지진계측자료를 사용한 화련 지반-구조물 상호작용계의 미지계수 추정

Identification of the Hualien Soil-Structure Interaction System Using Earthquake Response Data

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국문요약

본 논문에서는 지반-구조물 상호작용계의 강성에 관련된 물성값들을 지진계측결과를 바탕으로 효과적으로 추정할 수 있는 방법에 대해 연구하였으며, 제안된 방법의 검증은 국제공동 연구의 일환으로 최근 대만의 화련에 건설된 대형지진시험구조물에서 계측된 지진 응답을 사용하여 수행하였다. 지반-구조물 상호작용계의 지진응답해석을 위해서 구조물과 근역지반은 축대칭 유한요소로 모형화하고 원역지반은 축대칭 무한요소를 사용하였으며, 이때 입력 지진하중은 부구조법에 근거한 파입력기법이 고려되었다. 지진계측결과를 사용하여 각 영역의 물성값을 제약적 최속강하법을 사용하여 추정하였는데, 추정된 계수들을 사용하여 계산된 지진응답이 계측치와 매우 잘 일치하여, 추정결과의 타당성을 검증할 수 있었다.

1. Introduction

An international joint research among researchers from Korea, Japan, Taiwan, USA and France has been conducted to investigate soil-structure interaction (SSI) effects during strong earthquake events and to verify SSI analysis computer programs which have been proposed by the participants. A large scale seismic test (LSST) structure was constructed in Taiwan, where the seismic activity is high. It is a 1/4-scale model of a nuclear power containment building as shown in Figure 1 [1]. A series of forced vibration tests (FVT) were carried out, and the test results were used for enhancing the soil and structural parameters used in the analysis model [2]. Then the ground accelerations and the structural responses were measured during several earthquake events.

This paper demonstrates how system identification techniques can be successfully applied to a soil-structure interaction system using the results of the earthquake response data only without FVT. The parameters identified are the shear moduli of several near-field soil regions as well as Young's moduli of the shell sections of the

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structure. The error function for the parameter estimation is constructed using the complex horizontal frequency response function obtained at the top of the structure. The complex horizontal frequency response function is calculated using the linear relationship between the input motion (horizontal response data measured at the free field) and the output motion (horizontal response data measured at the top of structure). The constrained steepest descent method is employed to obtain the revised parameters. Then parameters are identified for NS-direction based on the earthquake response data. The simulated earthquake responses using the identified parameters for the Hualien soil-structure interaction system are in excellent agreement with the observed response data.

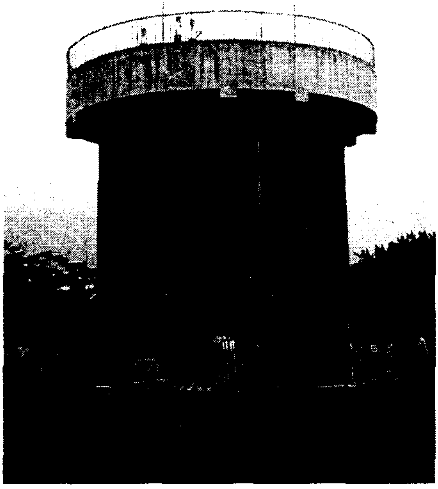


Figure 1 Hualen LSST structure

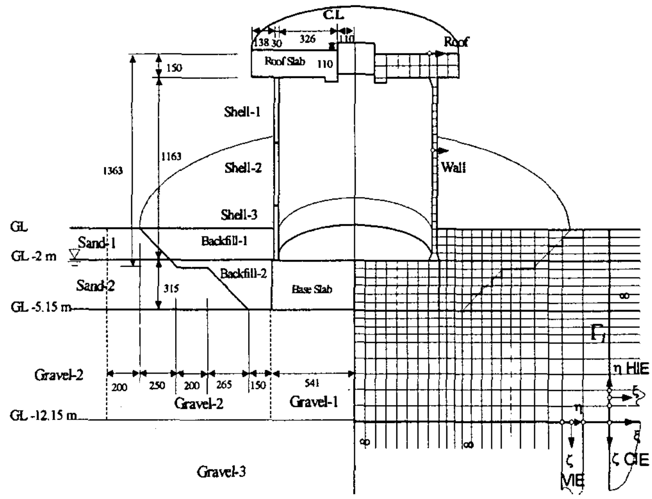


Figure 2 Modeling of the Hualien soil-structure system

2. Unified Model

For a benchmark comparison of the various SSI analysis techniques proposed methods by the consortium members [1], a unified soil model was constructed using the data from the geological exploration and in-situ tests carried out by CRIEPI of Japan [3]. The stiffness properties of the containment structure were evaluated from the design drawings and the values recommended by Taiwan Power Company (TPC) [2]. The parameters of the unified model are shown in Table 1.

