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EARTHQUAKE BLIND PREDICTION ANALYSIS USING RING EXCITING THIN LAYER METHOD IN THE INTERNATIONAL PROJECT OF HUALIEN LARGE SCALE SEISMIC TEST

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ABSTRACT

The ring exciting thin layer method, proposed by Tajimi, using the Green's functions for ring loads is outlined. Since the present method is characterized by a multiple cylindrical walls model, irregular profiles of soils such as backfills can be realistically taken into account. Employing the ring exciting thin layer method, the blind analysis was conducted to predict the seismic behaviors of the soil-structure interaction system during the earthquake of January 20,1994 in the Hualien Large Scale Seismic Test program.

KEYWORDS

soil-structure interaction ; earthquake response analysis ; blind prediction analysis ;
ring exciting thin layer method ; Green's function,multiple cylindrical wall ;
Hualien Large Scale Seismic Test program ; nuclear power plants.

INTRODUCTION

The seismic test program of a quarter-scale reinforced concrete containment model constructed in Hualien, a seismically active site in Taiwan, has been conducted for the international project (U.S., Taiwan, Korea, France and Japan) to study the actual seismic behaviors of the soil-structure system, as well as, to verify the SSI analyses methods employed for the seismic response predictions of the nuclear power plants(Tang et al. 1991). Participating in this Hualien LSST project, we performed the earthquake blind prediction analysis using the ring exciting thin layer method proposed by the author (Tajimi 1994,1995) to deal realistically with the irregular profiles of backfills. The scope of the present paper is to outline the ring exciting thin layer method and to describe its first application to the practical seismic response analysis.

RING EXCITING THIN LAYER METHOD

General

The ring exciting thin layer method(Tajimi 1994,1995) was developed to conduct the dynamic analysis of a soil-structure system with a circular foundation resting on the surface or embedded in the soils under the irregular soil profiles. Fig.1 illustrates a typical model of the soil-structure interaction system introduced for the ring exciting thin-layer method, where an axisymmetric 3-dimensional geometry is assumed. This technique, extended from the point exciting thin layer one(Tajimi 1980), is based on the Green's function approach employing the ring load solutions to describe the soil–foundation interaction. As introduced in the point exciting thin layer method, the exact displacement functions in the horizontal direction and a linear variation of the displacement in the vertical direction are defined in the thin layered strata. However, the point exciting thin layer technique is limited to the horizontally layered soil models, whereas the present ring exciting thin layer method allows the irregular profiles of the soils by employing the multiple cylindrical walls model shown in Fig.1.

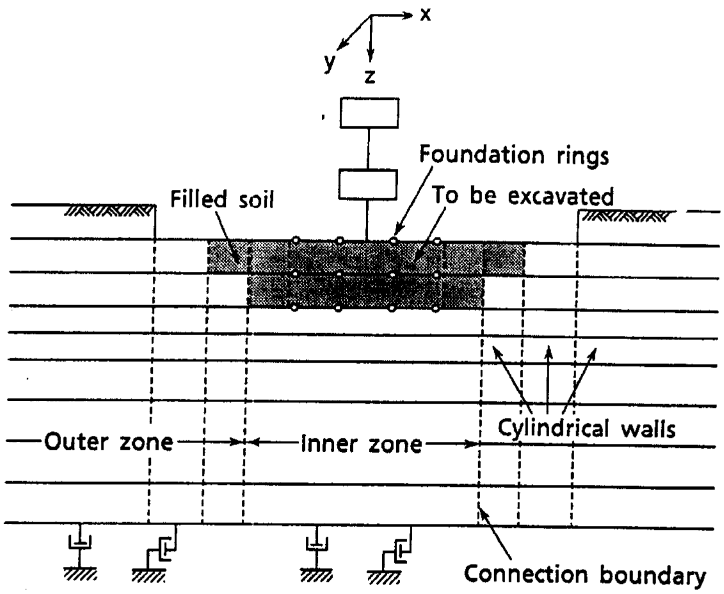


Fig.1 A typical model idealized for ring exciting thin layer method

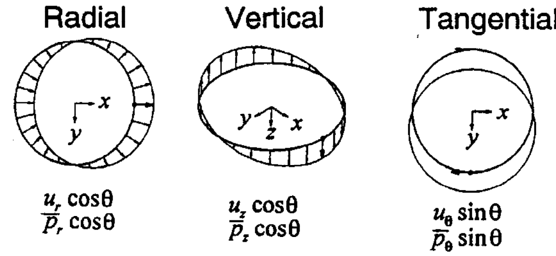


Fig.2 Distribution of ring loads and ring displacements for three excitation modes (Sway and rocking)

