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EARTHQUAKE-RESPONSE CONSTRAINED DESIGN OF PILE-SUPPORTED ELASTIC SHEAR BUILDINGS FOR SITE-DEPENDENT RESPONSE SPECTRA

Tsuneyoshi NAKAMURA and Izuru TAKEWAKI

Department of Architecture, Kyoto University,
Sakyo-ku, Kyoto, Japan

SUMMARY

The purpose of this paper is to propose a new method of earthquake-response constrained design of pile-supported elastic shear buildings. This method is an extension of the method due to Nakamura and Yamane (Ref. 1) of earthquake-response constrained design of elastic shear buildings with fixed bases. The following problem is considered: Given the stiffnesses and the damping coefficients of the piles and a site-dependent design response spectrum for a shear building, find the story stiffnesses of the shear building for a specified distribution of mean maximum interstory drifts. The validity of this method is demonstrated by time history analysis to design earthquakes.

INTRODUCTION

Most of the previous investigations on the effect of soil-structure interaction have been primarily concerned with methods of behavioral analysis (Refs. 2, 3). No direct method of design in the sense of the paper due to Nakamura and Yamane (Ref. 1) appears to have been presented even for elastically supported elastic shear buildings to be designed so as to exhibit a specified distribution of mean maximum responses to design moderate earthquakes. The term "direct method" implies that the stiffnesses of the shear building are determined almost directly from the desired response levels and distributions without a large number of successive improvements of designs mostly by means of sensitivity analysis.

In this paper, a pile-supported building is modeled as an elastically supported elastic shear building. The purpose of this paper is as follows: (1) to derive the closed form solution to a problem of minimizing a weighted sum of story stiffnesses of an elastically supported shear building subject to a fundamental natural frequency constraint; (2) to show that the level of mean maximum interstory drifts can be adjusted in terms of the fundamental natural frequency of the elastically supported shear building; (3) to illustrate that, if the weight coefficients are regarded as parameters for adjustment, then the distribution of the mean maximum interstory drifts can be adjusted so as to coincide with a specified one; (4) to derive several sets of story stiffnesses of shear buildings which exhibit specified distributions of mean maximum interstory drifts under design moderate earthquakes compatible with three sets of site-dependent design response spectra; and (5) to clarify the influence of soil condition on the distributions of story stiffnesses and story shear coefficients.

OPTIMUM DESIGN OF PILE-SUPPORTED ELASTIC SHEAR BUILDINGS FOR SPECIFIED FUNDAMENTAL NATURAL FREQUENCY

Consider an elastically supported elastic shear building of f stories shown in Fig.1(a). The elastic support springs represent the stiffnesses of piles. The mechanical properties of piles and soil are assumed to be given. The piles are to be connected to a rigid foundation and the stiffnesses and damping coefficients of each pile are to be evaluated from the single pile theory due to Novak et al.(Ref. 4)(See Fig.1(b)). Frequency independence of the stiffnesses and damping coefficients of the piles is reasonably assumed according to Refs. 2 and 4. Let m_i and I_{Ri} denote the mass and the moment of inertia of the mass around its centroid of floor $i+1$, respectively. I_{Ri} and m_i are not to be varied in any stage of optimization. The story height of story i is denoted by h_i. Let k_i denote the stiffness of the structural elements of story i with respect to the relative horizontal displacement between floor i and floor $i+1$. A set of story stiffnesses $k^T=\{k_1\ k_2\ \cdots\ k_f\}$ is taken as design variables and is called 'design k'.

It is assumed that the total cost of the structural elements is represented by

$$W(k) = b^T k \tag{1}$$

where $b^T=\{\beta_1{}^2\ \beta_2{}^2\ \cdots\ \beta_f{}^2\}$ and β_i denotes the positive square root of the ith cost factor to be weighted on k_i. Consider the following optimum design problem.

