Nonlinear Seismic Response Analysis of Embedded Reactor Building Based on the Substructure Approach in Time Domain

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Abstract

A practical method for elasto-plastic seismic response analysis is described under considerations of nonlinear material law of a structure and dynamic soil-structure interaction. The method is essentially based on the substructure approach of time domain analysis. Verification of the present method is carried out for a typical BWR-MARKII type reactor building which is embedded in a soil, and the results are compared with those of the frequency response analysis which gives good accuracy for linear system. As a result, the present method exhibits sufficient accuracy. Furthermore, elasto-plastic analyses considering the soil-structure interaction are made as an application of the present method, and nonlinear behaviors of the structure and embedment effects are discussed.

1. Introduction

It is a well-known fact that the soil-structure interaction has a considerable influence on seismic response of a nuclear reactor building. Especially for an embedded reactor building, the problem must be strictly evaluated. It is generally considered that there are two approaches to analyze the soil-structure interaction system. One is a so-called direct method in which the structure and its surrounding soil are treated as a total system (e.g., FLUSH by Lysmer, et al. [1]). Another is a so-called substructure method which first deals the structure and the soil systems separately, then combines these results (e.g., Lysmer [2] and Luco, et al. [3]). Though the analysis procedure of the latter method is much more complicated, it is superior to the former in following aspects: 1) suitable models can be made to the respective systems, and 2) number of degrees of freedom may be substantially reduced for the numerical calculation.

Nonlinear behaviors of a reactor building subjected to a strong ground motion are significant factors on the seismic design. In general, an elasto-plastic analysis for seismic excitation is made in time domain because of time-history characteristics of nonlinear behaviors of the structure. Meanwhile, if the soil-structure interaction effects are strictly evaluated by the elasto-plastic analysis, there should be no other method but to use the direct method. However, the direct method is impractical because it requires large degrees of freedom for the soil region and a great deal of computation time. This paper presents an alternative method based on the substructure approach for the elasto-plastic analysis considering both the nonlinear material law of the structure and the soil-structure interaction.
2. Method of Analysis

The present method is formulated in accordance with the substructure method which is expanded into time domain analysis. The fundamental procedure of the method is illustrated in Fig. 1. Assuming linearity in the soil-structure interaction system and applying superposition law, the equations of motion of the structure and the soil subjected to seismic excitation can be written in the following forms, respectively;

\[
\begin{align*}
\begin{bmatrix} M_1 & C_{10} & K_{10} \\ C_{10} & M_2 & C_{20} \\ K_{10} & C_{20} & K_{20} \end{bmatrix} \begin{bmatrix} \ddot{u}_1 \\ \ddot{u}_2 \\ \ddot{u}_3 \end{bmatrix} + \begin{bmatrix} C_{10} & C_{10} & C_{10} \\ C_{10} & C_{20} & C_{20} \\ C_{10} & C_{20} & C_{20} \end{bmatrix} \begin{bmatrix} \dot{u}_1 \\ \dot{u}_2 \\ \dot{u}_3 \end{bmatrix} + \begin{bmatrix} K_{10} & K_{10} & K_{10} \\ K_{10} & K_{20} & K_{20} \\ K_{10} & K_{20} & K_{20} \end{bmatrix} \begin{bmatrix} u_1 \\ u_2 \\ u_3 \end{bmatrix} &= \begin{bmatrix} M_1 & M_1 & M_1 \\ M_1 & M_2 & M_2 \\ M_1 & M_2 & M_2 \end{bmatrix} \begin{bmatrix} \ddot{u}_b \\ \ddot{u}_b \\ \ddot{u}_b \end{bmatrix} \begin{bmatrix} 0 \\ 0 \\ -F_z \end{bmatrix} \\
\begin{bmatrix} M_2 & C_{20} & K_{20} \\ C_{20} & M_3 & C_{30} \\ K_{20} & C_{30} & K_{30} \end{bmatrix} \begin{bmatrix} \ddot{u}_2 \\ \ddot{u}_3 \\ \ddot{u}_3 \end{bmatrix} + \begin{bmatrix} C_{20} & C_{20} & C_{20} \\ C_{20} & C_{30} & C_{30} \\ C_{20} & C_{30} & C_{30} \end{bmatrix} \begin{bmatrix} \dot{u}_2 \\ \dot{u}_3 \\ \dot{u}_3 \end{bmatrix} + \begin{bmatrix} K_{20} & K_{20} & K_{20} \\ K_{20} & K_{30} & K_{30} \\ K_{20} & K_{30} & K_{30} \end{bmatrix} \begin{bmatrix} u_2 \\ u_3 \\ u_3 \end{bmatrix} &= \begin{bmatrix} M_2 & M_2 & M_2 \\ M_2 & M_3 & M_3 \\ M_2 & M_3 & M_3 \end{bmatrix} \begin{bmatrix} \ddot{u}_b \\ \ddot{u}_b \\ \ddot{u}_b \end{bmatrix} \begin{bmatrix} 0 \\ 0 \\ F_z \end{bmatrix}
\end{align*}
\]