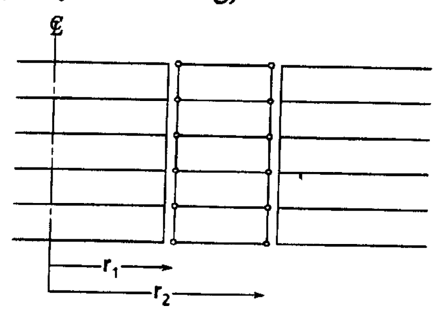


Fig.3 Section of a cylindrical wall

Green's functions for ring loads

The first step of the analysis procedure is to introduce the Green's functions for dynamic ring loads in the unbounded thin layer soil model(Basic model). Although Kausel(1981) described the Green's functions for ring loads uniformly distributed in the x–direction, it was necessary for the present method to formulate the solutions for two kinds of horizontal ring loads applied independently in the radial and in the tangential directions. Shown in Fig.2, the distributions of the ring loads and the displacements under the sway and rocking motion were defined in the present method for radial, vertical and tangential modes. Through the Hankel transformation of the ring loads and the quadratic eigenvalue analysis of the thin layer model, the ring displacements arising at the i–th interface due to the ring loads at the j–th interface were finally formed (Tajimi 1994);

$$\{u_i\} = \frac{1}{\pi}\Big(\sum_k \frac{2\alpha_k^{\,2}}{D_{\alpha k}}[\xi_{ki}][\psi_{\alpha k}][\xi_{kj}] + \sum_k \frac{Y_{ki}Y_{kj}}{D_{\beta k}}[\psi_{\beta k}]\Big)\{p_i\} \tag{1}$$

,where $\{u_i\} = (u_{ri}, u_{zi}, u_{\theta i})^T$ $\{p_i\} = (p_{ri}, p_{zi}, p_{\theta i})^T$ $[\xi_{ki}] = diag(X_{ki}, Z_{ki}, X_{ki})$ $[\xi_{kj}] = diag(X_{kj}, Z_{kj}, X_{kj})$

,and $D_{\alpha k} = \alpha_k^{\,2}\{X\}_k^T[A_p]\{X\}_k + \alpha_k^{\,2}\{Z_k\}^T[A_s]\{Z_k\} - \{X\}_k^T[E_s]\{X\}_k - \{Z\}_k^T[E_p]\{Z\}_k$

In the above equations, $\alpha_k, \{X\}_k, \{Z\}_k$ denote the eigenvalue (wavenumber) and the corresponding eigenvectors of free harmonic motions of the generalized Layleigh's waves of the thin layered system, and $\beta_k, \{Y\}_k$ denote those of the Love's waves. ($\mathrm{Im}(\alpha_k) < 0, \mathrm{Im}(\beta_k) < 0$) Matrices $[A_p],[A_s],[E_p],[E_p]$ of order NxN which appear in the eigenvalue equations of the Layleigh's and Love's waves depend only on the geometry, the material properties of the discrete layers of which total number is N. Matrices $[\psi_{\alpha k}],[\psi_{\beta k}]$ of order 3x3 are formed by the infinite integrals.

Stiffness matrix of multiple cylindrical walls model

A cylindrical wall has the inner and outer interfaces of radii of r_1 and r_2, respectively, shown in Fig.3. Since, in the cylindrical wall, the waves propagate in both the positive and negative radial directions, not only the eigenvalues(wavenemvers) of α_k, β_k but also those of $-\alpha_k$, $-\beta_k$ must be considered.

Expressing the nodal displacements vector {U} at the boundary interfaces of the cylindrical wall in terms of the modal shapes and the participation factors, it is possible to write ;

$$\{U\} = [V]\{q\} \tag{2}$$

, where [V] is a transformation matrix of order 6Nx6N formed by modal matrices [X],[Y],[Z] and diagonal matrices in terms of Hankel functions of arguments $\alpha_k r_1, \alpha_k r_2, \beta_k r_1, \beta_k r_2, -\alpha_k r_1, -\alpha_k r_2, -\beta_k r_1, -\beta_k r_2$, and {q} is a modal participation factors vector. Use of the principle of virtual work in connection with the vertical interfaces of the cylindrical wall yields ;

$$\{P\} = [D]\{q\} \tag{3}$$

, where {p} is a nodal force vector, and a matrix [D] of order 6Nx6N is formed by the integrals with substitution of the above general solution Eq. (2). The stiffness matrix of the i-th cylindrical wall $[R]_i$ is derived from Eqs (2) and (3) ;

$$[R]_i = [D]_i [V]_i^{-1} \tag{4}$$

For the case that the outer radius r_2 approaches infinity, the stiffness matrix of the cylindrical wall corresponds to that of the transmitting boundary.

