RESPONSE ANALYSIS OF ADJACENT STRUCTURES AND COMPARISON WITH RECORDED DATA

Farhad BEHNAMFAR[1] And Yoshihiro SUGIMURA[2]

SUMMARY

Dynamic analysis of a real-life building adjacent to another structure under earthquake time histories is discussed in this paper. The building under investigation is one of the experimental twin buildings of Tohoku University, Sendai, Japan. Both buildings are instrumented for recording the earthquake acceleration. The recorded data is used here to verify the theoretical method of analysis. The system of two adjacent buildings is modeled using a 2D boundary element procedure and acceleration spectrum of the base and roof levels are computed with the recorded free-field motion as input. The buildings in the model are represented by their equivalent characteristics in the first mode. Variation of earthquake motion in space is also included assuming an incoherency function. The calculated spectra are verified against the recorded spectra at the same locations. The above procedure is executed for two different earthquake events. The results of the analysis illustrate good agreement between the theoretical and recorded responses at certain conditions regarding the incident wave field. It is shown that even using a rough structural and soil model in structure-soil-structure analysis can give results which are exact enough for engineering use.

INTRODUCTION

Calculating the dynamic response of a structure is usually carried over assuming a rigid bearing medium, i.e., a soil without internal deformations, under the structure. Since any such medium has a finite rigidity, its own relative displacements also contribute to the structural response. So, as the soil becomes softer, the recorded response of a structure is expected to depart more and more from the values calculated with the above basic assumption. The softness of the soil beneath the structure also results in an exchange of energy or cross-interaction between the building under consideration and the nearby buildings which alters the structural response pattern even more.

Taking into account the effects of soil flexibility and cross-interaction with the adjacent buildings rigorously is possible via different formulations within the finite element and boundary element methods. In these methods the finite element model of the structures are coupled with the soil by discretizing either the structure-soil interface or a limited region of the soil around the structure. Then the free-field motion of the soil is induced at the boundary nodes as the input motion for which the dynamic response of the system is calculated.

The problem of the structure-soil-structure interaction has been addressed by several researchers. Dynamic response of adjacent rigid surface foundations was investigated in 2D case by [Hryniewicz, 1993] and in 3D case by [Lin et al., 1987]. The effects of flexibility of such foundations were explored by [Qian et al., 1996]. As for the adjacent structural response, the case of surface structures was examined by [Luco & Contesse, 1973] and of the embedded structures by [Imamura et al., 1992].

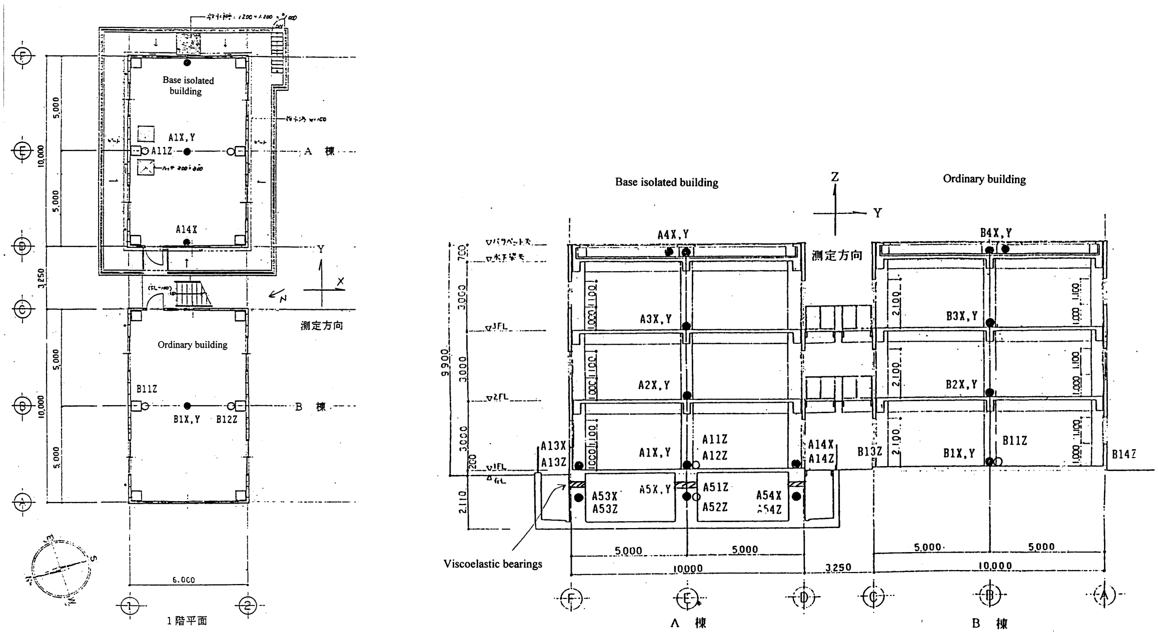
The above theoretical methods involve many assumptions regarding structure, soil, and incident wave field. Assessing the effects and accuracy of these assumptions needs instrumenting some adjacent buildings, recording the structural response under ambient, forced, or more importantly, earthquake vibrations, and, comparing the records with the analytical results. This has been the motivation behind the present study. The needed database