where \([M], [C] \) and \([K] \) denote a mass, a damping and a stiffness matrices, respectively; and \([u] \) is a resulting displacement vector relative to a base motion \(u_b \). Subscripts and superscripts in these equations represent the separated subsystems as shown in Fig. 1(a). \(|F_z|\) denotes an interaction force vector produced by separation at the soil-structure interface. Here, the effective seismic input vector \(|u^s|\) of the structure system can be obtained from Eq. (2) by putting \(|F_z|=0\); and the displacement vector \(|\Delta u|\) due to the concentrated load \(F_z\) at the interface can be obtained from Eq. (2) by putting \(|u_b|=0\). Applying superposition law, the resulting displacement vector \(|u|\) is given by

\[
|u| = |u^s| + |\Delta u|
\]

Assuming a harmonic load of an arbitrary circular frequency \(\omega\) to the interaction force, a relationship between the force vector \(|F_z|\) and the displacement vector \(|\Delta u|\) at the interface can be derived similarly. They are

\[
|F_z| = [K_z]|\Delta u|
\]

\[
[K_z] = \left( -\omega^2M_s^2 + i\omega C_s + K_s \right) - (i\omega C_0 + K_0) (\omega^2M_0 + i\omega C_0 + K_0) - (i\omega C_0 + K_0) \right)
\]

where \([K_z]\) denotes an impedance matrix, whose property obviously depends on the frequency. In order to include the frequency dependency in time domain, a single degree of freedom (SDOF) system is introduced as shown in Fig. 2. A relationship between an exciting force \(P\) and the resulting displacement \(\delta\) of the system is given by

\[
P = (K_s - \omega^2M_s + i\omega C_s)\delta
\]

where \(K_s, M_s, C_s\) and \(C_D\) are a soil spring, an effective mass and a viscous coefficient of the dashpot which accounts for radiation damping of the soil, respectively. The frequency dependency of the impedance function in Eq. (5) and diagonal components of the impedance matrix in Eq. (4) are schematically illustrated in Fig. 3. By fitting these two functions at a reference frequency \(\omega_0\), which may be the natural frequency of the soil-structure system, \(K_s, C_D\) and \(M_s\) are obtained as

\[
\begin{align*}
K_s &= K_s^{\omega=0} \\
C_D &= K_s^{\omega=\omega_0}/\omega_0 \\
M_s &= (K_s - K_s^{\omega=\omega_0})/\omega_0^2
\end{align*}
\]

Approximating all components of the impedance matrix \([K_z]\) in the same manner, Eq. (4) can be converted in time domain as

\[
|F_z| = [M_s]|\Delta u| + [C_D]|\Delta u| + [K_s]|\Delta u|
\]

Substituting Eq. (7) into Eq. (1) and considering Eq. (3), the equation of motion for the
structure system is finally given by

\[
\begin{bmatrix}
M_t + M_A & C_{1A} & C_{2A} \\
C_{1A} & M_{1A} + M_2 & C_{3A} \\
C_{2A} & C_{3A} & M_{3A} + M_4
\end{bmatrix}
\begin{bmatrix}
\ddot{u}_1 \\
\ddot{u}_2 \\
\ddot{u}_3
\end{bmatrix}
+ \begin{bmatrix}
K_{1A} & K_{2A} & K_{3A} \\
K_{2A} & K_{3A} + K_s & K_{4A} \\
K_{3A} & K_{4A} & K_{5A}
\end{bmatrix}
\begin{bmatrix}
\dot{u}_1 \\
\dot{u}_2 \\
\dot{u}_3
\end{bmatrix}
= \begin{bmatrix}
M_t & I \\
M_{1A} & 0
\end{bmatrix}
\begin{bmatrix}
\ddot{u}_4 \\
\dot{F}_d
\end{bmatrix}
\]

(8)

\[
\{F_d\} = [M_A]\{\ddot{e}_A\} + [C_A]\{\dot{e}_A\} + [K_s]\{e_A\}
\]

Strictly speaking, Eq. (8) is obtained by assuming linearity of the system and subsequent superposition law. However, since time-history characteristics of nonlinear behaviors of the structure have great effects on the stiffness matrix \([K_{sL}]\) in Eq. (8), the present analysis procedure may be employed practically to the nonlinear analysis considering the soil-structure interaction.

