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FUNDAMENTAL STUDY ON IDENTIFICATION OF SOIL-STRUCTURE INTERACTION SYSTEMS IN FREQUENCY DOMAIN

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ABSTRUCT

A frequency-domain approach of identification of soil-structure systems under earthquake excitations is presented. A single-story shear building resting on a half-space soil layer is used as an objective model in this primary study and the frequency dependent behavior of the soil layer is considered. The parameters of the upper structure and soil layer are identified independently by a parametric and nonparametric method, respectively. In nonparametric method, the elements of the impedance matrix of soil are approximated by power polynomials and then the constants of them are identified through least-squares approach. Two simulated numerical examples are presented to illustrate the effectiveness of this approach in dealing with soil-structure interaction models possessing frequency dependent or independent parameters.

1. INTRODUCTION

The effect of soil-structure interaction is recognized to be important for the design of earthquake-resistant structures and can not, in general, be neglected. The identification of such a kind of systems through the use of experimental data or response records will give substantial improvement in constituting the analytical model. Therefore, an effective and efficient identification approach will undoubtedly be needed. Until now few published papers concern with this problem.

An important fact has been realized that the soil-structure interaction impedance matrix is frequency dependent. Veletsos and Wei[1] and Luco and Westmann[2] suggested the theoretical expressions of the frequency dependent impedance matrix for a circular footing supported on the surface of a half-space soil layer. Wolf[3] summarized varieties of the impedance matrices for the surface structures and embedded structures and showed graphically the variation of them with the dimensionless frequency. Now a simplified sway-rocking model, which consists of two couples of frequency dependent (or independent) springs and dampers in the translational and the rotational, respectively, has been often used. Obviously, the frequency dependent characteristics of soil-structures interaction problems should be devoted more attention in the identification procedure.

Recognition of the effectiveness of the identification of structures in structural dynamics leads to many methods developed, which can be categorized into the time-domain and the

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frequency-domain. As the time-domain method is efficient in dealing with nonlinear structures or time-variant structures, the use of it has attracted the attention of researchers and has been increasing since the 1970s. This method can be further classified into two groups: parametric identification and nonparametric identification. The former seeks to determine the value of parameters in an assumed model of the structure to be identified, while the latter produces the best functional representation of the structure. Since the model structure in many practical problems is by no means clear, an increasing amount of attention has recently been devoted to nonparametric identification methods. Some literatures with regard to earthquake engineering problems are mentioned below.

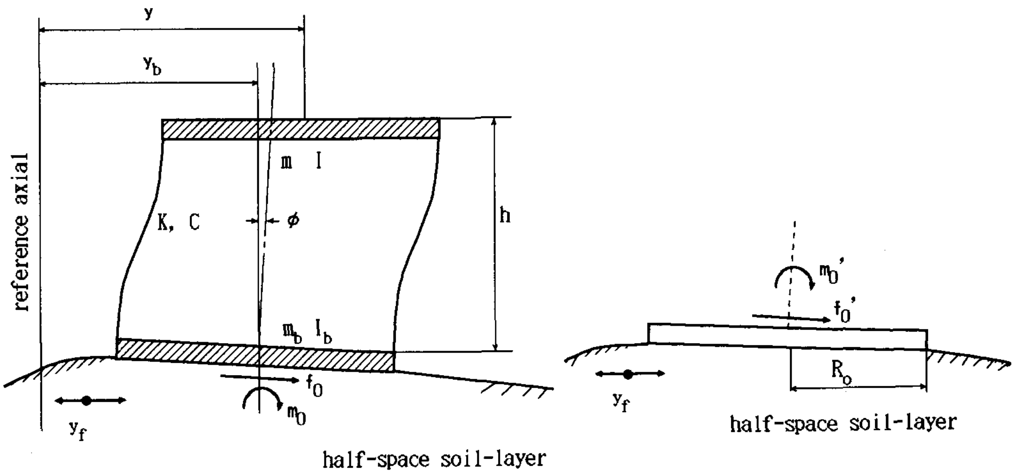
Udwadia and Marmarelis[4] were perhaps the first to apply nonparametric identification techniques, in which the Volterra-series, or Wiener-kernel, approach was employed to analyse the nonlinear properties of structures. Masri and Caughey developed a nonparametric identification method by approximating the restoring force in a series of orthogonal Chebyshev polynomials [5, 6]. An analogous method by expanding the nonlinear restoring force function in a power polynomial was presented by Toussi, S. and Yao, J. T. P. [7] and Iwan, W. D. and Peng, C. Y. [8]. The main features of nonparametric identification procedure generally are: (1) a priori assumption about the system model is almost not needed or not strictly required, (2) it is applicable to linear, nonlinear and hysteretic systems with relatively minimal requirements of computation time and storage and (3) the intial guesses of the parameters to be identified are generally not needed.

