IV: Design Issues and Seismic Performance

- Design concepts for yielding structures on flexible foundation, by Javier Avilés and Luis E. Pérez-Rocha.

- Seismic design of a structure supported on pile foundation considering dynamic soil-structure interaction, by Yuji Miyamoto, Katsuichiro Hijikata and Hideo Tanaka.

- Implementation of soil-structure interaction models in performance based design procedures, by Jonathan P. Stewart, Craig Comartin, and Jack P. Moehle.

- Design and actual performance of a super high R/C smokestack on soft ground, by Shinichiro Mori.

Design Concepts for Yielding Structures on Flexible Foundation

Javier Avilés\textsuperscript{a) and Luis E. Pérez Rocha\textsuperscript{b) }}

The effects of soil-structure interaction on the seismic response of a simple yielding system representative of code-designed buildings are investigated. The design concepts developed earlier for fixed-base systems are extended to account for such effects. This is done by use of a non-linear replacement oscillator recently proposed by the authors, which is characterized by an effective ductility along with the known effective period and damping of the system for the elastic condition. Numerical results are computed for interaction conditions prevailing in Mexico City, the interpretation of which shows the relative importance of the elastic and inelastic interaction effects in this location. Strength reduction factors relating elastic to inelastic response spectra are also examined. To account for their behavior in a design context, a site-dependent reduction rule proposed elsewhere for fixed-base systems is suitably adjusted for interacting systems, using the solution for the non-linear replacement oscillator. Finally, a brief explanation is given of the application of this information in the formulation of the new interaction provisions in the Mexico City building code.

INTRODUCTION

As it is well-known, the performance-based seismic design requires more accurate analyses including all potential important factors involved in the structural behavior. This is the way to improve the prediction of the expected level of structural damage associated with a given level of earthquake. One of these factors is the soil-structure interaction (SSI). Although the SSI effects have been the subject of numerous investigations in the past, they

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have been generally examined at the exclusion of the non-linear behavior of the structure. The first studies of SSI using an analogy with a single oscillator were made by Jennings and Bielak (1973) and Veletsos and Meek (1974). They showed that the effects of inertial SSI can be sufficiently approximated by simply modifying the fundamental period and associated damping of the fixed-base structure. After these investigations, the increase in the natural period and the change in the damping ratio (usually an increase) resulting from the soil flexibility and wave radiation have been extensively studied by several authors (Bielak J, 1975; Luco JE, 1980; Wolf JP, 1985; Avilés J and Pérez-Rocha LE, 1996), using as base excitation a harmonic motion with constant amplitude. Based on the same analogy, the effects of kinematic SSI on the relevant dynamic properties of the structure have also been evaluated (Todorovska and Trifunac, 1992; Avilés and Pérez-Rocha, 1998; Avilés et al, 2002) for different types of travelling seismic waves, showing that they are relatively less important. The modification of the period and damping by total SSI results in increasing or decreasing the structural response with respect to the fixed-base value, depending essentially on the resonant period in the response spectrum.

The inelastic response of rigidly supported structures was first studied by Veletsos and Newmark (1960). By using pulse-type excitations and broad-band earthquakes, they derived simple approximate rules for relating the maximum deformation and yield resistance of non-linear structures to the corresponding values of associated linear structures. There are not similar relationships taking the soil flexibility into consideration. Practical solutions are needed to easily estimate the yield resistance of flexibly supported structures that is required to limit the displacement ductility demand to the specified available ductility. The transient response of an elastoplastic surface-supported structure on an elastic half-space has been examined by Veletsos and Verbic (1974), who concluded that the structural yielding increases the relative flexibility between the structure and soil and hence decreases the effects of SSI. Based on the harmonic response of a bilinear hysteretic structure supported on the surface of a viscoelastic half-space, Bielak (1978) has shown that the resonant deformation may be significantly larger than would result if the supporting soil were rigid.

In many cases the SSI effects are of little practical importance, and they are less so for yielding systems. In the conditions of Mexico City, however, SSI is known to produce very
significant effects (Reséndiz and Roesset, 1986). In some structures, they are even more important than the so-called site effects, recognized as the crucial factor associated with the soil characteristics. A recent study by Avilés and Pérez-Rocha (2003) has revealed that the effects of SSI for yielding systems follow the same general trends observed for elastic systems, although not with the same magnitude. It has been detected that the SSI effects may result in large increments or reductions of the required strengths and expected displacements, with respect to the corresponding fixed-base values. When both SSI and structural yielding take place simultaneously, their combined effects prove to be beneficial for structures with fundamental period longer than the dominant period of the site, but detrimental if the structure period is shorter than the site period.

The design approach used so far to take the effects of SSI into account has not changed over the years: a replacement oscillator represented by the effective period and damping of the system. The most extensive efforts in this direction were made by Veletsos (1977) and his coworkers. Indeed, their studies form the basis of the SSI provisions currently in use in the US seismic design codes (ATC, 1978; NEHRP, 2000) for buildings. Although this approach does not account for the ductile capacity of the structure, it has been implemented in many codes in the world for the convenience of using standard free-field response spectra in combination with the system’s period and damping. But considering that in some practical cases the SSI effects may differ greatly between elastic and inelastic systems, the current SSI provisions based on elastic structural models could not be directly applicable to seismic design of typical buildings, expected to deform considerably beyond the yield limit during severe earthquakes.

This work is aimed at improving limitations in the way SSI is presently incorporated into code design procedures, specially that concerning with the effects on the structural ductility. As noted by Crouse (2002), the current SSI provisions in the ATC (1978) and NEHRP (2000) documents have a significant shortcoming, which is the lack of a link between the strength reduction factors and the effects of SSI. This author has suggested that the SSI provisions that allow a reduction in the base shear, after it has been reduced by ductility, should be used with caution or ignored altogether. Such a deficiency has also been recognized by Stewart et al (2003) in the recent revision to the SSI procedures in the NEHRP design provisions. The
strength reduction factors are supposed to account for the ductile capacity of the structure. They have been extensively studied in the past for firm ground, and even for soft soils considering site effects, but always excluding SSI. The use of these factors assuming rigid base may lead to strength demands considerably different from those actually developed in structures with flexible foundation (Avilés and Pérez-Rocha, 2003). Errors can be made either for the safe side, underestimating such factors, or for the unsafe side, overestimating them.

The soil-structure system investigated herein is formed by an elastoplastic one-story structure placed on a rigid foundation that is embedded into a layer of constant thickness underlain by a homogeneous half-space. The earthquake excitation is composed of vertically propagating shear waves. This interacting system, although simple, ensures a wide applicability of results in the current design practice because it satisfies various requirements stipulated by building codes. Design concepts are developed by reference to a non-linear replacement oscillator that is defined by an effective ductility together with the known effective period and damping of the system for the elastic condition. This approach supplies a practical and reliable tool to account fully for the effects of SSI in yielding structures. It forms the basis of the new SSI provisions in the Mexico City building code. The implemented procedure allows determination of required strengths and expected displacements in a more rational way, based on the use of site-dependent strength reduction factors properly adjusted to include SSI.

SIMPLIFIED REFERENCE MODEL

System parameters

The soil-structure system under investigation is depicted in figure 1. It consists of an elastoplastic one-story structure supported by a rigid foundation that is embedded in a viscoelastic stratum of constant thickness overlying a uniform viscoelastic half-space. This interacting system is similar to that considered in the ATC (1978) and NEHRP (2000) provisions, with the addition of the foundation depth, soil layering and structural yielding.
The structure is characterized by the height \( H_e \), mass \( M_e \) and mass moment of inertia \( J_e \) about a horizontal centroidal axis. The natural period and damping ratio of the structure for the elastic and fixed-base conditions are given by

\[
T_e = 2\pi \left( \frac{M_e}{K_e} \right)^{1/2} \quad \text{and} \quad \zeta_e = \frac{C_e}{2(K_eM_e)^{1/2}},
\]

in which \( C_e \) and \( K_e \) are the viscous damping and initial stiffness of the structure. The one-story structure may be viewed as representative of more complex multistory buildings that respond essentially as a single oscillator in their fixed-base condition. In this case, it would be necessary to interpret the parameters of the one-story structure as those of the multistory building when vibrating in its fixed-base fundamental mode. The foundation is assumed as a circular mat of radius \( r \), perfectly bonded to the surrounding soil, with depth of embedment \( D \), mass \( M_c \) and mass moment of inertia \( J_c \) about a horizontal centroidal axis. The layer is characterized by the thickness \( H_s \), Poisson’s ratio \( \nu_s \), mass density \( \rho_s \), shear wave velocity \( \beta_s \) and hysteretic damping ratio \( \zeta_s \). The corresponding material properties of the half-space are defined by \( \nu_o \), \( \rho_o \), \( \beta_o \) and \( \zeta_o \).

![Figure 1. Single elastoplastic structure placed on a cylindrical foundation embedded in a stratum overlying a half-space, under vertically incident shear waves.](image-url)
For the results reported in this study, it was assumed that $\frac{M_e}{M_o} = 0.25$, $\frac{J_e}{J_o} = 0.3$, $\rho_s/\rho_o = 0.8$, $\beta_s/\beta_o = 0.2$, $\zeta_s = 0.05$, $\zeta_o = 0.03$, $\nu_s = 0.45$ and $\nu_o = 1/3$. These values are intended to approximate typical building structures as well as site conditions as those prevailing in Mexico City.

Figure 2. Force and displacement demands on the elastoplastic structure and an associated elastic structure with the same initial stiffness, based on the equal displacement rule.