PROBLEM EFNF

For an elastically supported shear building with a specified set (b, M, k_{HH}, k_{RR}, k_{HR}), find k that minimizes $W(k)$ given by Eq.(1) subject to the constraint on undamped fundamental natural frequency

$$\omega_1(k)^2 = \min_{\phi} \frac{\phi^T K(k)\phi}{\phi^T M\phi} = \omega_a{}^2 \tag{2}$$

where $\omega_1(k)$, ω_a, $K(k)$, M and ϕ denote the fundamental natural frequency of the elastically supported shear building of design k, the specified fundamental natural frequency, the stiffness matrix, the mass matrix and an arbitrary mode vector, respectively. The first and second columns $(k_{HH}\ \ k_{RH})^T$, $(k_{HR}\ \ k_{RR})^T$ of a submatrix in $K(k)$ denote the stiffnesses of the support springs with respect to swaying and rocking motions, respectively.

The necessary and sufficient conditions for global optimality and the closed form solution to PROBLEM EFNF can be derived with a procedure similar to that in Ref.5 where the coupling term k_{HR} has not been included. The closed form solution to PROBLEM EFNF is derived as follows.

$$k_j = \frac{\Omega_a}{\beta_j} \sum_{i=j}^{f} m_i (U_F{}^* + \Theta_F{}^* H_i + \sum_{r=1}^{i} \beta_r) \tag{3}$$

where

$$U_F{}^* = \frac{D_2 D_5 - D_3 D_4}{D_1 D_4 - D_2{}^2}, \quad \Theta_F{}^* = \frac{D_2 D_5 - D_1 D_5}{D_1 D_4 - D_2{}^2}$$

$$D_1 = \sum_{i=0}^{f} m_i - \frac{k_{HH}}{\Omega_a}, \quad D_2 = \sum_{i=1}^{f} m_i H_i - \frac{k_{HR}}{\Omega_a}, \quad D_3 = \sum_{i=1}^{f} m_i \sum_{j=1}^{i} \beta_j$$

$$D_4 = \sum_{i=1}^{f} m_i H_i{}^2 + \sum_{i=0}^{f} I_{Ri} - \frac{k_{RR}}{\Omega_a}, \quad D_5 = \sum_{i=1}^{f} m_i H_i \sum_{j=1}^{i} \beta_j, \quad H_i = \sum_{j=1}^{i} h_j \tag{4}$$

DESIGN FOR A SPECIFIED LEVEL OF MEAN MAXIMUM INTERSTORY DRIFTS UNDER DESIGN MODERATE EARTHQUAKES

If a design displacement response spectrum $S_D(T;\, h)$ is given for an elastically supported shear building, then the mean maximum response of interstory drift excluding a rocking component between floor j and floor $j+1$ may be evaluated by the well-known SRSS technique.

$$\delta_{jmax} = \sqrt{\sum_{r=1}^{N} \{ \nu^{(r)} (\phi_j^{(r)} - \phi_{j-1}^{(r)}) S_D(T_r;\, h^{(r)}) \}^2} \tag{5}$$

where T_r, $h^{(r)}$, $\nu^{(r)}$, $\phi_{j-1}^{(r)}$ and N denote the rth natural period, the rth damping ratio, the rth participation factor, the jth component of the rth eigenvector and the number of adopted modes, respectively.

In this paper, the damping ratio corresponding to each mode is approximately derived by ignoring off-diagonal terms in the process of decomposition of equations of motion into a set of modal equations. Let c_{HH}, c_{RR} and c_{HR} denote the damping coefficients of the set of piles corresponding to k_{HH}, k_{RR} and k_{HR}, respectively.

Consider next the following problem.