Impedance function analysis

For a basic model (laterally unbounded model), the relation of the loads and displacements at the surface foundation rings can be expressed from Eq.(1) [Step 1 in Fig.4] ;

$$\{U\} = [A]\{p\} \tag{5}$$

Consider the inner core zone with adding the set of the rings on the layer interfaces at the connection boundary (connection boundary rings) , the following relation is formed ;

$$\begin{Bmatrix} \{U\}_f \\ \{U\}_c \end{Bmatrix} = \begin{bmatrix} [A_{11}] & [A_{12}] \\ [A_{21}] & [A_{22}] \end{bmatrix} \begin{Bmatrix} \{P\}_f \\ \{P\}_c \end{Bmatrix} \tag{6}$$

, where the subscript "f" and "c" denote the foundation rings and the connection boundary rings, respectively. The constraint force vector $\{P\}_c$ to confine the inner core zone on the connection boundary is given by putting $\{U\}_c = 0$ in Eq.(6) [Step 2 in Fig.4] ;

$$\{P\}_c = -[A_{22}]^{-1}[A_{21}]\{P\}_f \tag{7}$$

The stiffness matrix of the multiple cylindrical walls model is assembled by addition of submatrix $[R]_i$ in Eq.(4) and it can be rewritten;

$$\begin{Bmatrix} \{P\}_c \\ \{P\}_d \end{Bmatrix} = \begin{bmatrix} [R_{22}] & [R_{23}] \\ [R_{32}] & [R_{33}] \end{bmatrix} \begin{Bmatrix} \{U\}_c \\ \{U\}_d \end{Bmatrix} \tag{8}$$

,where subscript "d" denote the rings in the outer zone except on the connection boundary between the inner core zone and the outer zone. Putting $\{P\}_d = 0$ in Eq.(8) , the force–displacement relation of the connection boundary rings is given ;

$$\{P\}_c = [R_0]\{U\}_c \qquad [R_0] = [R_{22}] - [R_{23}][R_{33}]^{-1}[R_{32}] \tag{9}$$

Release the constraint force $\{P\}_c$ defined by Eq.(7) in the multiple cylindrical walls model [Step 3 in Fig.4], the displacements vector $\{U\}_f$ at the foundation rings can be formulated ;

$$\{U\}_f = ([A_{11}] - [A_{12}][A_{22}]^{-1}[A_{21}] - [B][R_0]^{-1}[A_{22}]^{-1}[A_{21}])\{P\}_f \tag{10}$$

, and

$$\{P\}_f = [K]_f \{U\}_f$$

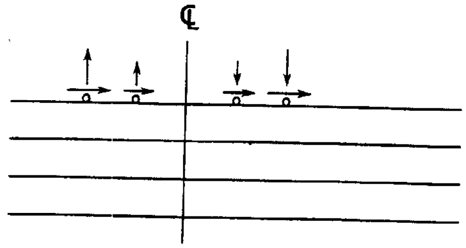
,where [B] is a transfer functions matrix from $\{u\}_c$ to $\{u\}_f$, and $[K]_f$ is a stiffness matrix of the foundation rings of the cylindrical walls model. For an embedded rigid foundation, the relationship between the displacement vector of the foundation rings $\{U\}_f$ and the displacements of the foundation (horizontal displacement U_h , rocking angle θ) can be written ;

$$\{U\}_f = [E]\begin{Bmatrix} U_h \\ \theta \end{Bmatrix} \tag{11}$$

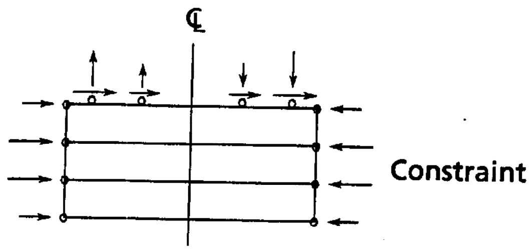
The stiffness matrix (impedance function matrix) for the coupled motion of sway and rocking is finally formulated ;

$$[K] = [E]^T [K]_f [E] \tag{12}$$

Step 1 : Ring excitation on flat model.



Step 2 : Fixed at the connection boundary.



Step 3 : Contraint force released in the multiple cylindrical wall model.