[1] *International Institute of Earthquake Engineering and Seismology, Tehran, Iran. Email: farhad@dena.iiees.ac.ir*
[2] *Department of Architecture and Building Engineering, Tohoku University, Sendai, Japan. sugi@strmech.archi.tohoku.ac.jp*

for performing this task was prepared now several years after instrumenting the experimental twin buildings of Tohoku University, Sendai, Japan and occurrence of many earthquakes during this period.

THE TWIN BUILDINGS

The experimental twin buildings of Tohoku University are located in the engineering campus. They are each a 3-story concrete wall-frame building. One of the buildings is base isolated and the other has a conventional foundation. The typical floor plan as well as the vertical cross section of the buildings are shown in Fig. 1, with the location of accelerometers installed depicted as black circles. Constructing the buildings and instrumenting them was originally done to experimentally investigate the benefits of base isolation, but its database was found to be fulfilling the needs of the present study since the response of one building automatically includes the dynamic effects of the other building in this system.



(a) (b)

Figure 1: The twin buildings. (a) Typical floor plan. (b) Vertical cross-section. Note: black circles show the accelerometer location.

ANALYTICAL MODEL AND PARAMETERS

For the purposes of this study it is convenient enough to investigate an idealized system shown in Fig. 2. The 2D structural models shown in this figure consist each of a rigid roof at their top held by columns that are connected to the rigid foundations bonded to the surface of a homogeneous, viscoelastic half-space with its material damping ratio ξ_g, poisson ratio ν, and shear-wave velocity c_s (Fig. 2a). The supporting columns are taken as massless and presumed to respond within their elastic range of behavior in lateral direction. The clearance between the structures, edge to edge, is represented by the parameter $2a$. The mass of each roof and its corresponding rotational mass moment of inertia are denoted by m_i and I_i, respectively (i=1,2). The dimension, $2l_i$, of a structure is assumed to be the same as that of its underlying foundation. Effects of cross-interaction between the two structures are taken into account exactly through calculating the dynamic stiffness matrix and effective input motion vector of the coupled system of foundations. The former is defined as the coefficient matrix in the force-displacement relationship of the system consisting of massless adjacent rigid foundations only and the latter is response of this system to a harmonic ground excitation.

The structures are excited at their bases by earthquake ground motion. Each accelerogram used is supposed to be dominated by only one of the two types of waves: in-plane shear waves (SV-waves) propagating in vertical