The frequency domain methods have been widely used for some time and have proved to be efficient methods in many cases, of which Udwadia et al. [9] and McVerry's research[10] are probably more worth to be mentioned. Recently, Sawada, et al. [11] developed a substructure approach in frequency domain to deal with the structural model with many degree-of-freedom. These methods may belong to the parametric category if classified analogous to the time domain since they seek to determine the parametric values of the structural models. For identification of a soil-structure interaction system taking its frequency dependent characteristics into account, the frequency-domain method is perhaps the only effective means and the nonparametric methods used in time-domain are very useful for reference. Similarly the best functional representation of the structural charateristics rather than the values of the modal parameters can be produced in frequency domain from the recorded response records. Therefore the development of the non-parametric identification method in frequency domain was stimulated.

The objective of this paper is to develope an approach for identification of soil-structure interactin systems, which are frequency dependent, under earthquake excitations. A single-story shear building resting on a half-space soil layer is selected as an illustrative model. The stiffness and damping coefficients in the impedance matrix of the soil layer are approximated by power polynomials and the constants of them are then identified through least-squares approach. In order to reduce the number of the constants or parameters to be estimated, the whole specified frequency range is divided into several intervals and the identification procedure is repeated in each interval. Two simulated numerical examples are presented to illustrate the effectiveness of this approach in dealing with frequency dependent or independent soil-structure interaction models.

2. EQUATIONS OF MOTION OF A SOIL-STRUCTURE SYSTEM

Dynamic analysis methods of a soil-structure system had been well developed[3]. Without loss of generality, a single-story shear structure-foundation system (**Fig. 1**) is used to illustrate the method. The system is supported on a deep layer of assumed elastic soil and subjected to a horizontal free-field seismic wave that is modified at the structure base due to the presence



a) substructure 1: structure-foundation b) substructure 2: soil-layer

Fig.1 Soil-structure interaction model

of the top structure.

The equations of motion for the system is formulated through substructuring, i.e., the structure-foundation (**Fig.1 a)**) and the soil layer(**Fig.1 b)**). The equations of motion in frequency domain for the structure-foundation substructure(**Fig.1a**) are:

$$(-m\omega^2+iC\omega+K)Y(i\omega)=(i\omega C+K)[Y_b(i\omega)+h\Phi(i\omega)] \quad (1a)$$
$$-\omega^2[mY(i\omega)+m_bY_b(i\omega)]=F_o(i\omega) \quad (1b)$$
$$-\omega^2[mhY(i\omega)+(I+I_b)\Phi(i\omega)]=M_o(i\omega) \quad (1c)$$

in which m=top mass of the structure; C=viscous damping of the structure; K=flexural stiffness of the structure; I=the moment of inertia of m for rocking; m_b=mass of the rigid base; I_b=the moment of inertia of m_b for rocking; h=the story height; $Y(i\omega)$= Fourier transform of y(t)(the absolute displacement of the top mass); $Y_b(i\omega)$=Fourier transform of $y_b(t)$(the absolute displacement of the base); $\Phi(i\omega)$= Fourier transform of $\phi(t)$(the rotation of the base); $M_0(i\omega)$= Fourier transform of $m_0(t)$(interaction rocking moment at the interface); $F_0(i\omega)$=Fourier transform of $f_0(t)$(interaction translation force at the interface). ω is the circular frequency.

The effect of soil on the structure-foundation can be expressed in terms of impedance matrix of soil that has been derived by many researchers as mentioned before[1,2,3]. Veletsos and Wei's expression[1] may be much more representative and is in the form (see **Fig.1 b)**):

$$\begin{Bmatrix} F_0'(i\omega) \\ M_0'(i\omega) \end{Bmatrix} = \begin{bmatrix} f_{ss}(ia_o) & 0 \\ 0 & f_{rr}(ia_o) \end{bmatrix} \begin{Bmatrix} Y_b(i\omega)-Y_f(i\omega) \\ \Phi(i\omega) \end{Bmatrix} \quad (2a)$$

with

$$f_{ss}(ia_o)=[K_{11}(a_o)+ia_oC_{11}(a_o)]K_s \quad (2b)$$
$$f_{rr}(ia_o)=[K_{22}(a_o)+ia_oC_{22}(a_o)]K_r \quad (2c)$$

where K_s and K_r are the constants that are defined as $K_s=8GR_0/(2-\nu)$ and $K_r=8GR_0^3/3(1-\nu)$, in which G=the shear modulus of soil, ν=Poisson's ratio of soil and R_0=the dimension of the base; $Y_f(i\omega)$ is the Fourier transform of free-field displacement $y_f(t)$; a_0 is the dimensionless frequency defined as $a_0=\omega R_0/(G/\rho)^{1/2}$ in which ρ is the mass density of soil; the dimensionless stiffness coefficients $K_{11}(a_0)$ and $K_{22}(a_0)$ and the dimensionless damping coefficients $C_{11}(a_0)$ and $C_{22}(a_0)$ are frequency dependant and the variation of them with the dimensionless frequency a_0 are shown in **Fig. 2**. For a system with specified soil parameters (G, ν, ρ) and base dimension (R_0), $f_{ss}(ia_0)$ and $f_{rr}(ia_0)$ can be written as $f_{ss}(i\omega)$ and $f_{rr}(i\omega)$.