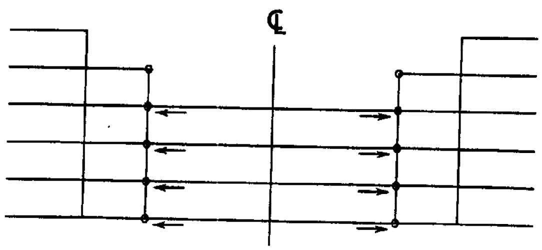


Fig.4 Analysis steps to compose stiffness matrix of $[k]_f$ of foundation rings

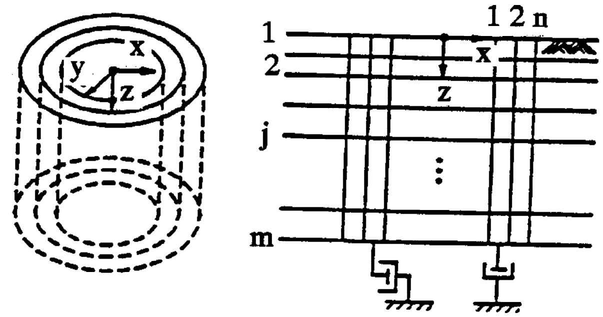


Fig.5 Structure of multiple cylindrical walls model

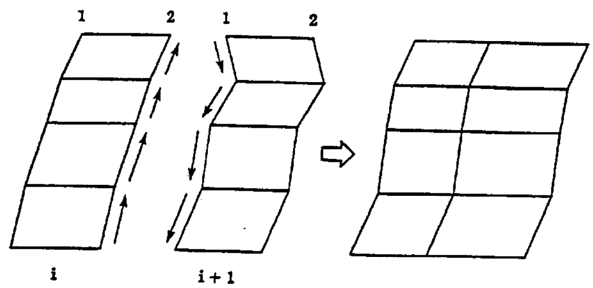


Fig.6 Connection of i–th and (i+1)th cylindrical walls

<u>Response of soils to earthquake motions (seismic excitation)</u>

A typical multiple cylindrical walls model of soils is illustrated in Fig.5. The displacement vector of the rings $\{U\}_i$ at the interfaces of the i–th cylindrical wall subjected to the harmonic base motions and, at the same time, to the external ring loads $\{P\}_i$ acting at the interfaces is expressed as ;

$$\{P\}_i = [R]_i(\{U\}_i - \{U\}_i^x) \tag{13}$$

, where $[R]_i$ is a stiffness matrix given in Eq.(4) and $\{U\}_i^x$ denotes the response of the cylindrical wall to the harmonic base motion in the x–direction. In fact, $\{U\}_i^x$ corresponds to the dynamic response of the one–dimensional shear soil column assumed using the same soil parameters as the cylindrical wall. The internal force vector induced to satisfy the continuous condition of the neighboring cylindrical walls, shown in Fig.6, is ;

$$[R]_i\{U\}_i^x = \{Q\}_i \tag{14}$$

By substituting Eq.(14) into Eq.(13) and by assembling the sub–equation of Eq.(13) , the following equation can be formed ;

$$\{P\} = [R]\{U\} - \{Q\} \tag{15}$$

For the case the external ring loads $\{P\}=0$, i.e., when the soil model is subjected only to the base motions, the equilibrium relation is ;

$$[R]\{U\} = \{Q\} \tag{16}$$

By solving Eq.(16), the response (motions of the rings) of the soil model $\{U\}_g$ to the seismic motions can be obtained.