3. Finite-Infinite Element Earthquake Response Analysis

The soil-structure interaction is a very complicated phenomenon requiring an analytical model with a certain level of sophistication for a meaningful earthquake response analysis. In Particular, frequency-dependent characteristics and the effect of radiation damping due to the unbounded nature of far-field soil medium should at least be incorporated. To that end, the axisymmetric finite-infinite element model used in the FVT correlation analysis is utilized in this study [2]. The infinite elements have nodes only along the interface between the near and the far filed regions (Γ_i) as shown in Figure 2.

Based on the rigid-exterior boundary method [4], earthquake responses can be computed by solving the following wave radiation equation

$$\begin{bmatrix} \mathbf{S}_{nn}(\omega) & \mathbf{S}_{ni}(\omega) \\ \mathbf{S}_{in}(\omega) & \mathbf{S}_{ii}(\omega)+\widetilde{\mathbf{S}}_{ii}(\omega) \end{bmatrix} \begin{Bmatrix} \mathbf{u}_n(\omega) \\ \mathbf{u}_i(\omega) \end{Bmatrix} = \begin{Bmatrix} \mathbf{0} \\ \mathbf{f}_i(\omega) \end{Bmatrix} \tag{1}$$

in which $\mathbf{u}(\omega)$ is the total displacement vector with respect to a fix point in space; $\mathbf{S}(\omega)$ is the dynamic stiffness matrix obtained by the finite element formulation for the near field; and $\widetilde{\mathbf{S}}(\omega)$ is the dynamic stiffness matrix computed by the infinite element formulation for the far field region; $\mathbf{f}_i(\omega)$ is the equivalent earthquake force along the interface (Γ_i) as shown in Figure 2, which can be calculated from the free-field responses as

$$\mathbf{p}_i^f(\omega) = \widetilde{\mathbf{S}}_{ii}(\omega)\mathbf{u}_i^f(\omega) - \mathbf{A}\mathbf{t}_i^f(\omega) \tag{2}$$

where $\mathbf{u}_i^f$ and $\mathbf{t}_i^f$ are the displacement and the traction on Γ_i obtained from the nonlinear free-field analysis which will be discussed in the next section, and $\mathbf{A}$ is a constant transformation matrix. The dynamic stiffness matrix of the far-field, $\widetilde{\mathbf{S}}_{ii}(\omega)$ can be computed by assembling the element matrices of the infinite elements as

$$\widetilde{\mathbf{S}}^{(e)}(\omega) = (1 + j2h^{(e)})\widetilde{\mathbf{K}}^{(e)}(\omega) - \omega^2\widetilde{\mathbf{M}}^{(e)}(\omega) \tag{3}$$

where $j = \sqrt{-1}$; $h^{(e)}$ is the hysteretic damping ratio of infinite element (e); and $\widetilde{\mathbf{K}}^{(e)}(\omega)$ and $\widetilde{\mathbf{M}}^{(e)}(\omega)$ are the element stiffness and mass matrices [5].

Table 2. Properties of soil-structure system for the unified and updated models

Soil and Structure Region	Stiffness Property*			Poisson's Ratios	Mass Density (kg/m^3)	Damping Ratio (h)
	Unified Model	FVT-Correlated [2]	This Study			
Backfill-1	400	270	283	0.38	2330	0.02
Backfill-2	400	325	319	0.48	2390	0.02
Sand-1	133	133	129	0.38	1690	0.02
Sand-2	231	231	225	0.48	1930	0.02
Gravel-1	383	308	305	0.48	2420	0.02
Gravel-2	333	281	275	0.47	2420	0.02
Gravel-3	476	388	400	0.47	2420	0.02
(Shell-1)		19.7	19.9			
Shell (Shell-2)	28.2	21.3	20.7	0.167	2570	0.02
(Shell-3)		21.8	21.2			
Roof & Base Slabs	28.2	28.2	28.2	0.167	2570	0.02

* Shear wave velocities for soil medium are in *m/sec*, and Young's moduli for structure are in *GPa*.