In figure 2 are exhibited the pertinent relations between force and displacement demands on the resisting elements of the elastoplastic structure and an associated elastic structure with the same initial stiffness. For the elastoplastic structure, the yield resistance is denoted by $V_y$, the yield deformation by $U_y$ and the maximum deformation by $U_m$. The ductility factor is then defined by $\mu_e = U_m/U_y$. If we assume that the maximum displacement demands are identical for both structures, which is true in the long-period range, the ductility factor can also be written as $\mu_e = V_m/V_y$. It is seen that, based on the equal displacement rule, the strength $V_y$ required by the elastoplastic structure is assessed by reducing the strength $V_m$ developed in the elastic structure with the prescribed allowable ductility $\mu_e$. Hence, the strength reduction factor $R_\mu = \mu_e$ applies in this case.
Equilibrium equations

The interacting system is subjected to vertically incident shear waves, propagating along the z-axis with particle motion parallel to the x-axis. The horizontal displacement at the ground surface generated by this wave excitation is denoted by $U_g$. The presence of the foundation modifies the free-field motion by the addition of diffracted and scattered waves. This results in a foundation input motion consisting of the horizontal and rocking components denoted by $U_o$ and $\Phi_o$, respectively.

The degrees of freedom of the structure-foundation system are the relative displacement of the structure $U_e$, the displacement of the foundation $U_c$ relative to the horizontal input motion $U_o$, and the rocking of the foundation $\Phi_c$ relative to the rocking input motion $\Phi_o$. The equilibrium equations of the coupled system in the time domain can be written as:

$$ M_s \ddot{U}_s(t) + P_s(t) = -M_o \ddot{U}_o(t) - J_o \ddot{\Phi}_o(t) \tag{1} $$

where $U_s = \{U_e, U_c, \Phi_e\}^T$ is the displacement vector of the system, whereas $M_o$ and $J_o$ are load vectors and $M_s$ is the mass matrix of the system, given elsewhere (Avilés and Pérez-Rocha, 1998). A dot superscript denotes differentiation with respect to time $t$. Also, $P_s = \{P_e, P_c, M_s\}^T$ is the vector of internal forces of the system. Here $P_e(t) = C_e \dot{U}_e(t) + V_e(t)$, with $V_e$ being the restoring force of the structure. The interaction force $P_s$ and moment $M_s$ of the soil acting on the foundation are defined by the convolution integral

$$ \begin{pmatrix} P_s(t) \\ M_s(t) \end{pmatrix} = \int_0^t \begin{bmatrix} \tilde{K}_{hh}(t-\tau) & \tilde{K}_{hr}(t-\tau) \\ \tilde{K}_{rh}(t-\tau) & \tilde{K}_{rr}(t-\tau) \end{bmatrix} \begin{bmatrix} U_e(\tau) \\ \Phi_e(\tau) \end{bmatrix} d\tau \tag{2} $$

where $\tilde{K}_{hh}$, $\tilde{K}_{rr}$ and $\tilde{K}_{hr}$ are respectively the horizontal, rocking and coupling dynamic stiffnesses of the foundation in the time domain. If the soil behaves linearly, these quantities...
can be evaluated in the frequency domain and then transformed into the time domain by application of the inverse Fourier transform. When they are evaluated in the frequency domain, the following complex-valued function applies:

\[
\tilde{K}_{mn}(\omega) = K_{mn}(\eta) + i\eta C_{mn}(\eta); \quad m, n = h, r
\]  

(3)

where \( \eta = \omega r/\beta_s \) is the dimensionless frequency, \( \omega \) being the exciting circular frequency. The terms \( K_{mn} \) and \( C_{mn} \) represent the frequency-dependent springs and dampers by which the supporting soil is replaced for each vibration mode of the foundation. The linear springs account for the stiffness and inertia of the soil, whereas the viscous dampers account for the energy dissipation by hysteretic behavior and wave radiation in the soil.

**Method of solution**

The analysis of SSI may be conducted either in the frequency domain using harmonic impedance functions or in the time domain using impulsive impedance functions. The frequency-domain analysis, however, is not practical for structures that deform into the non-linear range. On the other hand, the time-domain analysis can be accomplished by use of frequency-independent foundation models, so that constant springs and dampers are employed to represent the soil, as indicated for instance by Wolf and Somaini (1986). With this simplification, the convolution integral describing the soil interaction forces is avoided, and thus the integration procedure of the equilibrium equations is carried out as for the fixed-base case. Calculations are usually performed with the values of stiffness for zero frequency and the values of damping for infinite frequency. This is known as the doubly asymptotic approximation (Wolf, 1988) and it is, in effect, asymptotically exact at both low and high frequencies. To improve the approximation, springs and dampers may be evaluated at other specific frequency, for example at the system frequency for the elastic condition, as done in this investigation, or may be averaged over the frequency range of interest.

To compute the step-by-step non-linear response of the elastically supported structure, a time-integration scheme based on the Newmark method was applied. Required strengths are
computed by iteration on $V_y$ until the ductility demand given by $\mu_{\text{max}} = \frac{U_{e_{\text{max}}}}{U_y}$ is the same as the specified available ductility $\mu_e$. The iteration process is stopped when the difference between the computed and target ductilities is considered satisfactory for engineering purposes. The tolerance chosen here was of 1%. Due precautions are taken when the ductility demand does not increase monotonically as the yield strength of the structure decreases. In this case, there is more than one strength that produces a ductility demand equal to the target ductility. However, only the largest strength is relevant for design.

**ELASTIC COMPUTATION OF IMPEDANCE FUNCTIONS AND INPUT MOTIONS**

Fundamental steps in the analysis of SSI are the elastic computations of impedance functions and input motions for the foundation. The effect of yielding in the soil can be considered approximately in this approach. It is a common practice to account for the primary nonlinearities caused by the free-field motion, using the soil properties consistent with the induced strains; the secondary nonlinearities caused by the base shear and overturning moment acting on the foundation are usually neglected.

![Normalized springs and dampers for the horizontal, rocking and coupling modes of an embedded foundation with $D/r = 0.5$ in a soil stratum with $H_s/r = 3$.](image)

*Figure 3.* Normalized springs and dampers for the horizontal, rocking and coupling modes of an embedded foundation with $D/r = 0.5$ in a soil stratum with $H_s/r = 3$. 
The impedance functions are obtained making use of an efficient numerical technique based on the thin layer element method (Tassoulas and Kausel, 1983). In this technique, the base of the stratum is taken fixed. This is not, however, a serious restriction because it is always possible to choose a depth that is large enough to simulate the presence of an underlying half-space. The practical importance of using a rigorous technique is that the foundation embedment and layer depth affect significantly the springs and dashpots by which the soil is replaced. Probably, the most important effect is that, for a soil layer, a cutoff frequency exists below which the radiation damping is not activated (Meek and Wolf, 1991). The normalized springs and dampers (by using $G_s = \beta^3 \rho_s$) so obtained are displayed in figure 3 for $D/r = 0.5$ and $H_s/r = 3$. Given that the springs reflect both the stiffness and inertia of the soil, note that they can take negative values.

![Graphs showing impedance functions](image)

Figure 4. Amplitudes of the horizontal and rocking input motions for an embedded foundation with $D/r = 0.5$ in soil strata under vertically incident shear waves.

Having determined the impedance functions, the input motions are obtained by application of the averaging method of Iguchi (1984). With this simple but efficient technique, the harmonic response of the foundation to any wave excitation is calculated by taking a weighted average of the free-field displacements along the soil-foundation interface, and adding the displacement and rocking caused by the resultant force and moment associated with the free-field tractions along this surface. The transfer functions of the horizontal and rocking input motions so obtained are exhibited in figure 4 for the same data of figure 3. Incidentally, they prove to be independent of the layer depth for the case of...
vertically propagating shear waves. The effect of this parameter is implicit in the free-field motion used for normalization.

If the transfer functions $Q_h(\omega) = U_o(\omega)/U_g(\omega)$ and $Q_r(\omega) = r\Phi_o(\omega)/U_g(\omega)$ are known, the time histories of the foundation input motion for a particular earthquake are determined from a Fourier analysis as follows: (1) to compute the direct Fourier transform, $\hat{U}_g(\omega)$, of the horizontal free-field acceleration, $\dot{U}_g(t)$; (2) to calculate the Fourier transforms of the horizontal and rocking input accelerations as $\hat{U}_h^*(\omega) = Q_h(\omega)\hat{U}_g^*(\omega)$ and $\hat{\Phi}_r^*(\omega) = Q_r(\omega)\hat{U}_g^*(\omega)/r$; and (3) to compute the time histories of the foundation input motion, $\hat{U}_o(t)$ and $\hat{\Phi}_o(t)$, by taking the inverse Fourier transforms of $\hat{U}_o^*(\omega)$ and $\hat{\Phi}_o^*(\omega)$.

**NON-LINEAR REPLACEMENT OSCILLATOR**

The starting point for the simplified approach presented next to account for the SSI effects is the assumption that the peak non-linear response of the actual flexible-base structure may be approximated by that of a modified rigid-base structure having an equivalent ductility factor to be defined, and whose initial natural period and damping ratio are given by the known effective period and damping of the system for the elastic condition.