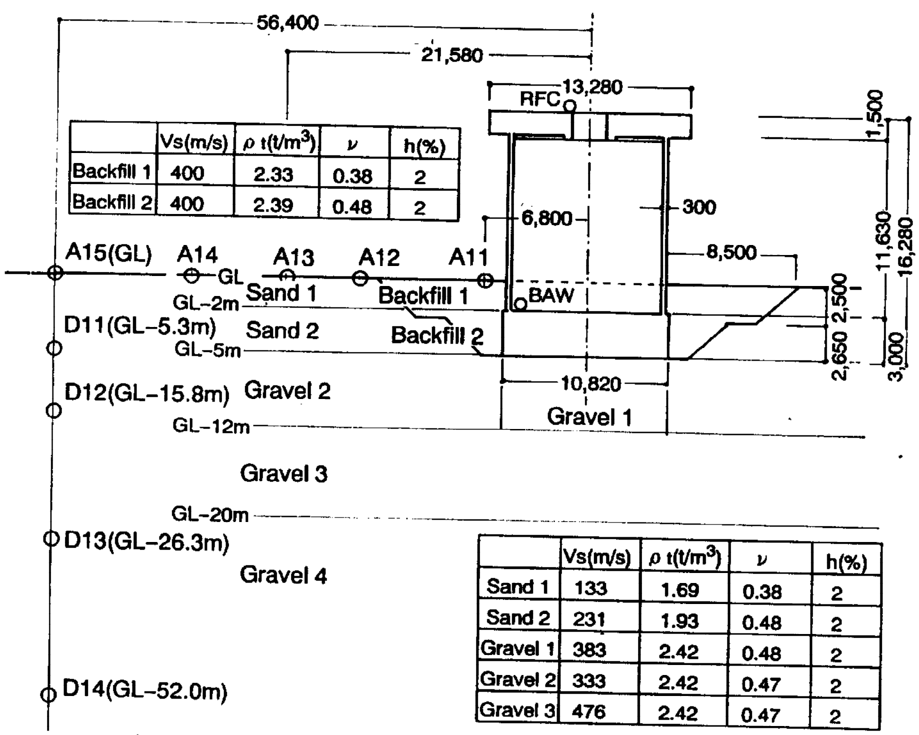
SSI analysis of soil–structure system

By employing the volume method, the SSI analysis can be conducted in the straightforward manner. The procedure is outlined as follows ;

1) To compute the response of the multiple cylindrical walls model $\{u\}_g$ in Eq.(16).
2) To compute the effective input motions by use of the impedance matrix of foundation given by Eq.12) with the mass matrix of soil volume replaced by foundation for sway and rocking motions.
3) To compute the response of the structure. In the present method, the structure is idealized by a stick model of lumped masses system.

HUALIEN LARGE SCALE SEISMIC TEST

The international project of the Hualien Large Scale Seismic Test Program has been conducted to investigate the soil–structure interaction effects during forced vibrations and earthquakes. (The project started in 1990.) The forced vibration test had already been finished both before backfill and after backfill (Morishita et.al 1993, Sugawara et.al 1995). After these tests, the blind prediction and correlation analyses of the forced vibration were successfully performed by the consortium members (Kishi et al.1995). Since April in 1993, the earthquakes have been observed to collect the data at the Hualien site. Fig.7 shows the model structure, the soil conditions, and the instrumental locations of the accelerometers including downhole array utilized in the blind analysis. At the present stage of the project, the blind prediction analyses using the earthquake data recorded on January 20, 1994 (M=5.6, Epicentral distance=27km) was performed by the consortium members.



	Vs(m/s)	ρ t(t/m³)	ν	h(%)
Backfill 1	400	2.33	0.38	2
Backfill 2	400	2.39	0.48	2

	Vs(m/s)	ρ t(t/m³)	ν	h(%)
Sand 1	133	1.69	0.38	2
Sand 2	231	1.93	0.48	2
Gravel 1	383	2.42	0.48	2
Gravel 2	333	2.42	0.47	2
Gravel 3	476	2.42	0.47	2

Fig.7 The model structure and the unified soil conditions with the instrumental locations at the Hualien site

EARTHQUAKE BLIND PREDICTION ANALYSIS

Control motion and analysis procedure

The free field records at the ground surface (A15 in Fig.7) were given as the control motions to predict the motion along the downhole array, as well as, the responses of the structure and its surrounding soils. Although three components (L,T,V) of the record at A15 were given, only L–component (NS–component) of them was examined in the present paper, shown in Fig.8. Illustrated in Fig.9, at the first step of the analysis, the seismic response analysis of the free field soil system has been performed by employing the one-dimensionally propagating shear wave theory (SHAKE-equivalent) to compute the motions along the downhole, and to simulate the input motions at the base of the SSI model. At the second step, the response analysis of the soil-structure system has been carried out by employing the ring exciting thin layer method. Since the maximum acceleration of the control motion ranged about 30-40gals, the linear dynamic analyses were performed without accounting for the strain dependency of the soil properties.

SSI analysis model

The structure was idealized by a stick model with the two masses interconnected by a flexural and a shear beam elements, described in Fig.10. At the same time, the soils including the embedded part of the structure were discretized into the thin layer elements with the multiple cylindrical walls for an axisymmetric geometry, shown in Fig.11. In the present blind analysis, we used the unified soil model (Okamoto et.al 1995), as well as, our modified soil model (Kishi et.al 1995) obtained through the post-test correlation analyses of the forced vibration tests.