Based on the condition of force continuity in the interface, Eqs. (1b) and (1c) can be combined with Eqs. (2) as follows:

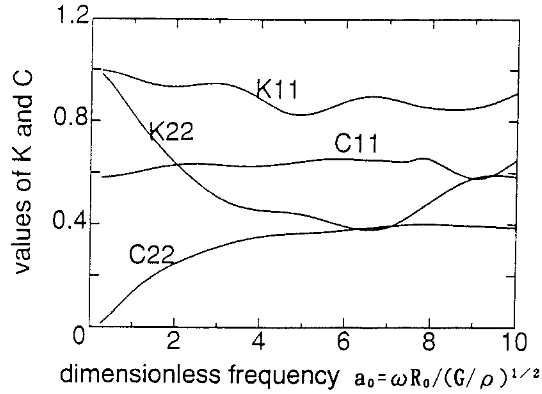


Fig. 2 Frequency dependent dimensionless K and C for $\nu=1/3$ [1]

$$\omega^2[mY(i\omega)+m_bY_b(i\omega)]=f_{ss}(i\omega)[Y_b(i\omega)-Y_f(i\omega)] \tag{3a}$$

$$\omega^2[mhY(i\omega)+(I+I_b)\Phi(i\omega)]=f_{rr}(i\omega)\Phi(i\omega) \tag{3b}$$

Eqs. (1a), (3a) and (3b) can be used to perform dynamic analysis or generate the response records for identification purpose.

3. IDENTIFICATION PROCEDURE

The parameters to be identified for this soil-structure interaction system can be separated into structural parameters (stiffness coefficient K and damping coefficient C) and the parameters of the soil layer (the dimensionless stiffness coefficients, $K_{11}(\omega)$ and $K_{22}(\omega)$, and damping coefficients, $C_{11}(\omega)$ and $C_{22}(\omega)$) that are frequency dependent. From Eqs. (1a), (3a) and (3b), it is seen that they can be identified independently if the responses of the top mass and the foundation are recorded. The absolute displacement of the base y_b and the rotation ϕ are used as the inputs for the top structure-foundation subsystem and meanwhile as the outputs for the soil layer. The output for the top structure-foundation is the response of the top mass y and the input for the soil is the prescribed free-field seismic motion y_f.

(1) For the Top Structure-foundation: Parametric Identification

For the top structure-foundation system, the identification of parameters is quite simple as their characteristics of frequency independent. In most practical situation, the Fourier transform integrals must be calculated by their discrete version, such as FFT, at the discrete frequency ω_j. From Eq. (1a), it can be obtained that:

$$i\omega_jC+K=\omega^2mY(i\omega_j)/[Y(i\omega_j)-Y_b(i\omega_j)-h\Phi(i\omega_j)]$$
$$=G(i\omega_j)=Re[G(i\omega_j)]+iIm[G(i\omega_j)] \qquad (j=1\sim N) \tag{4}$$

where N indicates the number of the sampling points. In every sampling point, there exists a deviation between the both side of Eq. (4):

$$e_j(C,K)=\{K-Re[G(i\omega_j)]\}+i\{\omega_jC-Im[G(i\omega_j)]\}=e_j(K)+ie_j(C) \qquad (j=1\sim N) \tag{5}$$

Using least-square method to mininize the deviations of $e_j(K)$ and $e_j(C)$ seperately over all the sampling points results in the estimated stiffness coefficient $\hat{k}$ and damping coefficient $\hat{c}$:

$$\hat{k}=\frac{1}{N}\sum_{j=1}^{N}\mathrm{Re}\,[G\,(i\omega_j)\,] \qquad (6a); \qquad \hat{c}=\sum_{j=1}^{N}\mathrm{Im}\,[G\,(i\omega_j)\,]\,\omega_j/\sum_{j=1}^{N}\omega_j^2 \qquad (6b)$$