![Figure 5](image_url)

**Figure 5.** (a) Interacting system excited by the foundation input motion and (b) replacement oscillator excited by the free-field motion at the ground surface.
Effective period and damping of system

We shall call \(\tilde{T}_e\) and \(\tilde{\zeta}_e\) to the effective period and damping of the system. These quantities can be determined using an analogy between the interacting system excited by the foundation input motion and a replacement oscillator excited by the free-field motion (Avilés and Pérez-Rocha, 1998), as illustrated in figure 5 introducing some permissible simplifications. The mass of this equivalent oscillator is identical to that of the given structure. Under harmonic base excitation, it is imposed that the resonant period and peak response of the interacting system be equal to those of the replacement oscillator. In this way, Avilés and Suárez (2002) have deduced the following expressions:

\[
\tilde{T}_e = \left( T_h^2 + T_r^2 + T_e^2 \right)^{\frac{1}{2}}
\]

\[
\tilde{\zeta}_e = \frac{1}{Q_h + (H_e / r + D / r)Q_r} \left( \frac{\tilde{\zeta}_h T_h^3}{\tilde{T}_e^3} + \frac{\zeta_r T_r^2}{\tilde{T}_e^2} + \frac{\zeta_r T_r^2}{1 + 2\tilde{\zeta}_h \tilde{T}_e^2} \right)
\]

where \(T_h = 2\pi(M_e / K_{hh})^{\frac{1}{2}}\) and \(T_r = 2\pi(M_e (H_e + D)^2 / K_{rr})^{\frac{1}{2}}\) are the natural periods if the structure were rigid and its base were only able either to translate or to rock, and \(\zeta_h = \bar{\omega}_h C_{hh} / 2K_{hh}\) and \(\zeta_r = \bar{\omega}_r C_{rr} / 2K_{rr}\) are the damping ratios of the soil for the horizontal and rocking modes of the foundation. As the natural periods \(T_h\) and \(T_r\) must be evaluated at the system frequency, \(\bar{\omega}_c = 2\pi / \tilde{T}_e\), an iterative process is required for calculating the system period from equation (4). Once this is done, the system damping is directly calculated from equation (5). It should be noted that the factor \(Q_h + (H_e / r + D / r)Q_r\) represents the contribution of kinematic interaction to the energy dissipation in the interacting system. This effect is taken into account by considering the base excitation to be unchanged, equal to the free-field motion, while the system damping is increased. By this means, the same overall result is achieved.
Figure 6. Amplitudes of the transfer functions for the interacting system (solid line) and the replacement oscillator (dashed line), considering $H_e/\beta_s T_e = 0.25$, $H_e/r = 1$, $D/r = 0.5$ and $H_s/r = 3$.

With the system’s period and damping determined by this way, a satisfactory agreement between the transfer functions of the interacting system and the replacement oscillator is obtained over a wide interval of frequencies on both sides of the resonant frequency, as shown in figure 6 for $H_e/r = 1$, $D/r = 0.5$ and $H_s/r = 3$, taking a relative stiffness of the structure and soil $H_e/\beta_s T_e = 0.25$. Note that while the system damping increases, the system period is practically not affected. As the transfer function of the interacting system is not exactly the one of a single oscillator, the replacement oscillator approach is restricted to some applications (Avilés and Suárez, 2002).

**Effective ductility of system**

To fully characterize the replacement oscillator, an equivalent ductility factor requires to be defined. We shall call $\tilde{\mu}_e$ to this factor, also referred to as the effective ductility of the system. The force-displacement relationships for the resisting elements of the actual structure and the replacement oscillator are assumed to be of elastoplastic type, as shown in figure 7. By equating the yield strengths and maximum plastic deformations developed in both systems under monotonic loading, it has been found (Avilés and Pérez-Rocha, 2003) that
\[ \tilde{\mu}_e = 1 + (\mu_e - 1) \frac{T_e^2}{\tilde{T}_e^2} \]  

This is the natural and convenient way of expressing the global ductility of an interacting system. This expression implicitly assumes that the translation and rocking of the foundation are the same in both yielding and ultimate conditions, which holds when the soil remains elastic and the structure behaves elastoplasticity. Note that the values of \( \tilde{\mu}_e \) vary from 1 to \( \mu_e \), so that the effective ductility of the system is lower than the allowable ductility of the structure. The effective ductility \( \tilde{\mu}_e \) will be equal to the structural ductility \( \mu_e \) for infinitely-rigid soil (for which \( \tilde{T}_e = T_e \)) and to unity for infinitely-flexible soil (for which \( \tilde{T}_e = \infty \)). This seems to be the most rational way of formulating a replacement oscillator with the same capacity of plastic energy dissipation as the interacting system.

Figure 7. Resistance diagrams for the actual structure (solid line) and the replacement oscillator (dashed line), considering elastoplastic behavior.

It is interesting to note that the ductility reduction from \( \mu_e \) to \( \tilde{\mu}_e \) is due to the stiffness reduction from \( K_e \) to \( \tilde{K}_e \) only. By no means this implies that the foundation flexibility reduces the ductile capacity of the structure. The apparent paradox stems from the fact that the deformation of the replacement oscillator involves both the deformation \( U_e \) of the
structure as well as the rigid-body motion $U_c + (H_c + D)\Phi_c$ induced by the translation and rocking of the foundation, which is defined indirectly by the stiffness of the oscillator. The presence of this motion is precisely the responsible for the reduction of the global ductility of the system, without any change in the degree of permissible inelastic deformation.

**Replacement oscillator and its relation to actual structure**

The replacement oscillator is considered to experience the same yield strength as the actual structure, that is:

$$V_y = \tilde{V}_y$$  \hspace{1cm} (7)

Also, both systems would experience the same plastic deformation, but different total deformations because of the difference between yield deformations, as appreciated in figure 7. Let $U_y$ and $\tilde{U}_y$ be the yield deformations of the actual structure and the replacement oscillator, respectively, and $U_m$ and $\tilde{U}_m$ the corresponding maximum deformations. Accordingly, the ductility factors are defined in each case as $\mu_e = U_m / U_y$ and $\tilde{\mu}_e = \tilde{U}_m / \tilde{U}_y$.

In view of $V_y = K_x U_y$ and $\tilde{V}_y = \tilde{K}_x \tilde{U}_y$, in which $K_x = 4\pi^2 M_e / T_e^2$ and $\tilde{K}_x = 4\pi^2 \tilde{M}_e / \tilde{T}_e^2$, it follows from equation (7) that $U_y$ and $\tilde{U}_y$ are interrelated by

$$U_y = \frac{T_e^2}{T_e^2} \tilde{U}_y$$  \hspace{1cm} (8)

By substituting $U_y = U_m / \mu_e$ and $\tilde{U}_y = \tilde{U}_m / \tilde{\mu}_e$ into equation (8), one finds that $U_m$ is related to $\tilde{U}_m$ by the expression
\[ U_m = \frac{T_e^2 \mu_e}{\tilde{T}_e \tilde{\mu}_e} \tilde{U}_m \] (9)

The difference between the deformations of the actual structure and the replacement oscillator, as revealed by equations (8) and (9), is due to the fact that the elastic deformation developed in the latter must be shared by two springs in series representing the flexibilities of the structure and foundation. In consequence, \( \tilde{U}_m - U_m \) identical to \( \tilde{U}_y - U_y \) should be interpreted as the contribution by the translation and rocking of the foundation.

**STRENGTH AND DISPLACEMENT DEMANDS**

We are now to show that, for a given earthquake, the peak response of the actual flexible-base structure with natural period \( T_e \), damping ratio \( \zeta_e \) and ductility factor \( \mu_e \) remains in satisfactory agreement with that of a modified rigid-base structure with enlarged period \( \tilde{T}_e \), increased damping \( \tilde{\zeta}_e \) and reduced ductility \( \tilde{\mu}_e \), determined according to equations (4) to (6). In particular, the validity of equations (7) and (9) will be confirmed by comparison of strength and displacement spectra determined approximately for the replacement oscillator with those obtained rigorously for the interacting system.

The control motion will be given by the great 1985 Michoacan earthquake recorded at a soft site, \( SCT \), representative of the lakebed zone of Mexico City. The dominant site period is \( T_s = 4H_s/\beta_s = 2 \) s, with \( H_s = 37.5 \) m and \( \beta_s = 75 \) m/s. Also, the empirical relationship \( H_e/T_e \approx 25 \) m/s will be assumed, by considering an inter-story height of 3.6 m, the effective height as 0.7 of the total height and the fundamental period as 0.1 s of the number of stories. Note that for any value of \( T_e \), the value of \( H_e \) is obtained from the constant ratio \( H_e/T_e \). With this, the value of \( r \) is determined from a fixed slenderness ratio \( H_e/r \) and, in turn, the value of \( D \) is determined from a fixed embedment ratio \( D/r \). This implies that the structure changes in height as a function of the period and the foundation dimensions vary when the structure height changes, as happens with many types of buildings.
Normalized strength \( \left( \frac{V_y}{M_y g} \right) \) and displacement \( \left( \frac{U_m}{U_y} \right) \) spectra are displayed in figure 8 for \( D/r = 0.5 \) and \( H_e/r = 3 \), considering elastic \( (\mu_e = 1) \) and inelastic \( (\mu_e = 4) \) behavior. Results for the fixed-base case are also included for reference. It can be seen that the strength and displacement demands for the interacting system are well predicted by using the replacement oscillator. As happens with fixed-base systems, the spectral acceleration for very short period as well as the spectral displacement for very long period are independent of the value of \( \mu_e \). While the former tends to the peak ground acceleration, the latter approaches the peak ground displacement. The degree of approximation involved in the strength spectra is the same as in the displacement spectra, since equations (7) and (9) are identical but expressed differently. Recall that the latter follows directly from the former by a simple mathematical manipulation.