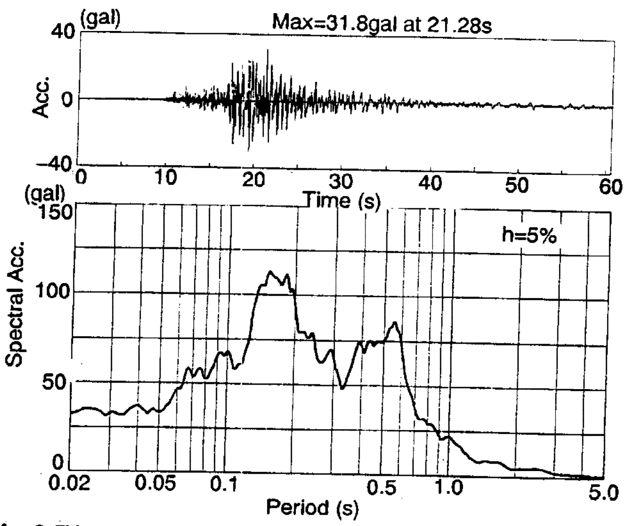


Fig.8 The control motions at A15 given for the blind analysis (L-component)

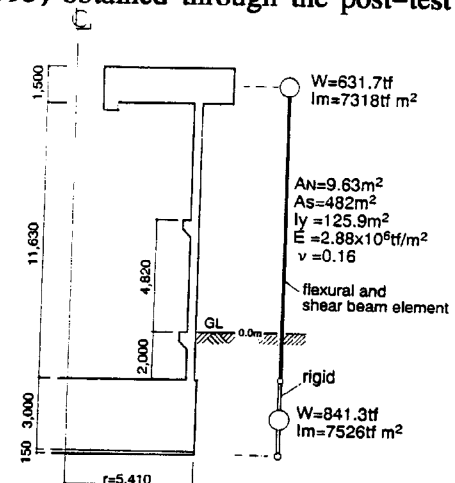


Fig.10 The structure and its idealization

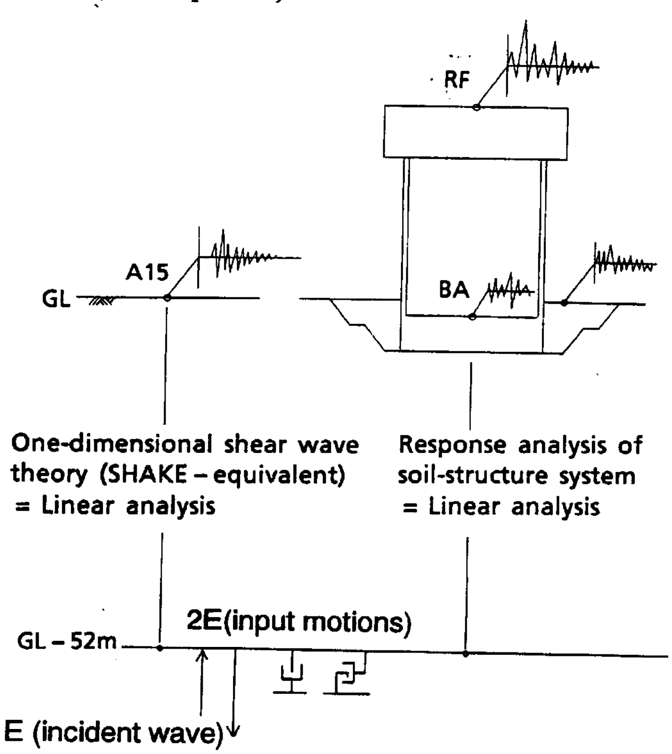


Fig.9 Analysis procedure for earthquake blind prediction

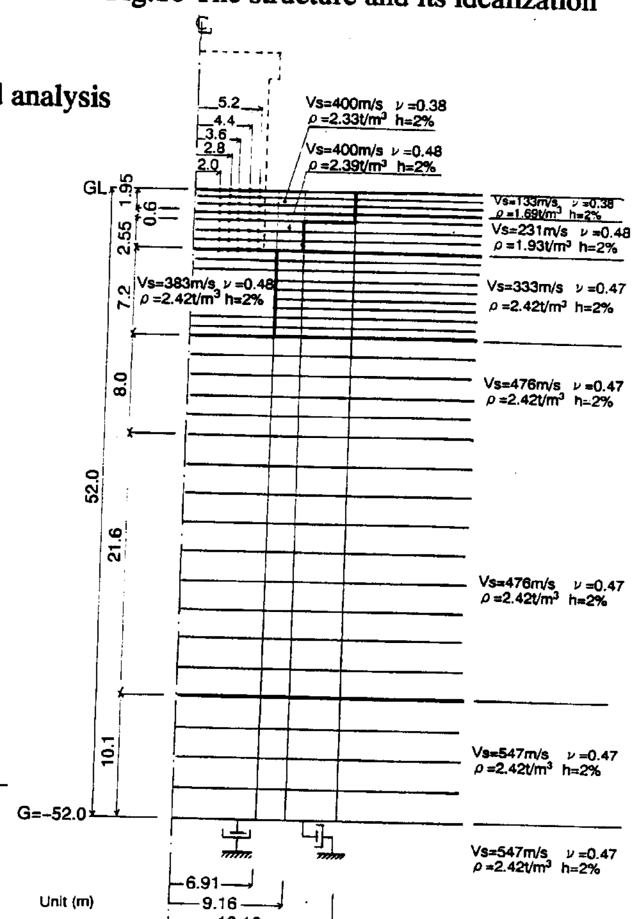


Fig.11 Mathematical model of the soils for the ring exciting thin layer method under the unified soil conditions

Results of the blind prediction analysis

As for the responses of the free field soil system, a good comparison of the predicted and the observed motions along the downhole array demonstrated that the validity of the SHAKE-equivalent technique utilizing the free field soil properties provided in the present study. As analysis results of the SSI system, Figs. 12 and 13 show the predicted maximum acceleration of the system, and the predicted floor response spectra at roof and at 1st floor, respectively, compared with