(2) For the Soil Layer: Nonparametric Identification

From **Fig.2**, it can be seen that the coefficients $K_{11}(\omega)$, $K_{22}(\omega)$, $C_{11}(\omega)$ and $C_{22}(\omega)$ are single-valued function of frequency. These observations suggest that the coefficients can be approximated with polynomials of the following form:

$$K_{11}(\omega)=\sum_{k=1}^{p}a_k\omega^{k-1} \qquad (7a); \qquad C_{11}(\omega)=\sum_{k=1}^{q}b_k\omega^{k-1} \qquad (7b)$$

$$K_{22}(\omega)=\sum_{k=1}^{r}d_k\omega^{k-1} \qquad (7c); \qquad C_{22}(\omega)=\sum_{k=1}^{s}e_k\omega^{k-1} \qquad (7d)$$

in which p, q, r and s indicate the degrees of the polynomials. Once the constants a_k ($k=1\sim p$), b_k($k=1\sim q$), d_k($k=1\sim r$) and e_k ($k=1\sim s$) are identified through least-squares approach, the best functional representation of the soil properties that are frequency dependent can be obtained. Certainly, Eqs.(7) is applicable to the frequency independent case just by setting p, q, r and s to be one. Dividing the whole frequency range into several intervals and approximating the coefficients within each interval lead to the degrees of the polynomials reduced greatly so that the number of constants to be identified reduced.

Substituting Eqs.(7a) and (7b) into Eq.(3a) and keeping in mind that the Fourier transform integrals are calculated practically at the discrete frequency ω_j($j=1\sim N_j$, where N_j is the number of samples within each interval), the following expression is obtained

$$\sum_{k=1}^{p}a_k\omega_j^{k-1}+i\,\omega_j\sum_{k=1}^{q}b_k\omega_j^{k-1}=\omega_j^2\,[mY\,(i\omega_j)+m_bY_b\,(i\omega_j)\,]\,/\,[Y_b\,(i\omega_j)-Y_f\,(i\omega_j)\,]\,k_s$$
$$=F_1\,(i\omega_j)=\mathrm{Re}\,[F_1\,(i\omega_j)\,]+i\,\mathrm{Im}\,[F_1\,(i\omega_j)\,]\,; \quad (j=1\sim N_j) \qquad (8a)$$

Analogously, substituting Eqs.(7c) and (7d) into Eq.(3b) yields:

$$\sum_{k=1}^{r}d_k\omega_j^{k-1}+i\,\omega_j\sum_{k=1}^{s}e_k\omega_j^{k-1}=\omega_j^2\,[mhY\,(i\omega_j)+(I+I_b)\,\Phi\,(i\omega_j)\,]\,/\,\Phi\,(i\omega_j)\,k_r$$
$$=F_2\,(i\omega_j)=\mathrm{Re}\,[F_2\,(i\omega_j)\,]+i\,\mathrm{Im}\,[F_2\,(i\omega_j)\,]\,; \quad (j=1\sim N_j) \qquad (8b)$$

For the sake of simplicity, Eqs.(8) can be expressed as vector-matrix form:

$$\mathbf{H}_a\boldsymbol{\alpha}_a+i\,\mathbf{H}_b\boldsymbol{\alpha}_b=\mathbf{V}_a+i\,\mathbf{V}_b \qquad (9a); \qquad \mathbf{H}_d\boldsymbol{\alpha}_d+i\,\mathbf{H}_e\boldsymbol{\alpha}_e=\mathbf{V}_d+i\,\mathbf{V}_e \qquad (9b)$$

whrer

$$\mathbf{H}_a=\begin{bmatrix}1 & \omega_1\cdots\omega_1^{p-1}\\ 1 & \omega_2\cdots\omega_2^{p-1}\\ \cdots & \cdots\\ 1 & \omega_{Nj}\cdots\omega_{Nj}^{p-1}\end{bmatrix} \qquad (10a); \qquad \mathbf{H}_b=\begin{bmatrix}\omega_1 & \omega_1^2 & \cdots & \omega_1^q\\ \omega_2 & \omega_2^2 & \cdots & \omega_2^q\\ \cdots & & & \\ \omega_{Nj} & \omega_{Nj}^2 & \cdots & \omega_{Nj}^q\end{bmatrix} \qquad (10b)$$

$$\mathbf{H}_d=\begin{bmatrix}1 & \omega_1\cdots\omega_1^{r-1}\\ 1 & \omega_2\cdots\omega_2^{r-1}\\ \cdots & \cdots\\ 1 & \omega_{Nj}\cdots\omega_{Nj}^{r-1}\end{bmatrix} \qquad (10c); \qquad \mathbf{H}_e=\begin{bmatrix}\omega_1 & \omega_1^2 & \cdots & \omega_1^s\\ \omega_2 & \omega_2^2 & \cdots & \omega_2^s\\ \cdots & & & \\ \omega_{Nj} & \omega_{Nj}^2 & \cdots & \omega_{Nj}^s\end{bmatrix} \qquad (10d)$$