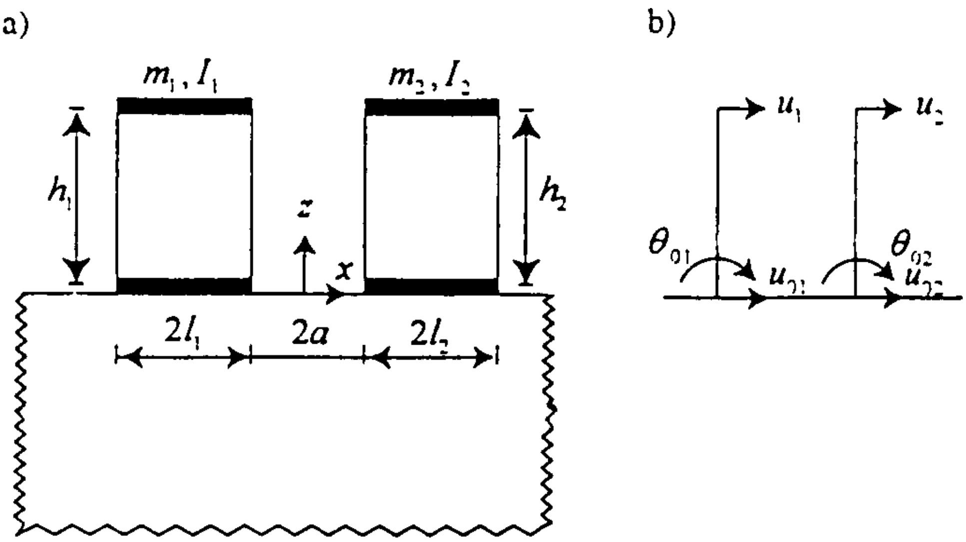


Figure 2: System considered. (a) Prameters involved. (b) Degrees of freedom.

direction, thus producing a horizontal free-field motion in x-direction; and surface Rayleigh waves (R-waves) producing horizontal as well as vertical free-field motions. The ground motion is assumed to vary randomly in space in addition to time.

DYNAMIC EQUATIONS OF MOTION

Equations of motion of the system of Fig. 2 are written in the frequency domain as follows: translation of each structural mass (m_1 and m_2) in *x*-direction; translation of each building as a whole in *x*-direction; and, rocking of each building as a whole about *y*-axis.

Contribution of the underlying soil to the above equations is in the form of interaction forces that are computed as Eq. (1):

$$\{P_k\}_{4\times 1} = [K]_{4\times 4}\{U_k\}_{4\times 1} \tag{1}$$

in which $\{P_k\}$, $\{U_k\}$and $[K]$ are as Eq. (2):

$$\{P_k\} = \{\{P_{k1}\}, \{P_{k2}\}\}^T, \{U_k\} = \{\{U_{k1}\}, \{U_{k2}\}\}^T,\ [K] = \begin{bmatrix} [K_{11}] & [K_{12}] \\ [K_{21}] & [K_{22}] \end{bmatrix} \tag{2}$$

and $\{P_{ki}\}$ is the vector of interaction forces applied on the foundation *i*, $\{U_{ki}\}$ is a vector having the translational and rocking degrees of freedom of the rigid foundation *i* as its elements, and, $[K]$ is the dynamic stiffness matrix of the coupled foundations. The submatrices $[K_{ij}]$ with $i \neq j$ contain the coupling terms between the adjacent foundations. Arranging the equations written as above, a system of equations in matrix form is resulted as of Eq. (3):

$$[A(\omega)]\{U(\omega)\} = \{P(\omega)\} \tag{3}$$

where $\{U(\omega)\}$ is an 6×1 vector of degrees of freedom of the total system given by Eq. (4) (refer to Fig. 2):

$$\{U(\omega)\} = \{u_1, u_{01}, h_1\theta_{01}, u_2, u_{02}, h_2\theta_{02}\} \tag{4}$$

The load vector $\{P(\omega)\}$ is consisted of appropriate combinations of elements of input motion vector of the foundations. Various elements of the system stiffness matrix $[A(\omega)]$ contain mass, damping and stiffness properties of the structures and soil.