![Figure 8](image)

**Figure 8.** Normalized strength and displacement spectra for the **SCT** recording of the 1985 Michoacan earthquake and a structure with \( D/r = 0.5 \) and \( H_e/r = 3 \). Exact solution for the interacting system (solid line), approximate one for the replacement oscillator (dashed line) and that without SSI (dotted line).

Although the consequences of SSI depend on the characteristics of the ground motion and the system itself, a crucial parameter is the period ratio of the structure and site. The required
strengths and expected displacements in the spectral region $T_e/T_s < 1$ become considerably greater than those predicted if the structure is assumed as rigidly supported, while the opposite effect takes place in the spectral region $T_e/T_s > 1$. Note that the inelastic displacements around the site period are smaller than the elastic ones, a fact that is more pronounced for the fixed-base case. In view of the period lengthening, the resonant response with SSI occurs for a structure period shorter than the site period, with larger amplitude than the fixed-base value because of the reduction in damping.

It is interesting to note that, for elastic behavior, the spectral ordinates around the site period are reduced extraordinarily with respect to their fixed-base values. In this case, the SSI effects are equally or more significant than those induced by site conditions. These results could explain, in part, why some structures with fundamental period close to the site period were capable of withstanding, without damage, supposedly high (and unaccounted for) strength and displacement demands during the 1985 Michoacan earthquake.

![Figure 9. Variations against period of the interaction factor for the SCT recording of the 1985 Michoacan earthquake and a structure with $D/r = 0.5$ and $H_e/r = 3$, considering $\mu_e = 1$ (solid line) and 4 (dashed line).]

To know the extent to which the SSI effects differ between elastic and inelastic systems, the interaction factor
relating flexible- to rigid-base strength spectra was computed, using the exact results given in figure 8. It is clear that this factor should be used for assessing the resistance with SSI, $V_y(\beta_s)$, starting from that without SSI, $V_y(\infty)$.

In figure 9, the shapes of $R_\beta$ for elastic and inelastic behavior are compared. It can be seen that the SSI effects are in general more important for the former case than for the latter. Results vary in an irregular manner. It is impractical to account for this variation in the context of code design of buildings. However, smooth curves can be developed for design purposes, as it will be shown later. There is a clear tendency indicating that SSI affects the structural response adversely ($R_\beta > 1$) for $T_e < T_s$ and positively ($R_\beta < 1$) for $T_e > T_s$. The largest increments and reductions are of the same order. For very short and long periods of the structure, the SSI effects are negligible.

**STRENGTH-REDUCTION FACTOR**

Contemporary design criteria admit the use of strength reduction factors to account for the non-linear structural behavior. We are to compute the ratio between the strength required for elastic behavior, $V_m(1)$, and that for which the ductility demand equals the target ductility, $V_y(\mu_e)$, that is:

$$R_\beta(T_e, \beta_s) = \frac{V_m(1)}{V_y(\mu_e)}$$  \hspace{1cm} (11)

It should be noted that this factor depends not only on the natural period $T_e$ and the ductility factor $\mu_e$, but also on the foundation flexibility measured by the shear wave velocity $V_y$.
β_s. To a lesser degree, this factor is also influenced by the damping ratio ζ_e. It is evident that determination of $R_\mu$ allows estimation of inelastic strength demands starting from their elastic counterparts.

![Figure 10](image.png)

**Figure 10.** Variations against period of the strength-reduction factor with (dashed line) and without (solid line) SSI for the SCT recording of the 1985 Michoacan earthquake and a structure with $\mu_e = 4$, $D/r = 0.5$ and $H_e/r = 3$.

Strength-reduction factors were computed by using the exact results given in figure 8. The shape of $R_\mu(\beta_s)$ is compared with that of $R_\mu(\infty)$ in figure 10. The difference between the two cases is noticeable. In general, $R_\mu(\beta_s) > R_\mu(\infty)$ for $T_e < T_s$, whereas $R_\mu(\beta_s) < R_\mu(\infty)$ for $T_e > T_s$. It can be seen that $R_\mu$ has irregular shape, inadequate to be incorporated in building codes. However, smooth curves can be developed for design purposes, as it will be shown later. The limits imposed by theory to this factor at very short and long periods of vibration are: $R_\mu = 1$ if $T_e = 0$ and $R_\mu = \mu_e$ if $T_e = \infty$, irrespective of the foundation flexibility. For other natural periods, however, there are no theoretical indications regarding the values of this factor. Note that the values of $R_\mu(\infty)$ for natural periods close to the site period are substantially higher than $\mu_e$, the largest value predicted by the equal displacement rule. It is clear that, in this period range, such a rule may be quite conservative for narrow-band earthquakes as those typical of Mexico City. Also note that site effects, reflected in that
$R_\mu > \mu_e$ around the site period, are counteracted by SSI. The reason for this is that SSI tends to shift the structure period to the long-period spectral region, for which the equal displacement rule is applied.

It should be pointed out that the strength-reduction factor given by equation (11) is to be used in combination with flexible-base elastic spectra which, in turn, can be determined from rigid-base elastic spectra using the effective period and damping of the system previously defined. By this way, the yield resistance and maximum deformation of interacting yielding systems are estimated from the corresponding values of fixed-base elastic systems.

APPROXIMATE REDUCTION RULE

As the difference between the shapes of $R_\mu(\beta_\nu)$ and $R_\mu(\infty)$ may be of large significance, the reduction of elastic strength spectra to assess inelastic strength spectra could not be attained accurately with approximate rules derived assuming rigid base. Thus, it is necessary to devise a site-dependent reduction rule that includes SSI. To this end, we are to adapt an available rigid-base rule using the solution for the non-linear replacement oscillator.

The shape of $R_\mu(\infty)$ has been extensively studied in the last years using recorded motions and theoretical considerations. In particular, Ordaz and Pérez-Rocha (1998) observed that, for a wide variety of soft sites, it depends on the ratio between the elastic displacement spectrum, $U_m(T_e, \zeta_e)$, and the peak ground displacement, $U_g^{\text{max}}$, in the following way:

$$R_\mu(\infty) = 1 + (\mu_e - 1) \left( \frac{U_m(T_e, \zeta_e)}{U_g^{\text{max}}} \right)^\alpha \quad (12)$$

where $\alpha \approx 0.5$. It is a simple matter to show that this expression has correct limits for very short and long periods of vibration. Contrarily to what happens with available reduction rules, the values given by equation (12) can be larger than $\mu_e$, which indeed occurs if $U_m > U_g^{\text{max}}$. 

[21]
In the conditions of Mexico City, this takes place when $T_e > 1 \text{ s}$. This reduction rule is more general than others reported in the literature, because the period and damping dependency of $R_\mu(\infty)$ is properly controlled by the actual shape of the elastic displacement spectrum, and not by a smoothed shape obtained empirically.

Following the replacement oscillator approach, this reduction rule may be readily implemented for elastically supported structures by replacing in equation (12) the relationships $\mu_e - 1 = (\bar{\mu}_e - 1) \tilde{T}_e^2 / T_e^2$, from equation (6), and $U_m = (T_e^2 / \tilde{T}_e^2) \bar{U}_m$, from equation (9) for $\mu_e = 1$, with which we have that

$$R_\mu(\beta_s) = 1 + (\bar{\mu}_e - 1) \frac{\tilde{T}_e}{T_e} \left( \frac{\bar{U}_m(\tilde{T}_e, \bar{\zeta}_e)}{U_g^{\max}} \right)^\alpha$$  \hspace{1cm} (13)

It should be pointed out that equation (12) will yield the same result as equation (13) if the elastic displacement spectrum without SSI is replaced by that with SSI. The two spectra $U_m(T_e, \zeta_e)$ and $\bar{U}_m(\tilde{T}_e, \bar{\zeta}_e)$ are used to emphasize the fact that the former corresponds to the actual structure, whereas the latter to the replacement oscillator. The steps involved in the application of equation (13) can be summarized as follows:

1. By use of equations (4) to (6), compute the modified period $\tilde{T}_e$, damping $\tilde{\zeta}_e$ and ductility $\bar{\mu}_e$ of the structure whose rigid-base properties $T_e$, $\zeta_e$ and $\mu_e$ are known.

2. From the prescribed site-specific response spectrum, determine the elastic spectral displacement $\bar{U}_m$ corresponding to $\tilde{T}_e$ and $\bar{\zeta}_e$, just as if the structure were fixed at the base.

3. The value of $R_\mu(\beta_s)$ is then estimated by application of equation (13), provided the peak ground displacement $U_g^{\max}$ is known.
Figure 11. Exact strength-reduction factors (solid line) versus approximate ones (dashed lines) for the SCT recording of the 1985 Michoacan earthquake and a structure with $\mu_e = 4$, $D/r = 0.5$ and $H_e/r = 3$.

Figure 11 shows the comparison between the exact strength-reduction factors depicted in figure 10 and those obtained by the approximate reduction rule. It is seen that, although the representation of equations (12) and (13) is not perfect, the proposed rule reproduces satisfactorily the tendencies observed in reality. In view of the many uncertainties involved in the definition of $R_\mu$, it is judged that this approximation is appropriate for design purposes.