$$\boldsymbol{\alpha}_a=(a_1,a_2,\cdots a_p)^T \quad (10e); \qquad \boldsymbol{\alpha}_b=(b_1,b_2,\cdots b_q)^T \quad (10f)$$
$$\boldsymbol{\alpha}_d=(d_1,d_2,\cdots d_r)^T \quad (10g); \qquad \boldsymbol{\alpha}_e=(e_1,e_2,\cdots e_s)^T \quad (10h)$$
$$\mathbf{V}_a=(Re[F_1(i\omega_1)],Re[F_1(i\omega_2)],\cdots,Re[F_1(i\omega_{Nj})])^T \quad (10i)$$
$$\mathbf{V}_b=(Im[F_1(i\omega_1)],Im[F_1(i\omega_2)],\cdots,Im[F_1(i\omega_{Nj})])^T \quad (10j)$$
$$\mathbf{V}_d=(Re[F_2(i\omega_1)],Re[F_2(i\omega_2)],\cdots,Re[F_2(i\omega_{Nj})])^T \quad (10k)$$
$$\mathbf{V}_e=(Im[F_2(i\omega_1)],Im[F_2(i\omega_2)],\cdots,Im[F_2(i\omega_{Nj})])^T \quad (10\ell)$$

Taking a sufficient number of the samples within each frequency interval results in a situation where the number of the sum of p and q or r and s is less than the number of samples. In other words, the sum, Mj=p+q or Mj=r+s, is less than the number of the sampling point, Nj, within each interval. Because there are more equations than the number of unknowns, no solution can satisfy all of the simultaneous equations and, consequently, it would be inappropriate to consider Eqs. (9) as equal. Therefore, Nj x 1 error vector, $\mathbf{e}$'s, are introduced:

$$\mathbf{e}(\boldsymbol{\alpha}_a\boldsymbol{\alpha}_b)=\mathbf{H}_a\boldsymbol{\alpha}_a-\mathbf{V}_a+i(\mathbf{H}_b\boldsymbol{\alpha}_b-\mathbf{V}_b)=\mathbf{e}_a+i\,\mathbf{e}_b \quad (11a)$$
$$\mathbf{e}(\boldsymbol{\alpha}_d\boldsymbol{\alpha}_e)=\mathbf{H}_d\boldsymbol{\alpha}_d-\mathbf{V}_d+i(\mathbf{H}_e\boldsymbol{\alpha}_e-\mathbf{V}_e)=\mathbf{e}_d+i\,\mathbf{e}_e \quad (11b)$$

That results in a classical least-squares problem. If using the weighted least-squares approach, the constants vectors, $\boldsymbol{\alpha}$'s, are estimated by minimizing the following quantities:

$$\mathbf{E}_a=\mathbf{e}_a^T\mathbf{Q}_a\mathbf{e}_a \quad (12a); \qquad \mathbf{E}_b=\mathbf{e}_b^T\mathbf{Q}_b\mathbf{e}_b \quad (12b)$$
$$\mathbf{E}_d=\mathbf{e}_d^T\mathbf{Q}_d\mathbf{e}_d \quad (12c); \qquad \mathbf{E}_e=\mathbf{e}_e^T\mathbf{Q}_e\mathbf{e}_e \quad (12d)$$

in which $\mathbf{Q}$'s= a symmetrical, N_j x N_j, nonsingular and often diagonal or unit matrix. Setting the derivative of $\mathbf{E}$'s in Eqs. (12) with respect to vector $\boldsymbol{\alpha}$'s equal to zero results in estimates of vector $\boldsymbol{\alpha}$'s as follows (the detail procedure can be refered to Reference[12]):

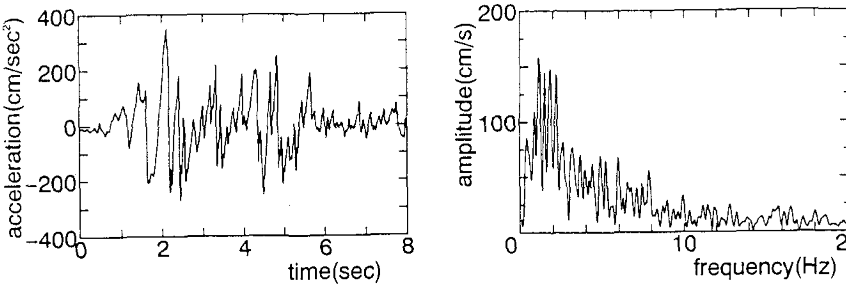
$$\hat{\boldsymbol{\alpha}}_a=(\mathbf{H}_a^T\mathbf{Q}_a\mathbf{H}_a)^{-1}\mathbf{H}_a^T\mathbf{Q}_a\mathbf{V}_a \quad (13a); \qquad \hat{\boldsymbol{\alpha}}_b=(\mathbf{H}_b^T\mathbf{Q}_b\mathbf{H}_b)^{-1}\mathbf{H}_b^T\mathbf{Q}_b\mathbf{V}_b \quad (13b)$$
$$\hat{\boldsymbol{\alpha}}_d=(\mathbf{H}_d^T\mathbf{Q}_d\mathbf{H}_d)^{-1}\mathbf{H}_d^T\mathbf{Q}_d\mathbf{V}_d \quad (13c); \qquad \hat{\boldsymbol{\alpha}}_e=(\mathbf{H}_e^T\mathbf{Q}_e\mathbf{H}_e)^{-1}\mathbf{H}_e^T\mathbf{Q}_e\mathbf{V}_e \quad (13d)$$