VARIATION OF GROUND MOTION IN SPACE

The covariance matrix of the ground motion at a reference point in the site is defined as Eq. (5):

$$[C]=\{u_0^f\}\{\tilde{u}_0^f\}^T=\begin{bmatrix} u_0\tilde{u}_0 & u_0\tilde{w}_0 \\ w_0\tilde{u}_0 & w_0\tilde{w}_0 \end{bmatrix} \tag{5}$$

in which $\{u_0^f\}=\{u_0,w_0\}^T$ is the vector of free-field motion at the reference point containing horizontal u_0 and vertical w_0 components. At a different point i, the ground motion differs from that of the reference point presumably in the form of Eq. (6) (see [Wolf, 1985]):

$$\{u_i^f\}=f_i\,exp(-ikx_i)\{u_0^f\} \tag{6}$$

in which $i=\sqrt{-1}$, k is wave number, x_i is the distance of point i from the reference point, and f_i is the incoherency function defined in Eq. (7):

$$f_i=exp\left[-(\gamma\omega x_i/\beta)^2\right] \tag{7}$$

In Eq. (7), γ is the incoherency parameter which is between 0 and 0.5, ω is the excitation frequency, and β is the velocity of the incident wave. Now, the covariance matrix of the free-field ground motion between two arbitrary points i and j of the site can be written as Eq. (8):

$$[B]_{ij}=\{u_i^f\}\{\tilde{u}_j^f\}=\begin{bmatrix} u_i\tilde{u}_j & u_i\tilde{w}_j \\ w_i\tilde{u}_j & w_i\tilde{w}_j \end{bmatrix} \tag{8}$$

Combining Eqs. (5)-(7) with Eq. (8), the covariance matrix at every two points of the site is written as Eq. (9):

$$[B]_{ij}=[C]g_{ij}f_if_j \tag{9}$$

in which $g_{ij}=exp\left[-ik(x_i-x_j)\right]$. Now if the interface between foundations and soil is discretized into a number of elements so that there are N nodes in the system, it can be shown that Eq. (10) exists between the covariance matrix of the foundations' response $[B_0]$ and that of the free-field motion at N points in the site $[B]$:

$$[B_0]=[R][B][\tilde{R}]^T \tag{10}$$

in which $[R]$ is the matrix of transfer functions between free-field motion $\{u^f\}$ and foundations' response $\{U_0\}$, as:

$$\{U_0\}=[R]\{u^f\} \tag{11}$$

Combining Eqs. (9) and (10) results in Eq. (12):

$$B_{0ij}=\sum_{r=1}^{2}\sum_{s=1}^{2}A_{rs}^{ij}C_{rs},\ i,j=1,4 \tag{12}$$

in which:

$$A_{rs}^{ij}=\sum_{k=s,2}^{2(N-1)+s}\tilde{R}_{jk}\sum_{m=r,2}^{2(N-1)+r}R_{im}g_{m1k1}f_{m1}f_{k1}\ ,\ m1=\begin{cases} m/2: m\ even \\ (m+1)/2: m\ odd \end{cases},\ k1=\begin{cases} k/2: k\ even \\ (k+1)/2: k\ odd \end{cases} \tag{13}$$

A_{rs}^{ij} are transfer functions between power and cross spectra of the free-field motion at the reference point and of the foundation response. Therefore, $\sqrt{A_{rs}^{ij}}$ are transfer functions between response spectra of foundations and ground motion.

NUMERICAL IMPLEMENTATION

The structural model of Fig. 2 is analyzed for two free-field motions recorded near the twin buildings. These are called "earthquake 1" and "earthquake 2". Earthquake 1 occurred on 23.4.1987 with a PGA=91.3 gal while earthquake 2 happened on 6.2.1987 with a PGA=61.5 gal. These earthquakes have been selected because of two reasons. First, the analysis method is in elastic domain and the above PGA's are small enough to ensure an elastic behavior both within the structure and soil. Second, they are not too small to give a small signal/noise ratio. The calculated values are total acceleration spectra of base and roof motions which are compared with the recorded motions at the same places. For the sake of comparison in the dynamic analysis of the structure-soil-structure system, it is assumed that the ground motion each time is governed by one of the two types of waves: planar shear waves propagating in vertical direction, and Rayleigh waves. Also, an additional analysis is done assuming some random spatial variation in the ground motion according to Eq. (7) with taking $\gamma = 0.5$ which is an upper value.