observation. Although the predicted maximum accelerations compared well with the observation, a significant discrepancy of the predominant periods appears between the predicted and the observed spectra at roof. While the predominant frequencies of the spectra at roof during the earthquake were predicted to be 7.1Hz and 6.7Hz for the unified and for the modified soil models, respectively, the observed one was 5.3Hz. Since the resonant frequencies obtained during the forced vibration tests (Sugawara et.al 1995) were 6.1–6.3Hz and 6.5–6.6Hz for the horizontal excitations at roof and at 1st floor, respectively, such significant discrepancy was considered to be caused mainly by the non–linear characteristics of the properites of the soils adjacent to the foundation due to its motions during the earthquake (This effect is called "secondary non–linearity ".)

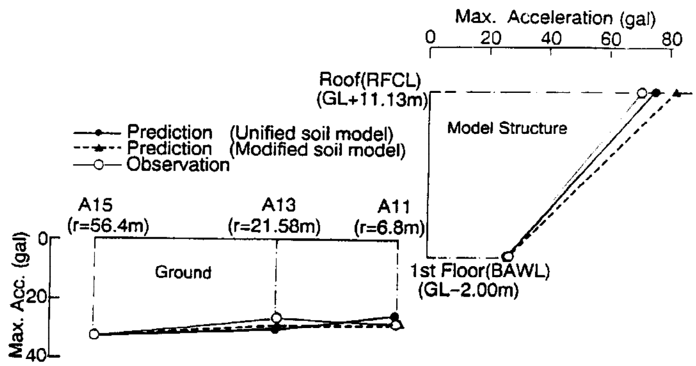
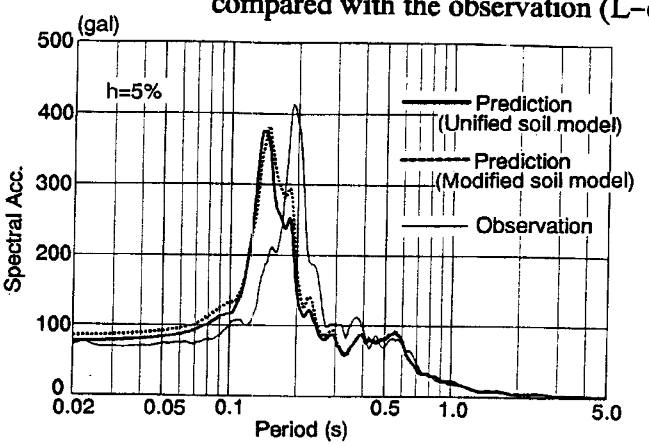
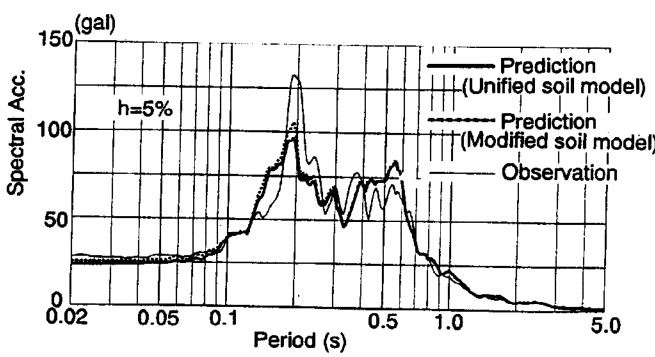