PROBLEM ECMD

For an elastically supported shear building with a specified set of M, k_{HH}, k_{RR}, k_{HR}, c_{HH}, c_{RR}, c_{HR} subjected to design earthquakes compatible with a non-decreasing design spectrum $S_D(T;\, h)$, find k^c which would result in a specified distribution of the mean maximum interstory drifts, ie.

$$\delta_{jmax} = \bar{\delta} \qquad \text{(for all } j\text{)} \tag{6}$$

For the sake of simplicity, a shear building with k which is the solution to PROBLEM EFNF corresponding to $b=1$ will be referred to as a shear building of design EFNF1 in the following. Similarly a shear building with k^c which is the solution to PROBLEM ECMD will be referred to as a shear building of design ECMD. Numerical investigations by use of the site-dependent design response spectrum due to Mostaghel and Ahmadi (Ref. 6) clarified the characteristics that the mean maximum interstory drift δ_{1max} of the shear building of design EFNF1 is a good approximation of the mean maximum interstory drift δ_{1max} of the shear building of design ECMD so far as the two shear buildings have the same fundamental natural period. This characteristics may be utilized in the following procedure for finding a solution to PROBLEM ECMD : (i) Find the fundamental natural period T_a of the shear building of design EFNF1 δ_{1max} of which attains the specified value $\bar{\delta}$. (ii) Find the story stiffnesses of the shear building of design ECMD which has the fundamental natural period T_a. The second procedure (ii) is successfully achieved by applying the same procedure of modifying b as that due to Nakamura and Yamane (Ref. 1) for a shear building with a fixed base.

A design derived through this procedure may also be called a spectrum-compatible displacement-constrained design.

DESIGN EXAMPLES

Twenty design examples of 10, 15, 20 and 30 stories are presented for each of the three different site-dependent design response spectra. The common prescribed parameters throughout all the design examples are the floor masses, $m_0=75.0(ton)$, $m_1 \sim m_f=45.0(ton)$, the moments of inertia, $I_{R0}=4.0\times10^6(ton\cdot cm^2)$, $I_{R1} \sim I_{Rf}=2.4\times10^6(ton\cdot cm^2)$ and the story heights, $h_1 \sim h_f=350.0(cm)$.

The site-dependent design response spectrum due to Mostaghel and Ahmadi (Ref. 6) is adopted in this paper which depends upon the characteristic period of soil, the maximum ground acceleration and the damping ratio. Consider here three types of soils which have the characteristic periods of $T_c=0.4$, 0.6 and $0.8(sec)$. The soils with T_c=0.4, 0.6 and 0.8(sec) are called soils of type I, II and III, respectively. The maximum ground acceleration is assumed to be $201 (cm/sec^2)$ for all the soil conditions.

It is assumed here that a rigid foundation is supported by two piles and the stiffnesses and damping coefficients of the foundation with respect to swaying and rocking motions are due to the dynamic interaction between the piles and soil. The stiffnesses and damping coefficients of the piles are to be evaluated from the formulas due to Novak et al.(Ref. 4). Let k_x, k_z and k_ψ denote the stiffnesses of a pile which are defined with respect to the horizontal and vertical displacements and rotation of the pile head, respectively. Let c_x, c_z and c_ψ denote the corresponding damping coefficients of the pile. The coupling terms between the horizontal displacement and rotation are denoted by k_c and c_c for stiffness and damping, respectively. If the group effect of the piles is neglected and the distance between each pile and the centroid of the foundation is denoted by x_r, then the stiffnesses and damping coefficients of the set of piles are evaluated as given in Table 1. The damping matrix of the elastically supported shear building is to be expressed by a superposition of the damping matrix of the shear building with a fixed base and that of the foundation. The damping matrix of the shear building with a fixed base is to be proportional to the stiffness matrix in such a way that the damping ratio for the lowest mode is 2%.

Fig.2 shows the distributions of story stiffnesses of the shear buildings of design ECMD of 20 stories with the specified levels of the maximum interstory drift of $\bar{\delta}$=1.0, 1.25, 1.5, 2.0, 2.5cm under the three soil conditions. It can be observed from Fig.2 that the shear buildings of design ECMD designed for soil of type III require the greatest values of story stiffnesses among the three. Fig.3 shows the distributions of story shear coefficients of these shear buildings. Fig.3 indicates that the shear buildings designed for soil of type III attain the greatest values of story shear coefficients.

Fig.4 shows the plots of the base shear coefficients c_1 with respect to the fundamental natural periods T_1 of the elastically supported shear buildings of design ECMD of 10, 15, 20 and 30 stories with $\bar{\delta}$=1.0, 1.25, 1.5, 2.0, 2.5cm for the three soil conditions. It can be observed from Fig.4 that a T_1-c_1 curve has a fairly different level of magnitude depending upon the soil conditions.