Results of the analysis using earthquake 1 as the free-field motion are shown in Fig. 3. In the case of shear waves, Fig. 3a illustrates the Fourier amplitude of the horizontal motion at the base of the ordinary building. It is seen that the shear wave pattern resembles the recorded motion at low frequencies very good, but there is some overestimation in the amplitude at high frequencies. Also fluctuation of the analyzed motion is in harmony with

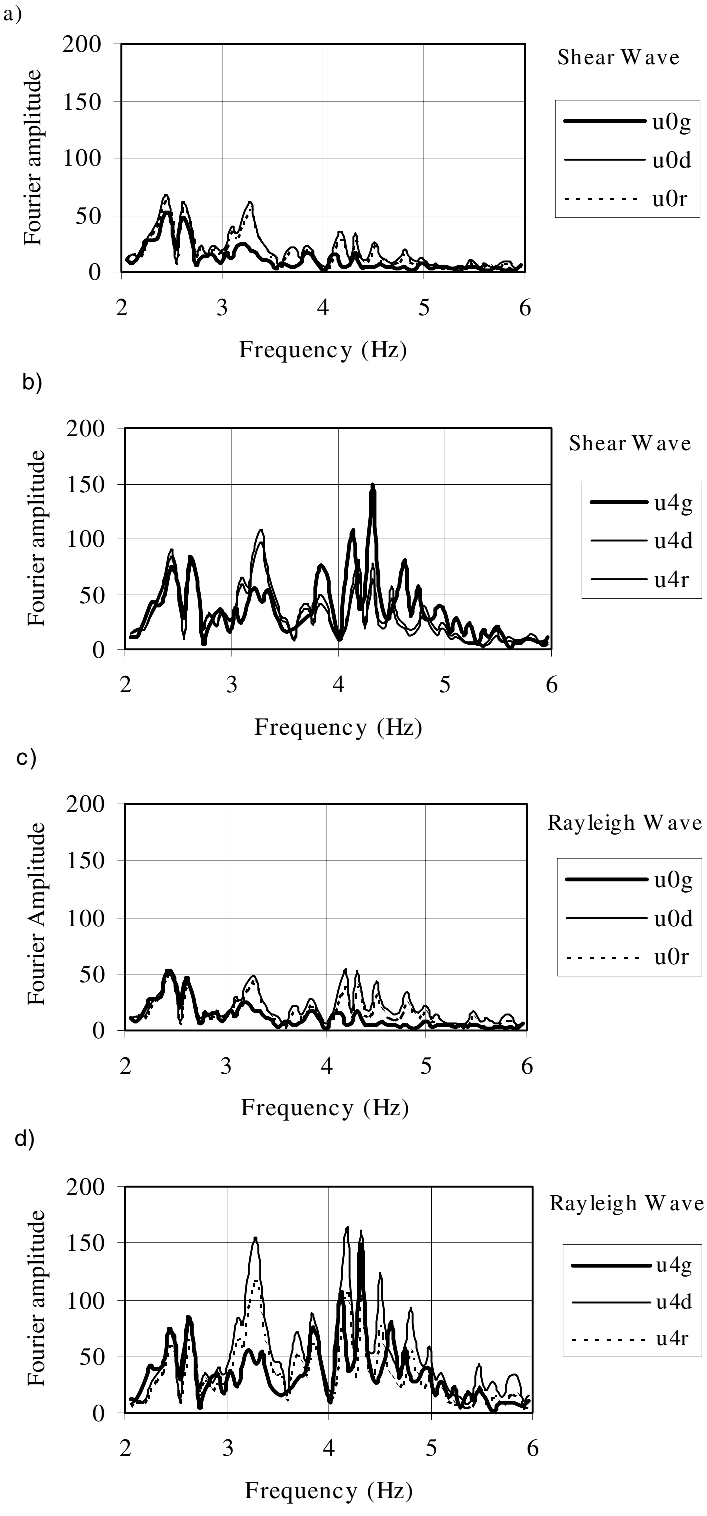


Figure 3: Acceleration spectra of the ordinary building subject to earthquake 1.
Note: index 1 is for base level and 4 for roof level; index g is for recorded spectrum, d for deterministically computed, and r for randomly computed spectra.