Fig.12 The predicted maximum accelerations of the structure and its surrounding soils, compared with the observation (L–component)



(a) FRS at roof



(b) FRS at 1st floor

Fig.13 The predicted floor response spectra at roof and at 1st floor, compared with the observation (L–component)

Correlation study

After comparing the predictions and the observations, the correlation analysis has been conducted by modifying both the soil and the structure models. It was available in the present project that the additional geological investigation of the site soils focussing on the survey of the heterogeneity in the backfill and the foundation gravel zones was carried out. On the basis of this additional investigation, the modified soil model was newly proposed for the correlation, shown in Fig.14. The correlation analysis has been performed by taking into account the soil model shown in Fig.14 and by reducing the structural stiffness (multiplied by 0.85) estimated by Sugawara et.al(1995). It should be emphasized that the analysis of the forced vibration test at roof floor by using this newly modified model yielded the resonant frequency of 6.2Hz, which was well correlated with the measurement of 6.1–6.3Hz. Furthermore, the soil model was modified by taking into account the non–linear effects of the soils around the foundation. On the basis of the G/G0– γ and the h– γ curves obtained in the laboratory cyclic triaxial tests, the stiffness of the soils around the foundation was reduced by 19%, and at the same time, their damping increased with value of 2% and 4% in the foundation gravel zone and the backfill, respectively. Fig.15 shows the response spectra at roof computed by the correlation analyses, compared with the observation. It can be recognized in Fig.15 that the overall correlation was good. The predominant frequency of the spectra at roof during the earthquake was evaluated to be 5.4Hz by the present correlation analysis.

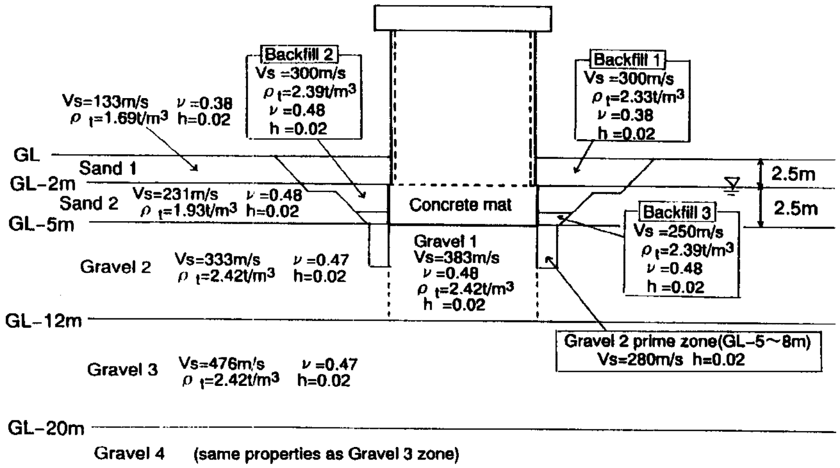


Fig.14 The soil model modified after the additional geological investigation (CRIEPI 1995)

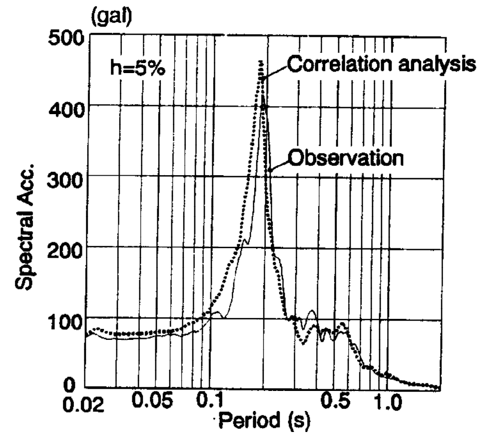


Fig.15 The correlation analysis result compared with the observation (FRS at roof , L-component)

CONCLUSIONS

The earthquake prediction analysis demonstrated that the ring exciting thin layer method outlined in the present paper has a great ability to compute seismic responses of soil-structure interaction systems. The ring exciting thin layer method using the Green's functions for ring loads is characterized by a multiple cylindrical walls model to deal realistically with irregular soil profiles for an axisymmetric geometry. As results of the prediction analysis on the earthquake of January 20,1994 , although the overall predictions showed good, there appeared a significant discrepancy between the predicted and the recorded predominant periods at roof. The correlation study was successfully conducted with paying attention to the results of the additional geological investigation, and to the non-linear characteristics of the soils due to the motions of the foundation, as well as, to the structural stiffness.

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