3. Verification of The Method

Verification of the present method was carried out for a typical BWR-MARKII type reactor building which is embedded in a soil as shown in Fig. 4. Simulated earthquake motion was adopted as an input. The structure was idealized by a lumped mass system consisting of 11 masses. The structure system was described by beam elements, and the material damping factor of 5% was assumed. The mathematical model, which represents the coupling modes of rocking and swaying considering the embedment effects, is shown in Fig. 5. In order to calculate the impedance function and the effective seismic input, the soil system excavated by the structure was idealized by the two-dimensional finite element model as shown in Fig. 6. In the model, dashpots are added at the bottom boundary to account for radiation effects due to the elastic halfspace, and the energy transmitting boundary is used along the side boundary.

Accuracy of the results by the present method was examined in comparison with the frequency response analysis by the FFT method which yields good accuracy for linear system. Three different types of the impedance matrix form, which are diagonal (type 1), multi-diagonal (type 2) and full (type 3) matrices, were considered for the present method; while, the matrix form of type 3 was used for the frequency response analysis. In Fig. 7, the impedance functions of swaying and rocking modes approximated by a SDOF system are illustrated and compared with those by FEM. It is found that the approximations by the SDOF system are relatively good in a significant frequency range where seismic response is mostly influenced. In Figs. 8 and 9, the distributions of the maximum response values and the floor response spectra are illustrated for each type of the impedance matrices, and are compared with the results by the FFT method. As we see from these figures, the results by the present method coincide very well with those by the FFT method exclusive of type 1. Accordingly the impedance matrix must be appropriately evaluated by including non-diagonal components representing coupled rocking-and-swaying modes. Consequently validity of the present method was proved for the cases of the impedance matrices of type 2 or 3.

4. Application of The Method

As an application of the present method, elasto-plastic analyses were carried out for an embedded reactor building which has various depth of the embedment. In Table 1, three cases of the analyses are illustrated with the impedance matrix form evaluated in type 2. The mathematical model was similarly made as shown in Fig. 5. The nonlinear materials of the structure were idealized by skeleton curves of tri-linear type; and hysteretic curve for the relationship of shear force and shear strain was modeled by the origin-oriented type, while the relationship of bending moment and curvature was modeled by the peak-oriented type.
In Fig. 10, the maximum response values of shear force and bending moment on the skeleton curves are illustrated. The ductility factor becomes smaller as the embedment depth increases, and the factor is remarkably large in the middle level of the structure because the structure is shaped setback at the level as shown in Fig. 4. In Fig. 11, the floor response spectra are drawn at different levels of the structure. The peaks of the spectra move toward a lower period range and the peak values become smaller in lower levels as the embedment depth increases. Consequently both the nonlinear behaviors of the structure and the embedment effects due to the soil-structure interaction are appropriately represented by the present method.

5. Conclusion

The present method based on the substructure approach can be applied as one of the effective and practical method for the aseismic design analysis which takes account of both the nonlinear behaviors of a nuclear reactor building and the soil-structure interaction effects.

References


Fig. 1   Separation of soil-structure interaction system

(a) Total system  (b) Structure system  (c) Soil system

Fig. 3   Schematic illustration of frequency dependency of impedance functions

Fig. 4   Section of typical BWR-MARKII type reactor building

Fig. 5   Mathematical model of structure system considering embedment effects
Fig. 6 Finite element model of soil system

(a) Swaying mode of top-sidewall

(b) Swaying mode of rigid base mat

(c) Rocking mode of rigid base mat

Fig. 7 Impedance functions of excavated soil for various modes of vibration

Fig. 8 Distributions of maximum response values
(a) Max. acceleration (b) Max. shear force

Fig. 9 Floor response spectra at different levels of structure
(Simulated earthquake motion: Max. 250Gal)
Table 1  Sample cases of elasto-plastic analyses

<table>
<thead>
<tr>
<th>Case</th>
<th>Analysis model</th>
<th>Matrix form</th>
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</thead>
<tbody>
<tr>
<td>case 1</td>
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<td>(3x3)</td>
</tr>
<tr>
<td>case 2</td>
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</tr>
<tr>
<td>case 3</td>
<td><img src="image" alt="Case 3 Model" /></td>
<td>(8x8)</td>
</tr>
</tbody>
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![Graphs and charts](image)

Fig. 10  Maximum response values of shear force and bending moment on skeleton curves

**Fig. 11**  Floor response spectra at different levels of structure (Simulated earthquake motion: Max. 500Gal)