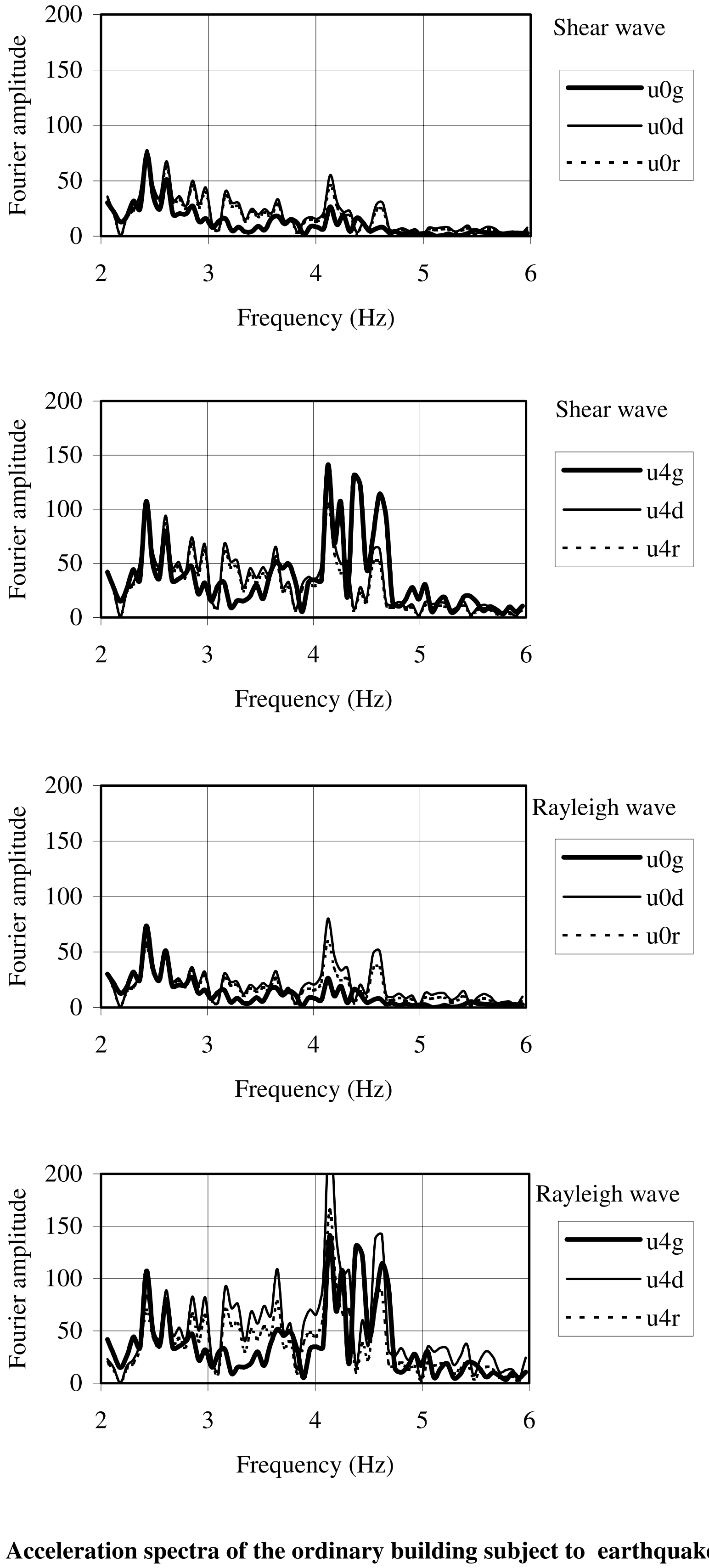


Figure 4: Acceleration spectra of the ordinary building subject to earthquake 2.
Note: index 1 is for base level and 4 for roof level; index g is for recorded spectrum, d for deterministically computed, and r for randomly computed spectra.