**CODE DESIGN PROCEDURE**

There is still controversy regarding the role of SSI in the seismic performance of structures placed on soft soil. Maybe for that the SSI provisions in the Mexico City building code are not mandatory and, therefore, rarely used in practice. The code has been revised recently, including a new approach to specify site-specific design spectra as well as new SSI provisions to be applied together with these spectra. For interacting elastic systems, a design procedure has already been formulated by Veletsos (1977), which permits the use of standard fixed-base response spectra. With the information that has been presented, this procedure may be adjusted for interacting yielding systems. This issue is addressed now.
For arbitrary locations in the city, elastic design spectra of normalized pseudoacceleration are computed as follows:

\[ \frac{Sa}{g} = a_o + (\zeta' c - a_o) \frac{T}{T_a}; \quad \text{if } T < T_a \]  \hspace{1cm} (14)

\[ \frac{Sa}{g} = \zeta' c; \quad \text{if } T_a \leq T \leq T_b \]  \hspace{1cm} (15)

\[ \frac{Sa}{g} = \zeta' c \left( k + (1-k) \left( \frac{T_b}{T} \right)^2 \right) \left( \frac{T_b}{T} \right)^2; \quad \text{if } T > T_b \]  \hspace{1cm} (16)

in which:

\[ \zeta' = \left( \frac{0.05}{\zeta} \right)^{1/\lambda}; \quad \text{if } T \leq T_b \]  \hspace{1cm} (17)

\[ \zeta' = 1 + \left( \frac{0.05}{\zeta} \right)^{1/\lambda} - 1 \left( \frac{T_b}{T} \right); \quad \text{if } T > T_b \]  \hspace{1cm} (18)

is a scaling factor used to account for the supplemental damping due to SSI, where \( \lambda = 0.5 \) and 0.6 for the transition \( (T_s < 1 \text{ s}) \) and lakebed \( (T_s > 1 \text{ s}) \) zones of Mexico City, respectively. The spectral shape depends on five site parameters: \( a_o \), the peak ground acceleration; \( c \), the peak spectral acceleration; \( T_a \) and \( T_b \), the lower and upper periods of the flat part of the spectrum; and \( k \), the ratio between peak ground displacement and peak spectral displacement. Specific expressions are given in the code (Ordaz et al, 2004) to compute these parameters in terms of \( T_s \). These spectra are reduced by two separate factors that account for inelastic behavior and overstrength of the structure. The latter factor is independent of SSI.
and therefore ignored here. As a novelty, the descending branch of the spectrum was adjusted to have a better description of the spectral displacement. In fact, for long period, the spectral displacement tends to the peak ground displacement, a fact usually overlooked in most building codes worldwide.

When applying equations (14) to (18), the natural period $T$ and damping ratio $\zeta$ should take the following values: $T_e$ and $\zeta_e$, for the fixed-base case; and $\tilde{T}_e$ and $\tilde{\zeta}_e$, for SSI. The code specifies that $\tilde{\zeta}_e$ cannot be taken less than 0.05, the nominal damping implicit in the design spectrum. With this provision is recognized, at least implicitly, the additional damping by kinematic interaction. For the SCT site and a structure with $D/r = 0.5$ and $H_e/r = 3$, the resulting design spectra with and without regard to SSI can be appreciated in figure 12, along with the corresponding response spectra for the control motion. As it can be seen, the latter spectra are safely covered by the former in the whole period range. However, the conservative smoothing of the design spectra does not reflect some particular changes by SSI. Specifically, the response increase observed around 1-2 s cannot be reproduced, since the plateaus of the design spectra with and without SSI coincide in this region.

![Figure 12](image_url)

**Figure 12.** Elastic design spectra with (dashed line) and without (solid line) SSI for a soft site with $T_s = 2$ s and a structure with $D/r = 0.5$ and $H_e/r = 3$. The corresponding response spectra for the SCT recording of the 1985 Michoacan earthquake are also shown for reference.
The required base-shear coefficients \( \bar{C} \) and \( C \) with and without regard to SSI are computed in the following way:

\[
\bar{C} = \frac{S_a(T_e, \zeta_e) / g}{R_\mu(\beta_e)} \quad (19)
\]

\[
C = \frac{S_a(T_e, \zeta_e) / g}{R_\mu(\infty)} \quad (20)
\]

with \( R_\mu(\infty) \) and \( R_\mu(\beta_e) \) given by equations (12) and (13), respectively. The two coefficients \( C \) and \( \bar{C} \) are used to emphasize the fact that the former should be evaluated for \( T_e, \zeta_e \) and \( \mu_e \), whereas the latter for \( \bar{T}_e, \bar{\zeta}_e \) and \( \bar{\mu}_e \).

As it is common practice, the SSI effects are accounted for on the fundamental mode of vibration only. So, when applying the static analysis procedure, the base shear modified by SSI can be determined as follows:

\[
\tilde{V}_o = CW_o - (C - \bar{C})W_e \quad (21)
\]

where \( W_o \) is the total weight and \( W_e = M_e g \) the effective weight of the structure. This expression is similar to that used in the ATC and NEHRP documents, except that it incorporates the effects of SSI on the structural ductility, an important subject ignored thus far in current building codes. Dividing equation (21) by the fixed-base shear \( V_o = CW_o \) and taking \( W_e = 0.7W_o \), we have that

\[
\frac{\tilde{V}_o}{V_o} = 0.3 + 0.7 \frac{\bar{C}}{C} \quad (22)
\]
Note that this factor has the same meaning of $R_\beta$ given by equation (10). Figure 13 shows the variation of $\widetilde{\nu}_o/V_o$ with $T_e$ for the same data of figure 12, considering elastic ($\mu_e = 1$) and inelastic ($\mu_e = 4$) behavior. Results reveal that the significance of SSI depends primarily on the period ratio of the structure and site. It is seen that the increments for $T_e < T_s$ are less important than the reductions for $T_e > T_s$, and that both effects are more important for elastic than for inelastic systems. The code specifies that $\widetilde{\nu}_o/V_o$ cannot be taken less than 0.75, nor greater than 1.25. The adoption of these values is justified on empirical rather than on theoretical grounds.

Figure 13. Variations against period of the design interaction factor for a soft site with $T_s = 2$ s and a structure with $D/r = 0.5$ and $H_e/r = 3$, considering $\mu_e = 1$ (solid line) and 4 (dashed line).

The use of the recommended SSI provisions will increase or decrease the required strength with respect to the fixed-base value, depending on the relation existing between the structure and site periods. The lateral displacement will undergo additional changes due to the contribution of the foundation rotation. The maximum displacement of the structure relative to the ground is determined from the expression
\[
\tilde{U}_m = \frac{\tilde{V}_o}{K_o} \mu_e + \frac{\tilde{V}_o (H_e + D)^2}{K_r} = \frac{\tilde{V}_o}{V_o} \left( U_m + (H_e + D) \frac{M_o}{K_r} \right)
\]  

(23)

where \( U_m = (V_o / K_o) \mu_e \) is the maximum deformation of the fixed-base structure and \( M_o = V_o (H_e + D) \) the corresponding overturning moment at the base. The peak displacements considering SSI are compared with those assuming the base as fixed in figure 14, using the values of \( \tilde{V}_o / V_o \) given in figure 13. The solid lines, which refer to the fixed-base structure, represent the effect of the structural deformation only, whereas the dashed lines, which refer to the interacting system, represent the combined effects of the structural deformation and the foundation rotation. As it can be seen, computation of \( \tilde{V}_o / V_o \) allows determination of the effects of SSI on both the base shear and the lateral displacement. Furthermore, this factor should be used to modify any response quantity computed as if the structure were fixed at the base in order to include SSI.

Figure 14. Lateral displacement considering SSI (dashed line) versus structural deformation assuming the base as fixed (solid line), for a soft site with \( T_s = 2 \) s and a structure with \( D / r = 0.5 \) and \( H_e / r = 3 \).

When applying the modal analysis procedure, the base shear associated to the first mode, \( V_1 = CW_1 \), may be modified by SSI as \( \tilde{V}_1 = \tilde{C}W_1 \), in which \( W_1 = W_e \). The contribution of the higher modes and the combination of the modal responses are performed as for structures without SSI.
CONCLUDING REMARKS

The concepts presented herein can be used to account for the effects of SSI in the seismic design of yielding structures. The strength and displacement demands are well predicted by the simplified procedure outlined, which provides a convenient extension to the well-known replacement oscillator approach. More involved procedures are justified only for unusual buildings of major importance in which the SSI effects are of definite consequence in design. Although given only for a specific site, results for other soft sites in Mexico City lead essentially to the same conclusions. Despite the simplicity of the SSI model investigated, it forms the basis of the current design practice, so the conclusions drawn from this study may also be applicable to more complex interacting systems as well. Some considerations were made aimed to devise more rational code SSI provisions. There continues to be a need for additional research on the multi-degree-of-freedom effects and the uncertainties involved in real buildings. Caution should be taken when using this information for pile-supported structures, since the pile effects decrease the system period and increase the system damping.

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Seismic Design of a Structure Supported on Pile Foundation Considering Dynamic Soil-Structure Interaction

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It is necessary to predict precisely the structure response considering soil-structure interaction for implementation of performance-based design. Soil-structure interaction during earthquake, however, is very complicated and is not always taken into account in seismic design of structure. Especially pile foundation response becomes very complicated because of nonlinear interactions between piles and liquefied soil. In this paper pile foundation responses are clarified by experimental studies using ground motions induced by large-scale mining blasts and nonlinear analyses of soil-pile foundation-superstructure system.

INTRODUCTION

Vibration tests using ground motions induced by large-scale mining blasts were performed in order to understand nonlinear dynamic responses of pile-structure systems in liquefied sand deposits. Significant aspects of this test method are that vibration tests of large-scale structures can be performed considering three-dimensional soil-structure interaction, and that vibration tests can be performed several times with different levels of input motions because the blast areas move closer to the test structure. This paper describes the vibration tests and the simulation analyses using numerical model of nonlinear soil-pile foundation-superstructure system (Kamijho 2001, Kontani 2001, Saito 2002(a), 2002(b)).