The same procedure just discribed is repeated for all of the interested frequency intervals and then the best functional representations of the elements in the frequency dependent impedance matrix can be produced.

4. NUMERICAL EXAMPLES

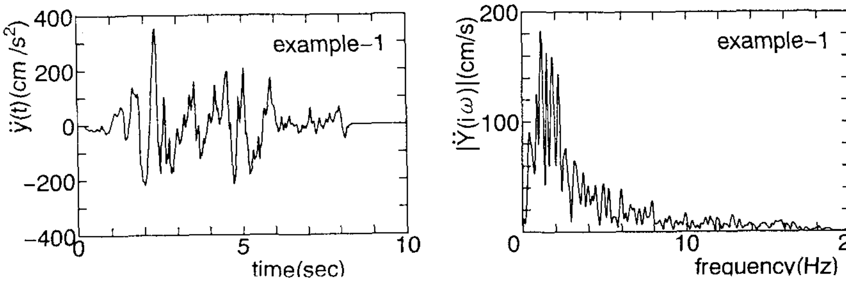
A program based on the method just discribed is developed and loaded in personal computers of NEC 9801 series. Two numerical examples are used to investigate the efficiency and the accuracy of the proposed method. It is assumed that the acceleration responses of the structure(i.e., acceleration responses of the top mass and the base)as well as the free-field acceleration are observable. The Fourier transforms used in this method can be calculated through FFT. The dimensionless soil stiffness and damping coefficients are at first assumed to be constant in the example one, and then to be frequncy dependent in the example two.

The first 8 sec of acceleration history of the El Centro earthquake (Imperial valley, 1940, shown in **Fig. 3a**)), which includes the components of wider frequencies (**Fig. 3b**)), is used as the free-field seismic acceleration. It is sampled at a constant interval $\triangle t$=0.02sec and added with trailing zeros resulting in 512 samples. The measured responses are generated from Eqs. (1a), (3a) and (3b) on the basis of the assumed known parameter values and the aseismic input. The model of a single-story shear structure-foundation system (as shown in **Fig. 1**) possesses parameters of: the top floor mass $m=4.5\times10^3$kg; the moment of inertia of m for rocking $I=5.0\times10^4 kg\cdot m^2$; the mass of rigid base $m_b=1.6\times10^4$kg; the moment of inertia of m_b for rocking $I_b=1.2\times10^4 kg\cdot m^2$; the



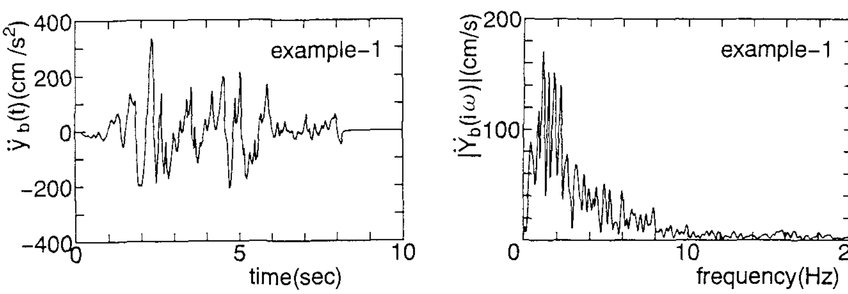
a) Time history　　　　　　b) Fourier spectrum

Fig. 3 El Centro earthquake (N-S, Imperial valley, 1940)



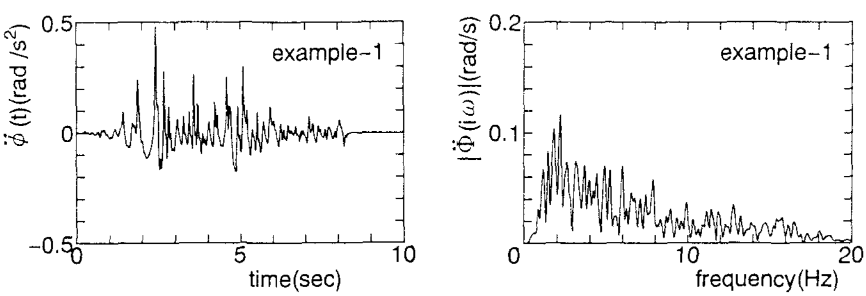
a) Time history　　　　　　b) Fourier spectrum