In order to demonstrate that the shear building of design ECMD indeed exhibits the specified response of mean maximum interstory drifts under spectrum-compatible artificial earthquakes, time history analysis has been performed on three shear buildings of 20 stories with $\bar{\delta}$=1.5cm. A set of ten different design earthquakes has been generated by the SIMQKE program (Ref. 7) for each of the three soil conditions so as to be compatible with the target velocity response spectrum. Newmark-β method with β=0.25 has been adopted in step-by-step time integration conducted in a matrix form. The circular marks in Fig.5 show the distributions of mean maximum interstory drifts of the three shear buildings under each set of ten design earthquakes which are in good agreement with the specified distributions of mean maximum interstory drifts.

In order to demonstrate further that the shear building of design ECMD also exhibits fairly desirable responses to recorded earthquakes, time history analysis has been conducted for two recorded earthquakes. The shear building of 20 stories with $\bar{\delta}$=1.5cm for soil of type I has been analysed for the magnified earthquake of EL CENTRO NS 1940 with factor 0.326 and for the magnified earthquake of TAFT EW 1952 with factor 0.584. The magnification factors have been determined in such a way that the corresponding response spectrum would just inscribe the target spectrum of the design earthquakes for soil of type I. Black and hollow circular marks in Fig.6 show δ_{jmax} distributions for EL CENTRO and TAFT earthquakes, respectively. It is apparent that both of two sets of δ_{jmax} distributions are fairly uniform in lower stories and exhibit fairly small deviations in upper several stories.

CONCLUSIONS

A new method of earthquake-response constrained design of pile-supported elastic shear buildings has been proposed for a site-dependent design response spectrum. It has been demonstrated by time history analysis that every shear building designed by means of this method indeed exhibits the specified distribution of mean maximum interstory drifts to design earthquakes with a reasonable accuracy. It has been demonstrated further that such a shear building exhibits fairly uniform distributions of maximum interstory drifts to two magnified recorded earthquakes.

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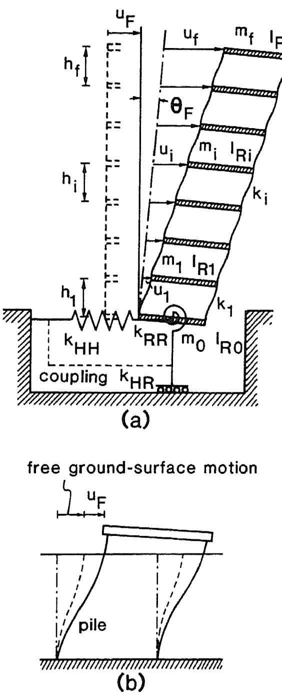


Fig.1 Elastically supported shear building

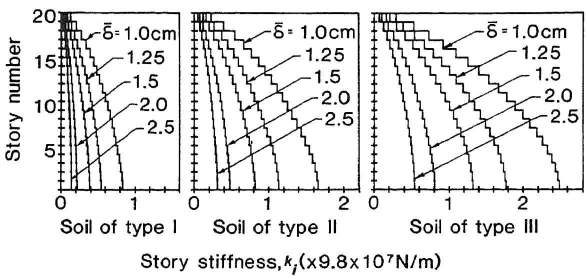


Fig.2 Distributions of story stiffnesses of 20-story shear buildings with $\bar{\delta}$=1.0,1.25,1.5,2.0,2.5cm

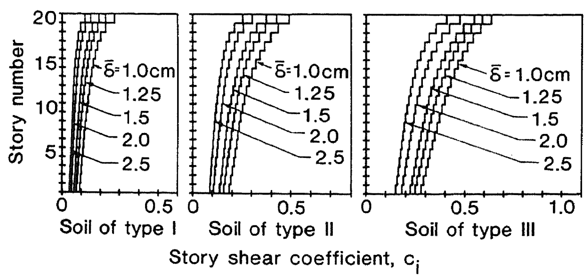


Fig.3 Distributions of story shear coefficients of 20-story shear buildings with $\bar{\delta}$=1.0,1.25,1.5,2.0,2.5cm

