Spectrum (1% Damping
σ = 10.°)



5.d. Horizontal Spectrum (1% Damping
σ = 15.°)



5.e Horizontal Spectrum (1% Damping
σ = 20.°)



6 Comparison of the Methods