4. Preliminary Investigation

Preliminary investigations for the sake of domain identification related to a finite element analysis are made on the soil properties obtained from the geological exploration, a static stress analysis results for the subsoil layers during various construction stages, and the predicted earthquake responses using the unified soil model. The results of the preliminary investigations are summarized as the following :

(1) Comparisons of the predicted earthquake responses using the unified model with the measured data indicate that the stiffness properties of the unified soil model are generally overly stiff [2]. Therefore, the upper bounds of the shear moduli of the soil media are taken as those of the unified model, while the lower bounds are determined from the geological test data as shown in Figure 3.

(2) From a static analysis, it is found that the shear moduli of Gravel-1 and Sand-2 shall be greater than those of Gravel-2 and Sand-1, respectively. The shear modulus of Gravel 3 region is taken to be greater than that of Gravel 1 based on the free field soil profile in Figure 3.

(3) The backfill soil region is divided into two layers with different properties, namely, Backfill-1 and Backfill-2, since the region near the ground surface is physically separated by ground water table and has a critical influence upon the response of structure. Based on the results of the static analysis, the shear modulus of Backfill-2 is taken to be greater than that of Backfill-1.

(4) It was speculated that the stiffness of the shell section of the structure, particularly in the upper part, may be weakened due to the temporary openings in the shell during the construction period [6]. Based on the above observation, the spatial variation of the shell stiffness along the height is represented by introducing three uniform regions as in Figure 2, while the stiffness of the roof slab and the foundation is taken as the value of the unified model.

Consequently, ten regions are selected for identification of the stiffness parameters, which include Shell-1, Shell-2, Shell-3, Sand-1, Sand-2, Backfill-1, Backfill-2, Gravel-1, Gravel-2, and Gravel-3 as shown in Figure 2. The lower and upper bounds of the stiffness properties are summarized in Table 2.

Table 2 Upper and lower bounds

Partitioned Regions	Lower Bounds*	Upper Bounds*
Backfill-1 Backfill-2	190	400
Sand-1	100	133
Sand-2	180	231
Gravel-1	280	383
Gravel-2	255	333
Gravel-4	333	476
Shell Structure	18.3	28.2

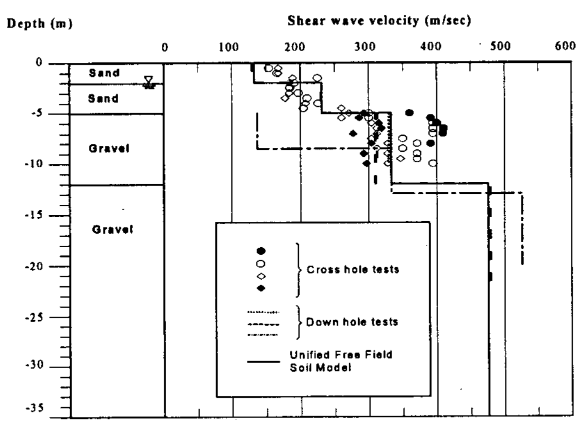


Figure 3 Soil profile of the Hualien test site [3]

5. Formulation of Parameter Identification

The shear and Young's moduli of the ten regions are updated to correlate the FE models with the earthquake response data. The parameters are represented by the base values $\overline{G}_r$, which are taken as those of the unified model, and the correction parameters p_r to be identified as:

$$G_r = \overline{G}_r(1+p_r) \quad \text{and} \quad |\,p_r\,| < 1 \qquad (r = 1,2,\cdots,N_p) \tag{4}$$

where N_p is the number of parameters, i.e. $N_p = 10$ in this study. Thus we can express the estimated response $\hat{\mathbf{u}}(\mathbf{p},\omega_s)$ of the soil-structure system under the earthquake excitation with a circular frequency of ω_s in terms of the stiffness parameters $\mathbf{p}$ as:

$$\hat{\mathbf{u}}(\mathbf{p},\omega_s) = [\mathbf{S}(\mathbf{p},\omega_s)]^{-1}\mathbf{f}(\omega_s) \tag{5}$$

$$\mathbf{S}(\mathbf{p},\omega_s) = \hat{\mathbf{K}}(\mathbf{p},\omega_s) - \omega^2\mathbf{M}(\omega_s) \tag{6}$$

in which $\mathbf{M}(\omega_s)$ is the mass matrix, and $\hat{\mathbf{K}}(\mathbf{p},\omega_s)$ is the complex stiffness matrix including the hysteretic damping effect represented as

$$\hat{\mathbf{K}}(\mathbf{p},\omega_s) = \overline{\mathbf{K}}(\omega_s) + \sum_{r=1}^{N_p} p_r \overline{\mathbf{K}}_r(\omega_s) \tag{7}$$

where $\overline{\mathbf{K}}$ is the stiffness matrix for the base case when p_r are zero, and $\overline{\mathbf{K}}_r$ denotes the stiffness matrix for the r-th region with the base value of $\overline{G}_r$. To identify the stiffness parameters minimizing the estimation error of the mathematical model in average sense, a constrained optimization problem is defined in this study as:

$$\min_{\mathbf{p}} J(\mathbf{p}) = \sum_{s=1}^{N_{fq}} \left(\mathbf{y}(\omega_s) - \hat{\mathbf{y}}(\mathbf{p},\omega_s)\right)^T \mathbf{W}\left(\mathbf{y}^*(\omega_s) - \hat{\mathbf{y}}^*(\mathbf{p},\omega_s)\right) \tag{8}$$

subject to the inequality constraints:

$$G_{Backfill-1} \le G_{Backfill-2},\ G_{Gravel-2} \le G_{Gravel-1},\ G_{Gravel-1} \le G_{Gravel-4},\ G_{Sand-1} \le G_{Sand-2} \tag{9a,b,c,d}$$

and bounds on the parameters:

$$G_{rl} \le G_r \le G_{ru} \qquad (r = 1,2,\cdots,N_p) \tag{10}$$

where G_{rl} and G_{ru} are the lower and upper bounds for G_r respectively as shown in Table 3. In Equation (8), $\mathbf{y}(\omega_s)$ is the measured frequency response; $\hat{\mathbf{y}}(\mathbf{p},\omega_s)$ is the estimated response at measurement point with a frequency of ω_s; superscript * denotes the complex conjugate; N_{fq} is the number of sampling frequency used for the calculation of the error function; and $\mathbf{W}$ is a diagonal matrix containing weight factors. In this study, the weighting factors are taken as the inverses of the square of the response amplitudes at the resonant frequency so that all response components may be equally weighted.

The present optimization problem with the inequality constraints can be solved using the constrained steepest descent method [2].

6. Results of Identification

Identification of the stiffness parameters is carried out for NS-horizontal directions using the earthquake records measured in the structure and free-field soil. The earthquake records measured on March 5, 1996 at Hualien LSST site are used for identification of the stiffness parameters. Because the maximum ground peak acceleration of the earthquake is only 0.01 g, influence of the nonlinear behavior of the soil-structure interaction system may be neglected.