Fig. 4 Response of the top mass



a) Time history　　　　　　b) Fourier spectrum

Fig. 5 Response of the base



a) Time history　　　　　　b) Fourier spectrum

Fig. 6 Response of the rotation

Table 1. The results for the case of constant soil coefficients

parameters	true value	all sampls used			half samples used		
		estimated value	error (%)	R.M.S. error	estimated value	error (%)	R.M.S. error
K	1.80E4	1.80E4	0.00	1.3E-4	1.80E4	0.00	2.3E-4
C	2.10E1	2.10E1	0.00	3.2E-5	2.10E1	0.00	5.4E-5
K_{11}	1.0230	1.0188	0.39	5.5E-1	1.0228	0.00	2.8E-4
C_{11}	0.2846	0.2846	0.00	1.1E-4	0.2846	0.00	1.5E-4
K_{22}	1.0500	1.0445	0.51	4.5E-1	1.0500	0.00	2.2E-1
C_{22}	0.2000	0.2000	0.00	3.6E-2	0.2000	0.00	5.0E-2

story height h=3.5m. The structural stiffness and damping coefficients: $K=1.8\times10^4$kN/m and C=21.0 kN・s/m.

Example-1

The true values of the constant dimensionless stiffness and damping coefficients of soil are: $K_{11}=1.023$ and $C_{11}=0.2846$ for the traslational; $K_{22}=1.050$ and $C_{22}=0.2000$ for the rotational. The generated responses and their Fourier amplitude spectra are shown in **Fig.4,5** and **6**. The identified values, their percentage error with respect to the true values and the root-mean-square error(R.M.S.error) of the diviationds (in Eqs.(5) and (11))are presented in **Table 1**.

At first,the sample points of responses from zero frquency to the folding frequency (Nyquist frequency) are all used in the identification. The corresponding percentage errors are almost zero except for K_{11} and K_{22}. These errors arise perhaps due to the smaller Fourier amplitudes of the eartquake excitation and the effect of aliasing or overlapping in the higher frequency range, which exists unavoidably in the discrete frequency-domain dynamic analysis. Then the sample points from zero frequency to the half of the folding frequency are used and all of the percentage errors achieve zeros.

Example-2

The numerical data of dimensionless stiffness and damping coefficients of soil layer given in **Fig.2** are calculated out for: the Poisson' ratio $\nu=1/3$, the shear modulus of the material of soil G=172 kN/m^2, the mass density $\rho=1.5$T/m^3, and the dimension of the base $R_0=2.7$m. Four polynomials with same degree of order, p=q=r=s=3, for $K_{11}(\omega)$, $K_{22}(\omega)$, $C_{11}(\omega)$ and $C_{22}(\omega)$, respectively, are used to approximated these data over a range of frequency from 0 to 20Hz which is divided into four intervals: 0～5Hz, 5～10Hz, 10～15Hz and 15～20Hz,and the corresponding constants of the polynomials are presented in the second column of **Table 2**. These constants are assumed to be the true values, from which the responses are generated and then identification is carried out to get the estimations of them. The percentage errors of the estimated constants with respect to the assumed true values are shown in the third column of **Table 2**.

It can be found that the percentage errors are all less than 2% and the trend of their increase is from lower frequency range to the higher.This fact may be explained by the smaller Fourier amplitudes of the earthquake excitation and the effect of aliasing or overlapping in

Table 2. The constants of the approximating polynomial

polynomial constants	true values				percentage error of estimatmations (%)			
	frequency intervals(Hz)				frequency intervals(Hz)			
	0∼5	5∼10	10∼15	15∼20	0∼5	5∼10	10∼15	15∼20
a_1	.1023E1	.8834E0	-.2580E0	.2808E1	.00	.02	.36	1.26
a_2	-.4030E-1	.3420E-1	.1729E0	-.2327E0	.01	.11	.06	1.67
a_3	.4700E-2	-.4100E-2	-.6500E-2	.6900E-2	.01	.07	.10	1.67
b_1	.2846E0	.3762E0	.2528E0	.1738E1	.00	.03	.22	.99
b_2	.9000E-2	-.1760E-1	.1180E-1	-.1628E0	.09	.13	.81	1.22
b_3	-.4000E-3	.1200E-2	-.5000E-3	.4600E-2	.27	.18	.63	1.25
d_1	.1050E1	.1006E1	.1735E1	-.2641E1	.00	.07	1.32	.65
d_2	-.1230E0	-.1227E0	-.2126E0	.3353E0	.03	.15	1.75	.16
d_3	.5100E-2	.6700E-2	.8400E-2	-.8700E-2	.21	.15	1.73	.13
e_1	-.196E-1	.1990E-1	.1602E0	.1514E0	.01	.12	.18	.80
e_2	.5100E-1	.3210E-1	.1400E-2	.6800E-2	.00	.00	1.60	1.80
e_3	-.3800E-2	-.1600E-2	.1000E-3	-.2000E-3	.00	.03	1.64	1.42