the recording almost exclusively. Values of the random analysis results are somewhat less that of the deterministic ones. In Fig. 3b the results of the analysis at the roof level are shown. Here again at small frequencies there is a very good agreement between the three sets of responses, however as the frequency increases, the agreement deteriorates. In Figs. 3c and 3d the analysis results for the propagation of Rayleigh waves are shown. As is seen, for small frequencies both at the base and roof levels very good agreement with the recorded motion is seen. At the roof level a better resemblance is seen almost for all the important frequency range except of an unwanted peak at a medium frequency. Again, if the spatial randomness of waves is taken into account, smaller response amplitudes are resulted which sometimes improve the accuracy of results.

In Fig. 4 the spectral responses subjected to earthquake 2 as the free-field motion are shown. In the two upper figures, namely Figs. 4a and 4b, the analysis results are shown under the incidence of shear waves. Similar to Fig. 3, a good agreement is seen for small frequencies both at the base and roof levels. At the roof level, the maximum response occurs at medium frequency where the agreement is poor with the shear wave pattern. Taking the spatially random characteristics of the wave propagation, seems not to affect the analysis results highly with shear waves. Turning to Figures 4c and 4d, a better agreement is seen between the analysis results and the recordings, specially at medium frequencies where the maximum response happens at the roof level. Again, effect of random spatial variation of ground motion is more highlighted with Rayleigh waves mostly by decreasing the response amplitude. Accounting for this phenomenon subjected to Rayleigh waves, tend to improve the accuracy of results at most frequencies in this case.

Overall, the Rayleigh wave pattern with spatial randomness seems to give the nearest values to the recorded motions.

CONCLUSIONS

Dynamic analysis of two adjacent structures accounting for cross-interaction between them was performed in this paper. The free-field motions used were the earthquake ground motions recorded near the buildings on the ground. In calculating the input motion to the foundations of the two buildings, two types of waves were used: first the vertically propagating shear waves, and second the horizontally propagating Rayleigh waves. Using an exponential function with a negative power, a certain level of incoherency was assumed in the ground motion which resulted in the variation of the excitation in space in addition to time.

Based on the dynamic analysis done, it is concluded that:

1. For the small range of frequencies, very good agreement is achieved between the calculated and recorded motions regardless of the wave type considered. For other frequencies, the Rayleigh wave pattern gives much better results. Taking into account the randomness of the waves improves the accuracy of the results in the latter case even more.

2. Although a simple one-degree-of-freedom system was used for representation of the structures, and a 2D soil-foundation system was considered to model the interaction effects, good agreement was achieved in most cases. This shows that the proposed model is suitable for engineering purposes.

3. The dynamic analysis for shear waves was done assuming vertical propagation of the waves. It seems necessary to redo this analysis for inclined shear waves to assess the effects of the horizontal propagation of the incident field.

REFERENCES

Hryniewicz, Z. (1993), "Dynamic response of coupled foundations on layered random medium for out-of-plane motion," *International Journal of Engineering Science*, 31, 2, pp221-228.

Imamura, A., Watanabe, T., Ishizaki, M., Motosaka, M. (1992), "Seismic response characteristics of embedded structures considering cross-interaction," *Proceedings of 10WCEE*, 3, pp1719-1724.

Lin, H. T., Roesset, J. M. and Tassoulas, J. L. (1987), "Dynamic interaction between adjacent foundations," *Earthquake Engineering and Structural Dynamics*, 15, pp323-344.

Luco, J. E. and Contesse, L. (1973), "Dynamic structure-soil-structure interaction," *Bulletin of Seismic Society of America*, 63, 1289-1303.

Qian, J., Tham, L. G. and Cheung, Y. K. (1996), "Dynamic cross-interaction between flexible surface footings by combined BEM & FEM," *Earthquake Engineering and Structural Dynamics*, 25, 509-526.

Wolf, J. P. (1985), "*Dynamic Soil-Structure Interaction*," Prentice-Hall, Inc.