VIBRATION TEST USING GROUND MOTIONS INDUCED BY MINING BLASTS

The vibration test method using ground motions induced by mining blasts is shown schematically in Figure 1. Vibration tests on a pile-supported structure in a liquefiable sand deposit were conducted at Black Thunder Mine of Arch Coal, Inc. Black Thunder Mine is

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one of the largest coalmines in North America and is located in northeast Wyoming, USA. At the mine, there is an overburden (mudstone layers) over the coal layers. The overburden is dislodged by large blasts called "Cast Blasts". The ground motions induced by Cast Blasts were used for vibration tests conducted in this research.

OUTLINES OF VIBRATION TESTS

A sectional view and a top view of the test pit and the pile-supported structure are shown in Figure 2 and Figure 3, respectively. A 12x12-meter-square test pit was excavated 3 meters deep with a 45-degree slope, as shown in Figure 2. A waterproofing layer was made of high-density plastic sheets and was installed in the test pit in order to maintain 100% water-saturated sand.

Outlines of the pile-supported structure are shown in Figure 4. Four piles were made of steel tube. Pile tips were closed by welding. Piles were embedded 70cm into the mudstone layer. The top slab and the base mat were made of reinforced concrete and were connected by H-shaped steel columns. The structure was designed to remain elastic under the

![Figure 1: Vibration test method at mining site](image)

![Figure 2: Sectional view of test pit](image)

![Figure 3: Top view of test pit](image)
conceivable maximum input motions, and the main direction for the structure is set in the EW direction. The construction schedule was determined so that the structure under construction received the least influence from mining blasts.

Instrumentation is shown in Figure 5. Accelerations were measured of the structure and one of the four piles. Accelerations in the sand deposit and free field adjacent to the pit were also measured in array configurations. Axial strains of the pile were measured to evaluate bending moments. Excess pore water pressures were measured at four levels in the test pit to investigate liquefaction phenomena.

PS measurements were conducted at the test site to investigate the physical properties of the soil layers. The shear wave velocity at the test pit bottom was about 200 m/s and this increased to 500 to 700 m/s with increasing depth. Core soil samples were collected for laboratory tests. The backfill sand was found near Black Thunder Mine. Great care was taken in backfilling the test pit with the sand, because the sand needed to be 100% water-saturated and air had to be removed in order to ensure a liquefiable sand deposit.

Figure 6 shows the completed pile-supported structure and the test pit. The water level was kept at 10 cm above the

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**Figure 4.** Pile-supported structure

**Figure 5.** Instrumentation

**Figure 6.** Test structure
sand surface throughout seismic tests to prevent dry out of the sand deposit.

VIBRATION TEST RESULTS

Vibration tests were conducted six times. The locations of the blast areas for each test are shown in Figure 7. The blast areas were about 60m wide and 500m long. The results of the vibration tests are summarized in Table 1. The maximum horizontal acceleration recorded on the adjacent ground surface varied from 20 Gals to 1,352 Gals depending on the distance from the blast area to the test site. The closest blast was only 90m from the test site. These differences in maximum acceleration yielded responses at different levels and liquefaction of

Table 1. Summary of vibration tests

<table>
<thead>
<tr>
<th>Level of Input Motions</th>
<th>Test #</th>
<th>Distance (m) *</th>
<th>Max. Acceleration **</th>
</tr>
</thead>
<tbody>
<tr>
<td>Small</td>
<td>Test-1</td>
<td>3000</td>
<td>20 28 29</td>
</tr>
<tr>
<td></td>
<td>Test-2</td>
<td>1000</td>
<td>32 84 48</td>
</tr>
<tr>
<td>Medium</td>
<td>Test-5</td>
<td>500</td>
<td>142 245 304</td>
</tr>
<tr>
<td>Large</td>
<td>Test-3</td>
<td>140</td>
<td>579 568 1013</td>
</tr>
<tr>
<td></td>
<td>Test-4</td>
<td>180</td>
<td>564 593 332</td>
</tr>
<tr>
<td>Very Large</td>
<td>Test-6</td>
<td>90</td>
<td>1217 1352 3475</td>
</tr>
</tbody>
</table>

*: distance from blast area to test site
**: at the ground level of adjacent free field (Gals)

Figure 7. Locations of blasts in vibration tests

Figure 8. Max. acceleration at free field

Figure 9. Max. acceleration of test pit
different degrees. Sand boiling phenomena were observed in the test pit with larger input motions.

In this paper, three tests (Test-1, 5, 3) indicated in Table 1 were chosen for detailed investigations, because those tests provided three different phenomena in terms of liquefaction of the sand deposit as well as in terms of dynamic responses of the structure. Horizontal accelerations in the EW direction are discussed hereafter.

**DYNAMIC RESPONSES IN LIQUEFIED SAND DEPOSITS**

The maximum accelerations recorded in the adjacent free field in vertical arrays are compared for three tests in Figure 8. The amplification tendencies from GL-32m to the surface were similar in the mudstone layers for three tests. The maximum accelerations recorded through the mudstone layers to the sand deposit are compared for these three tests in Figure 9. There was a clear difference among the amplification trends in the test pit. Test-1 showed a similar amplification trend to that of the mudstone layers as shown in Figure 8. Test-5 showed less amplification in the sand deposit. Test-3 showed a large decrease in

![Figure 8](image1.png)

![Figure 9](image2.png)

**Figure 10.** Acceleration records of Test-1 (Small Input Level)

**Figure 11.** Acceleration records of Test-5 (Medium Input Level)
acceleration in the test pit because of severe liquefaction of the sand deposit.

Acceleration time histories at the sand surface, the free field surface and GL-32m are compared for Test-1 (Small Input Level) in Figure 10. The response spectra from these records are also shown in the figure. The same set of acceleration time histories and these response spectra are shown in Figure 11 for Test-5 (Medium Input Level) and in Figure 12 for Test-3 (Large Input Level).

As can be seen from Figure 10 for Test-1, over all the frequency regions, the responses at the sand surface were greater than those at the free field surface, and the responses at the free field surface were greater than those at GL-32m. From Figure 11 for Test-5, the responses at the sand surface and the free field surface were greater than those at GL-32m over all frequency regions. The responses at the sand surface became smaller than those at the free field surface for periods of less than 0.4 seconds due to in a certain degree of liquefaction of the sand. From Figure 12 for Test-3, the responses at the sand surface became much smaller than those at the free field surface and even smaller than those at GL-32m. These response reductions in the test pit were caused by extensive liquefaction over the test pit, because shear

**Figure 12.** Acceleration records of Test-3 (Large Input Level)

**Figure 13.** Measured time histories of excess pore water pressure ratio
waves could not travel in the liquefied sand.

Time histories of excess pore water pressure ratios are shown in Figure 13. The excess pore water pressure ratio is the ratio of excess pore water pressure to initial effective stress. In Test-1, the maximum ratio stayed around zero, which means that no liquefaction took place. In Test-5, the ratios rose rapidly, reaching around one at GL-0.6m and GL-1.4m after the main vibration was finished. Ratios at GL-2.2m and GL-3.0m were about 0.7 and 0.5. The measurement showed that the liquefaction region was in the upper half of the test pit. In Test-3, ratios at all levels rose rapidly, reaching around one, which indicates extensive liquefaction over the entire region. The large fluctuations in pressure records during main ground motions were caused by longitudinal waves.

**Structure Responses Subjected to Blasts-Induced Ground Motion**

Figure 14 compares the acceleration time histories at the top slab, the base mat and GL-3m of the pile for Test-1 (Small Input Level). The response spectra from these records are also shown. The same set of acceleration time histories and their response spectra are shown in Figure 15 for Test-3 (Large Input Level).

![Figure 14. Acceleration records of test structure (Test-1 : Small Input Level)](image1)

![Figure 15. Acceleration records of test structure (Test-3 : Large Input Level)](image2)
As can be seen from Figure 14 for Test-1, the maximum accelerations increased as motions went upward. For all frequency regions, the responses at the top slab were greater than those at the base mat, and the responses at the base mat were greater than those at GL-3m of the pile. The first natural period of the soil-pile-structure system was about 0.2 seconds under the input motion level of Test-1. For Test-3, the maximum accelerations decreased as motions went upward, which were different from those of Test-1. The responses at the top slab and the base mat became smaller than or similar to the responses at GL-3m of the pile. Compared with Test-1 results, it became difficult to identify peaks corresponding to natural periods of the soil-pile-structure system from response spectra diagrams. These results show that soil nonlinearity and liquefaction greatly influence the dynamic properties of pile-supported structures.

**Measurement Results of Pile Stresses**

The distributions of maximum pile stresses, bending moments and axial forces, are shown in Figures 16 and 17. The bending moment took its maximum value at the pile head for all cases. However, the moment distribution shapes differed and the inflection points of the curves moved downward in accordance with the input motion levels, in other words, the degrees of liquefaction in the test pit. However, the axial forces are almost the same regardless of the depth and similar tendencies are shown in all the test results.
ANALYSIS RESULTS

Figure 18 shows the analysis model for 3-D response of soil-pile-structure system. The soil response analysis is conducted by a 3D-FEM effective stress analysis method. The analyses were performed by a step-by-step integration method and employed a multiple shear mechanism model for the strain dependency of soil stiffness and Iai-Towhata model for evaluating the generation of excess pore water pressure (Iai 1992). Table 2 shows the soil constants. The shear wave velocity was measured by PS-Logging and the density of the saturated sand was measured by a cone penetration test. Soil nonlinearity was taken into account for all layers and Table 3 shows the nonlinear parameter for this simulation analysis. Figures 19 and 20 show the nonlinear properties and the liquefaction curve for the reclaimed sand, respectively. These curves are based on laboratory tests.