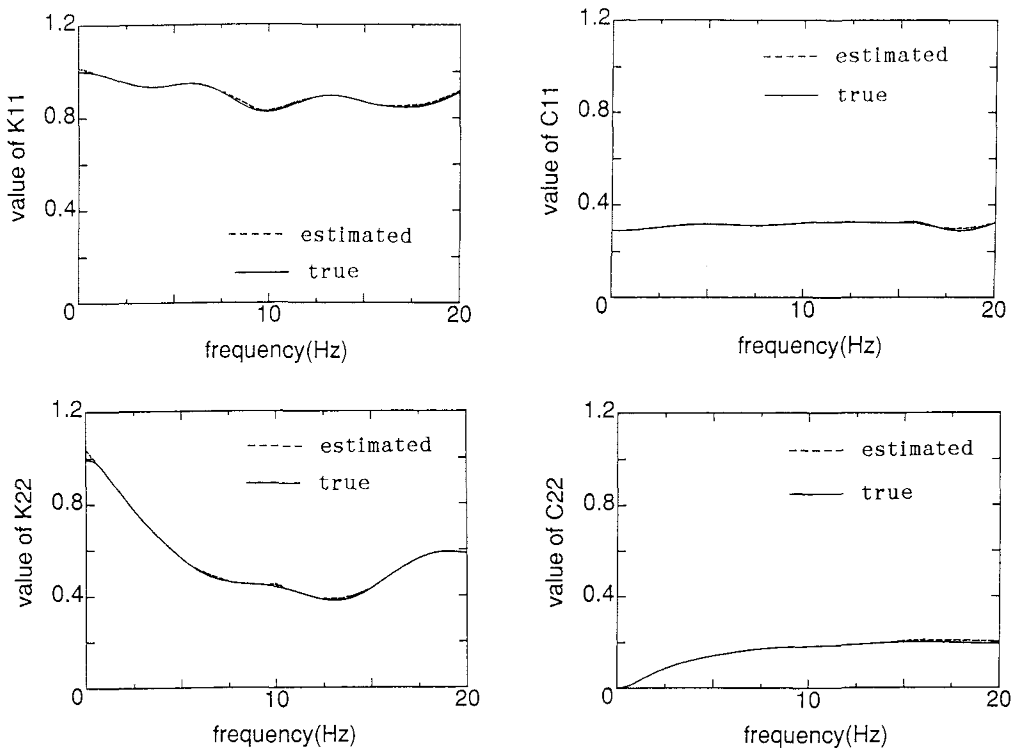


Fig. 7 Comparisons of estimated functions of K_{11}, C_{11}, K_{22} and C_{22} with the true ones

the higher frequency range as same as in the Example one. **Fig. 7** shows the comparisons of the estimated functions of $K_{11}(\omega)$, $K_{22}(\omega)$, $C_{11}(\omega)$ and $C_{22}(\omega)$ with the assumed true ones.

5. CONCLUDING REMARKS

A frequency-domain method for identification of soil-structure interaction systems is proposed. The method offers several features:

1)It is applicable to the soil-structure interaction systems with frequency independent or dependent parameters.

2)The initial guesses for the parameters to be identified are not necessary.

3)Both computation and storage requirements are relatively minimal.

4)The numerical examples show that identification achieves higher accuracy(for the frequency independent model, errors are 0%; for the frequency dependent model, errors are less then 2%).

5)This method can be directly extended to MDOF upper structure systems as the idependent indetification for the upper structure and soil.

Additioanal research is needed to assess followig problems:

(1)The influences of measurement noise, which is unavoidable in practical situations.

(2)The singularity of the inverse of matrix $\mathbf{H}^T\mathbf{QH}$ (in Eqs.(13)) that is expected to arise as the degree of the polynomials becomes larger in some cases.

(3)The effect of aliasing or overlapping existing in the frequency-domain dynamic analysis using discrete Fourier transforms, which may introduce unacceptable errors in the higher frequency range.

(4)The coupling terms for swaying and rocking in the impedance matrix that can not be neglected in some cases.

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