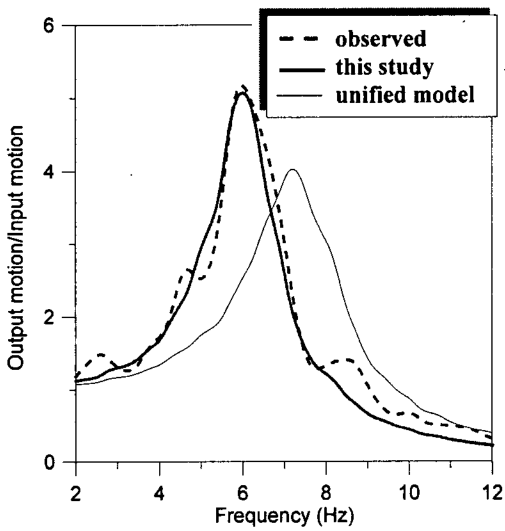


Figure 4 The error function for the identification

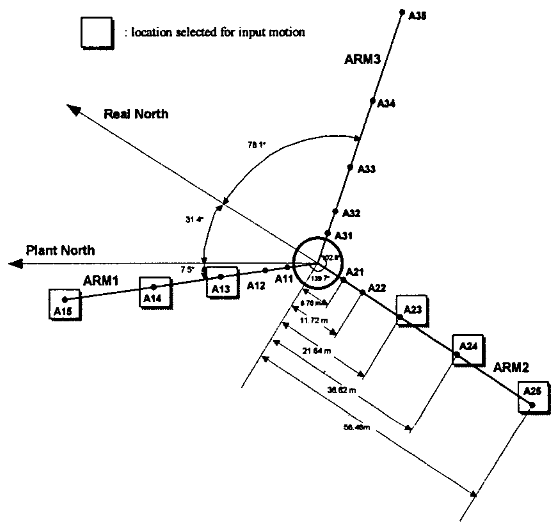


Figure 5 Sensor locations on ground surface

The error function for the parameter estimation is constructed using the complex horizontal frequency response function obtained at the top of the structure. In this study, smoothing technique of spectra by Parzen's spectral window is employed in order to carry out smoothing of the complex horizontal frequency response function. The complex horizontal frequency response function is calculated using the linear relationship between the input motion (horizontal response data measured at the free field) and the output motion (horizontal response data measured at the roof of structure) as shown in Figure 4. The input motion is artificially generated through an averaging process of the Fourier amplitude spectra of the ground accelerations measured at six locations of Figure 5, since the records do not give consistent response spectra [6]. The acceleration time histories of the input motion and the output motion are shown in Figure 6. Nine frequency points are selected for the identification: i.e., 4.0, 4.9, 5.5, 5.8, 6.0, 6.2, 6.6, 7.2, and 8.0

The properties of the unified model are used as the initial values for the identification. The constrained steepest descent method is employed to obtain the revised parameters. The estimates after the 15th iteration are shown in Table 2 along with those of the unified model and FVT-correlated model. Figure 7 shows the transition of the convergence procedures of the parameters during the iteration procedure. The identified values are generally smaller than those of the unified model and very similar to the FVT-correlated model. The responses computed from the results of earthquake analysis using the identified parameters agree very well with those from the observed responses, in contrast to those obtained using the unified model which are fairly off as shown in Figure 8 and 9. The results indicate that the identified parameters are very reasonable, suggesting that parameters can be accurately estimated with the earthquake response data only without using FVT.