The super-structure is idealized by a one-stick model and the pile foundations are idealized by a four-stick model with lumped masses and beam elements. The lumped masses of the pile foundations are connected to the free field soil through lateral and shear interaction springs. A nonlinear vertical spring related to the stiffness of the supported layer is also incorporated at the pile tip, as shown in Figure 21. The initial values of the lateral and shear interaction soil springs of the pile groups are obtained using Green’s functions by ring loads in a layered stratum and they are equalized to four pile foundations. The soil springs are modified in accordance with the relative displacements between soils and pile foundations and with the generation of excess pore water pressures (Miyamoto 1995).

3-D Responses of Liquefied Sand Deposits

Figure 22 shows the calculated time histories of the ground surface accelerations and the pore water pressure ratios. The amplitudes of the horizontal motions became smaller due to the generation of pore water pressure at time 2.5 seconds. However, the amplitude of the vertical motion was still large after 2.5 seconds. The analysis results are in good agreement with the test results.

Figure 23 shows the acceleration response spectrum of the ground surface in the EW direction. The blue line and the red line show the 3-D and 1-D analysis results respectively, and green line show the test results. All spectra have a first peak at 0.6 seconds, and the 3-D results are good agreement with the test result. Figure 24 shows the acceleration response spectrum of the ground surface in the UD direction. All spectra have a first peak at 0.3
seconds, and both of the 3-D and 1-D analysis results are in good agreement with the test result.

**Figure 18.** Three-dimensional analysis model for soil-pile-structure system

**Table 2.** Soil properties for simulation analysis

<table>
<thead>
<tr>
<th>Soil type</th>
<th>Thickness (m)</th>
<th>Unit Weight (kN/m³)</th>
<th>Vs (m/s)</th>
<th>Vp (m/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sand (Test Bed)</td>
<td>3.0</td>
<td>18.9</td>
<td>80</td>
<td>1530</td>
</tr>
<tr>
<td>Clay</td>
<td>2.0</td>
<td>16.7</td>
<td>200</td>
<td>400</td>
</tr>
<tr>
<td>Mudstone</td>
<td>5.0</td>
<td>18.6</td>
<td>320</td>
<td>1240</td>
</tr>
</tbody>
</table>

**Table 3.** Nonlinear parameters

<table>
<thead>
<tr>
<th>Reclaimed Sand</th>
<th>Reference Strain : 0.034%  (G/G₀=0.5)</th>
<th>Maximum Damping Factor : 28%</th>
</tr>
</thead>
<tbody>
<tr>
<td>Liquefaction Parameter</td>
<td>W₁=1.15, S₁=0.005, P₁=0.5,</td>
<td></td>
</tr>
<tr>
<td></td>
<td>P₂=1.12, C₁=1.6,</td>
<td></td>
</tr>
<tr>
<td>Phase Transformation Angle=28deg.</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Clay and Mud Stone</th>
<th>Reference Strain : 0.17%  (G/G₀=0.5)</th>
<th>Maximum Damping Factor : 25%</th>
</tr>
</thead>
</table>

**Figure 19.** Nonlinear properties of reclaimed sand

**Figure 20.** Liquefaction curve

**Figure 21.** Relationship between vertical displacements and axial forces at pile head
Dynamic Responses for Test Structure

Figure 25 compares the calculated time-histories of acceleration for the test structure with the test results. The horizontal motions for the top slab of the test structure have almost the same amplitudes in the EW and NS directions, and are different from the records for ground surface shown in Figure 22. However, the vertical motion for the base mat of the test structure is almost the same as that for the ground surface shown in Figure 22. The analysis results are in good agreement with the test results not only in the horizontal directions but also in the vertical direction.

Figure 26 shows the displacement orbit in the EW and NS directions for the top slab and the ground surface. The horizontal motions of the ground surface had an almost circular orbit. On the other hand, the top slab had an elliptical orbit and amplitudes for the EW direction became larger than those for the NS direction due to the different vibration property of the test structure. The analysis results are in good agreement with the test results, and it is confirmed that this analysis method is applicable to evaluate the 3-D responses of pile-supported structures in liquefied sand deposits.
Bending Moments and Axial Forces for Pile Foundation

The distributions of maximum pile stresses, bending moments and axial forces, are shown in Figure 27. Bending moments became larger at the pile head as well as at the interface between the reclaimed sand and the supporting layer. The calculated maximum bending moments at pile heads are almost the same in the EW and NS directions, since the maximum acceleration of the superstructure were almost the same in both directions. The calculated
maximum axial forces in the four piles are almost the same and about 90kN. The 1-D analysis result became smaller than the 3-D analysis results.

The time histories of the pile stresses at pile heads are shown in Figure 28. The analysis results are in good agreement with the test results, which indicates that this analysis method is applicable to evaluate pile stresses during liquefaction. The maximum bending moments occurred at 2.9 seconds in the EW direction and at 2.0 seconds in the NS direction. These times correspond closely with the superstructure responses, as shown in Figure 25. The time history of axial force at the pile head is similar with that of the bending moment in the EW direction, and it is different with that of the superstructure response in the UD direction shown in Figure 25.

**CONCLUSIONS**

Vibration tests were conducted of a pile-supported structure in a liquefiable sand deposit using ground motions induced by large mining blasts. Nonlinear responses of the soil-pile-structure system were obtained for various levels of liquefaction in the test pit. The vibration test method employed in this research was found to be very useful and effective for investigating the dynamic behavior of large model structures under severe ground motions. Simulation analysis results were in good agreement with the test results for the responses of the superstructure and pile stresses due to liquefaction. To evaluate the performance of pile foundation it is important to precisely predict pile response using nonlinear soil-pile foundation-superstructure system.

**ACKNOWLEDGMENTS**

We would like to express our deep appreciation to Dr. Aoyama, Professor Emeritus of the University of Tokyo, for his guidance throughout this experimental research. We would also like to thank Professor Robert Nigbor of University of Southern California, and the management and staff of Arch Coal's Black Thunder Mine for their assistance throughout the project.
REFERENCES


Implementation of Soil-Structure Interaction Models in Performance Based Design Procedures

Jonathan P. Stewart, a) Craig Comartin, b) and Jack P. Moehle c)

A soon to be published guidelines document for the design of seismic retrofits for existing buildings is based on performance-based design principles as implemented through so-called nonlinear static procedures (NSPs). In these procedures, the global inelastic deformation demand on the structure is computed from the response of an equivalent nonlinear single-degree-of-freedom (SDOF) system, the response of which is estimated from that of an elastic SDOF system. The guidelines were developed as part of the ATC-55 project, which is summarized by Comartin (this conference). The objective of the present paper is to describe one component of the ATC-55 project related to the implementation of soil-structure interaction (SSI) principles into NSPs. SSI effects are most important at short periods (i.e., $T$ less than approximately 0.5 s). Three SSI phenomena can contribute to NSPs. First, flexibility at the soil-foundation interface can be incorporated into nonlinear pushover curves for the structure. These foundation spring models were incorporated into NSPs that pre-existed the ATC-55 project, and are not emphasized here. Second, SSI affects demand spectra through the effective system damping, which is the damping ratio for which spectral ordinates should be calculated. Third, kinematic SSI reduces ordinates of the demand spectra. This paper describes how damping and kinematic SSI effects have been incorporated into the recommended seismic analysis procedures for existing buildings.

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1.0 INTRODUCTION

In this paper we present simplified procedures for including the effects of interaction between a structure and the supporting soils in nonlinear inelastic seismic analyses. The procedures described here were developed as part of the ATC-55 project and will be formally presented in FEMA-440 (2004).

There are three primary categories of soil-structure interaction (SSI) effects. These include: introduction of flexibility to the soil-foundation system with resulting lengthening of the system’s fundamental response period (flexible foundation effects); filtering of the character of ground shaking transmitted to the structure (kinematic effects); and dissipation of energy from the soil-structure system through radiation and hysteretic soil damping (foundation damping effects). Current analysis procedures in FEMA 356 (2000) and ATC 40 (1996) partially address the flexible foundation effect in guidelines for including the stiffness and strength of the geotechnical components of the foundation in the structural analysis model. However, those procedures do not address reduction of the shaking demand on the structure relative to the free field motion due to kinematic interaction or the foundation damping effect. Guidelines on including those effects in nonlinear inelastic analyses were introduced in FEMA-440 and are summarized here. More detailed information can be found in Appendix 8 of FEMA-440 (2004).

2.0 KINEMATIC INTERACTION EFFECTS

Kinematic interaction results from the presence of stiff foundation elements on or in soil, which causes foundation motions to deviate from free-field motions as a result of base slab averaging and embedment effects. The base slab averaging effect can be visualized by recognizing that the motion that would have occurred in the absence of the structure within and below the footprint of the building is spatially variable. Placement of a foundation slab across these variable motions produces an averaging effect in which the foundation motion is less than the localized maxima that would have occurred in the free-field. The embedment effect is simply associated with the reduction of ground motion that tends to occur with depth in a soil deposit.