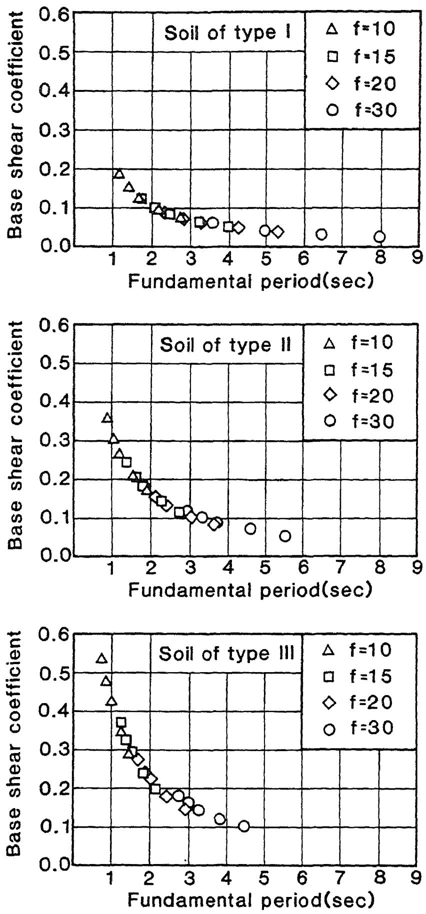


Fig.4 Plots of base shear coefficients with respect to fundamental natural period

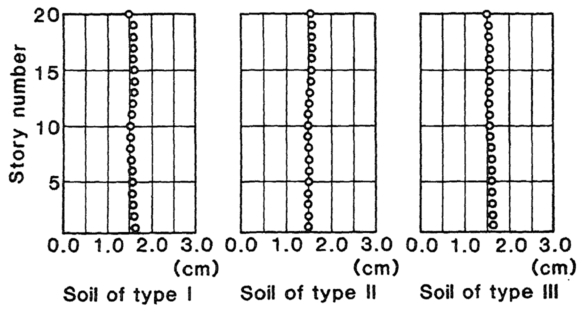
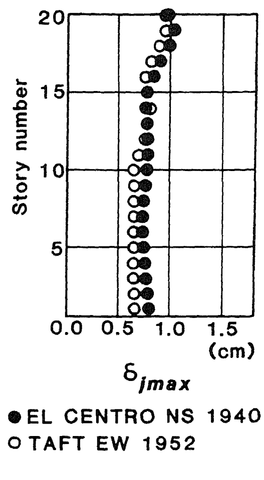


Fig.5 Distributions of mean maximum interstory drifts of 20-story shear buildings with $\bar{\delta}$=1.5cm to ten spectrum-compatible artificial earthquakes

Fig.6 Distributions of maximum interstory drifts of the 20-story shear building with $\bar{\delta}$=1.5cm for soil of type I to two magnified recorded earthquakes



	Soil of type I (T_c=0.4sec)	Soil of type II (T_c=0.6sec)	Soil of type III (T_c=0.8sec)
$\sum k_x$ (N/m)(=k_{HH})	1.414E09	7.756E08	3.987E08
$\sum c_x$ (N·sec/m)(=c_{HH})	1.016E07	8.536E06	5.994E06
$\sum (k_\psi+k_z x_r^2)$ (N·m/rad)(=k_{RR})	8.392E10	6.581E10	5.586E10
$\sum (c_\psi+c_z x_r^2)$ (N·m·sec/rad)(=c_{RR})	3.313E08	2.640E08	1.539E08
$\sum k_c$ (N/rad)(=k_{HR})	-1.716E09	-1.171E09	-8.040E08
$\sum c_c$ (N·sec/rad)(=c_{HR})	-7.252E06	-7.709E06	-7.254E06

Table 1 Stiffnesses and damping coefficients of the set of piles for three soil conditions