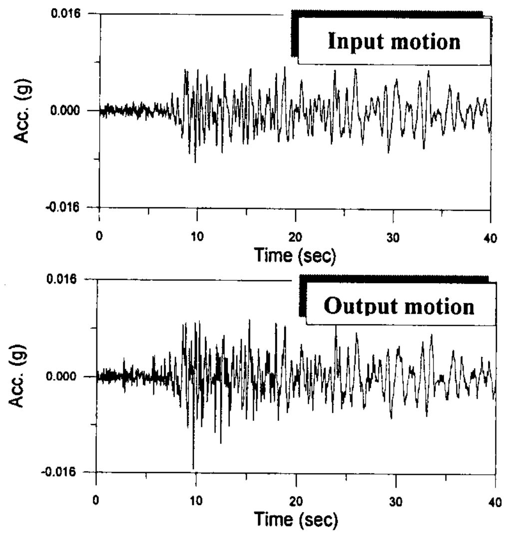


Figure 6 Time histories for NS-direction

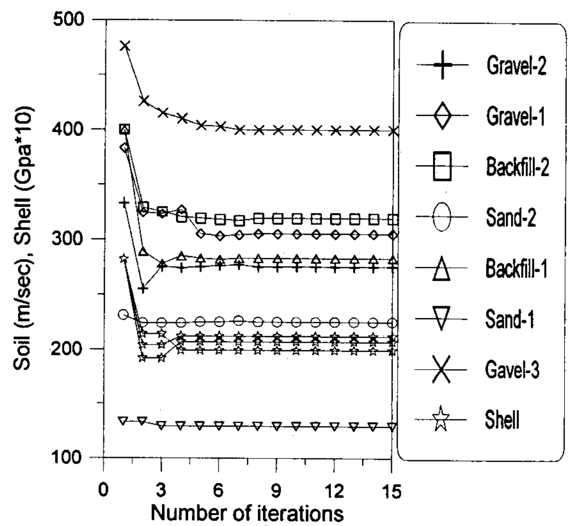
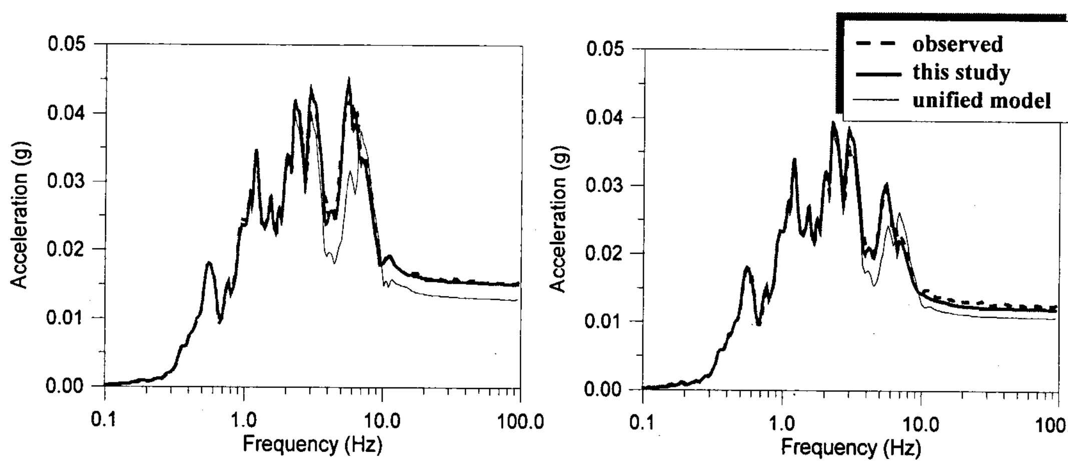


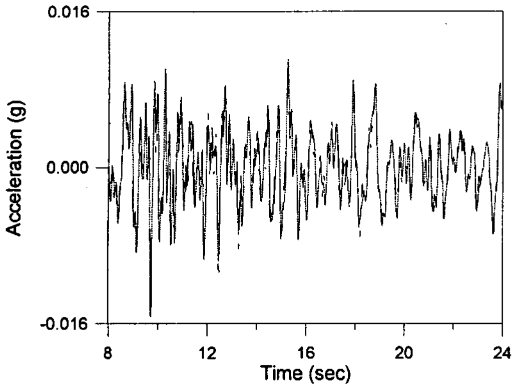
Figure 7 Convergence procedures of the parameters



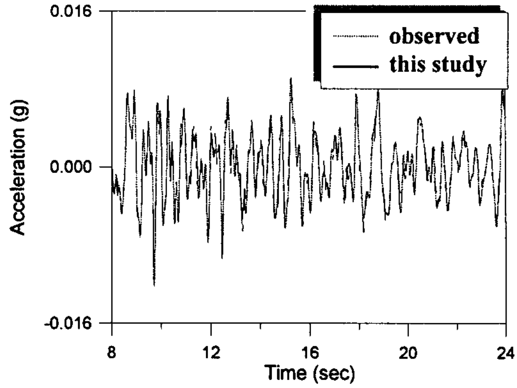
(a) Roof

(b) Wall

Figure 8 Response spectra obtained for NS-direction at Hualien LSST structure (5% Damping)



(a) Roof



(b) Wall

Figure 9 Time history obtained for NS-direction at Hualien LSST structure

7. Conclusions

This study presents a systematic way to identify a finite element model for a SSI system using the earthquake response data. The simulated earthquake responses using the identified parameters for the Hualien soil-structure interaction system are shown to be in excellent agreement with the observed response data. It is expected that the proposed method can be effectively used in the identification of soil-structure interaction systems for which performing FVT is very difficult, if not possible.

Acknowledgement

The authors would like to express their sincere appreciation for the financial supports from the Korea Earthquake Engineering Research Center (KEERC) and the Korea Science and Engineering Foundation (KOSEF) to this study.

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