This section covers simple models for the analysis of ground motion variations between the free-field and the foundation-level of structures. In general, these models must account for base slab averaging and embedment effects. Kinematic interaction for pile-supported
foundations is not covered. Theoretical models for kinematic interaction effects are expressed as frequency-dependent ratios of the Fourier amplitudes (i.e., transfer functions) of foundation input motion (FIM) to free-field motion. The FIM is the theoretical motion of the base slab if the foundation and structure had no mass, and is a more appropriate motion for structural response analysis than is the free-field motion.

In the following sections, formulations for transfer functions that account for base slab averaging and embedment effects are presented. Recommendations are then provided regarding how transfer functions can be used to modify a free-field response spectrum to estimate the FIM spectrum for use in nonlinear static procedures.

2.1 SHALLOW FOUNDATIONS AT THE GROUND SURFACE

Base-slab averaging results from inclined or incoherent incident wave fields. In the presence of those wave fields, translational base-slab motions are reduced relative to the free-field (rotational motions are also introduced, but are not considered here). The reductions of base-slab translation tend to become more significant with decreasing period. The period-dependence of these effects is primarily associated with the increased effective size of the foundation relative to the seismic wavelengths at low periods. In addition, ground motions are more incoherent at low periods.

Veletsos and co-workers (1989, 1997) developed useful models for theoretical base slab averaging that combine an analytical representation of the spatial variation of ground motion with rigorous treatment of foundation-soil contact. The transfer function amplitudes computed by the Veletsos group are presented in Figure 1 for circular and rectangular foundations subject to vertically incident, incoherent shear waves. Similar curves are available for other wave fields. The transfer functions in Figure 1 are plotted against the dimensionless frequency parameter \( \tilde{a}_0 \), defined as follows for circular and rectangular foundations subject to vertically incident waves, respectively,

\[
\tilde{a}_0 = \kappa a_0 \quad \text{(circular)}; \quad \tilde{a}_0 = \frac{ab_\kappa}{2V_{s,r}} \quad \text{(rectangular),}
\]

where \( a_0 = \omega r/V_{s,r}, \) \( V_{s,r} = \) strain-reduced shear wave velocity, \( r = \) radius of circular foundation, \( b = \sqrt{ab} , a \times b = \) full footprint dimensions of rectangular foundation, and \( \kappa = \) a ground motion incoherence parameter.
Figure 1. Amplitude of transfer function between free-field motion and FIM for vertically incident incoherent waves. Modified from Veletsos and Prasad (1989) and Veletsos et al. (1997).

Kim and Stewart (2003) calibrated Veletsos’ analysis procedure against observed foundation / free-field ground motion variations as quantified by frequency-dependent transmissibility function amplitudes, $|H|$. Veletsos’ models were fit to $|H|$ and apparent $\kappa$-values (denoted $\kappa_a$) were fit to the data. Those $\kappa_a$ values reflect not only incoherence effects, but necessarily also include average foundation flexibility and wave inclination effects for the calibration data set. The structures in the calibration data set generally have shallow foundations that are inter-connected (i.e., continuous mats or footings inter-connected with grade beams). Parameter $\kappa_a$ was found to be correlated to average soil shear wave velocity approximately as follows:

$$\kappa_a = -0.037 + 0.00074V_s \text{ or } \kappa_a \approx 0.00065V_s$$

(2)

where $V_s$ = small strain shear wave velocity in m/s. The fact that $\kappa_a$ is nearly proportional to $V_s$ (Eq. 2) causes dimensionless frequency term $\tilde{a}_0$ to effectively reduce to a function of frequency and foundation size ($b_c$). This is shown by the following, which is written for vertically propagating waves ($\alpha_v = 0$):

$$\tilde{a}_0 = \frac{\omega b_c \kappa}{2V_{s,r}} \approx \frac{\omega b_c n_1 V_s}{2n_2 V_s} = \frac{\omega b_c n_1}{2n_2}$$

(3)
where \( n_1 \approx 6.5 \times 10^{-4} \text{ s/m} \) and \( n_2 \) is the square root of the soil modulus reduction factor, which can be estimated as shown in Table 1 (BSSC, 2001). In the remainder of this paper, \( n_2 \) will be taken as 0.65, which is the appropriate value for regions of high seismicity such as coastal California.

<table>
<thead>
<tr>
<th>Peak Ground Acceleration (PGA)</th>
<th>0.10g</th>
<th>0.15g</th>
<th>0.20g</th>
<th>0.30g</th>
</tr>
</thead>
<tbody>
<tr>
<td>( n_2 )</td>
<td>0.90</td>
<td>0.80</td>
<td>0.70</td>
<td>0.65</td>
</tr>
</tbody>
</table>

Limitations of the model calibration by Kim and Stewart (2003), and hence the present approach, include: (1) foundations should have large in-plane stiffness, ideally a continuous mat foundation or interconnected footings/grade beams; (2) for non-embedded foundations, the foundation dimension should be less than 60 m unless the foundation elements are unusually stiff; (3) the approach should not be used for embedded foundations with \( e/r > 0.5 \); and (4) the approach should not be used for pile-supported structures in which the cap and soil are not in contact.

### 2.2 EMBEDDED SHALLOW FOUNDATIONS

Foundation “embedment” refers to a foundation base slab that is positioned at a lower elevation than the surrounding ground, which will usually occur when buildings have a basement. When subjected to vertically propagating coherent shear waves, embedded foundations experience a reduction in base-slab translational motions relative to the free-field.

Elsabee and Morray (1977) and Day (1978) developed analytical transfer functions relating base-slab translational motions to free-field translations for an incident wave field consisting of vertically propagating, coherent shear waves. Base-slab averaging does not occur within this wave field, but foundation translations are reduced relative to the free-field due to ground motion reductions with depth and wave scattering effects. Day’s (1978) analyses were for a uniform elastic half space, while Elsabee and Morray’s (1977) were for a finite soil layer. Results for both are shown together in Figure 3a for foundation embedment / radius ratio \( e/r = 1.0 \). The primary difference between the two solutions is oscillations in the
finite soil layer case at high frequencies. Also shown in Figure 3a is the following approximate transfer function amplitude model developed by Elsabee and Morray (1977):

\[ |H_u(\omega)| = \cos \left( \frac{e}{r} a_0 \right) = \cos \left( \frac{e \omega}{V_s} \right) \geq 0.454 \] (4)

where \( a_0 = \omega r/V_s \) and \( e \) = foundation embedment. Figure 3b shows the transfer function amplitude model is a somewhat more convenient form in which it is plotted as a unique function of \( e \omega/V_s \) (i.e., in this form there is no dependence on foundation radius).

![Figure 3a](image1.png) ![Figure 3b](image2.png)

**Figure 3a.** Transfer function amplitudes for embedded cylinders from Day (1978) and Elsabee and Morray (1977) along with approximation

**Figure 3b.** Transfer function amplitude model by Elsabee and Morray (1977)

The results in Figure 3 can be contrasted with the behavior of a surface foundation, which would have no reduction of translational motions when subjected to vertically incident coherent shear waves. Transfer function amplitudes in the presence of more realistic incident wave fields can be estimated at each frequency by the product of the transfer function ordinates from Section 2.1 (for base slab averaging) and those from this section at the corresponding frequency.

The analysis procedure described herein has been verified against recorded motions from two relatively deeply embedded structures with circular foundations (Kim, 2001).
2.3 APPLICATION OF TRANSFER FUNCTIONS TO CALCULATION OF SPECTRAL ORDINATES OF FOUNDATION MOTIONS

Design-basis free-field motions are generally specified in terms of acceleration response spectra. The question addressed in this section is how this spectrum should be modified once the transfer function amplitude for the site has been evaluated using the analysis procedures described above.

When free-field motions are specified only as response spectral ordinates, the evaluation of a modified response spectrum consistent with the FIM is needed. Veletsos and Prasad (1989) evaluated ratios of foundation / free-field response spectral ordinates (at 2% damping) for conditions where the corresponding transfer function ordinates could be readily determined. The transfer function ordinates and ratios of response spectra (RRS) were compared for an input motion with specified power spectrum and random phase. The results indicated that transfer function ordinates provide a reasonable estimate of response spectral ratios for low frequencies (e.g., < 5 Hz), but that at high frequencies (≥ 10 Hz), transfer function ordinates are significantly smaller than response spectrum ratios. The inconsistency at high frequencies is attributed to the low energy content of free-field excitation at high frequencies and the saturation of spectral ordinates at these frequencies.

The analytical results of Veletsos and Prasad (1989) were checked by (1) calculating the transfer function for a fixed set of conditions (surface foundation, \( r = 50 \text{ m}, \ V_s = 250 \text{ m/s} \)), (2) using this transfer function to modify a set of recorded free-field time histories to corresponding foundation-level time histories, and (3) evaluating the RRS using the two time histories. The results are presented in Appendix 8 of FEMA-440 (2004), and suggest that for ordinary ground motions that Veletsos’s results summarized above are reasonable. However, it appears that some caution should be exercised for long-period ground motions such as those encountered on soft soil sites or for near-fault ground motions in the forward directivity region.

2.4 RECOMMENDED PROCEDURE

Based on the above, the following simplified procedure is recommended for analysis of kinematic interaction effects:
Step 1: Evaluate effective foundation size \( b_e = \sqrt{ab} \), where \( a \) and \( b \) are the footprint dimensions (in feet) of the building foundation in plan view.

Step 2: Evaluate an RRS from base slab averaging \( (RRS_{bsa}) \) at the period of interest using Figure 4. The period that should be used in Figure 4 is the effective period of the foundation-structure system accounting for any lengthening due to foundation flexibility or structural yielding effects (denoted \( T_{eq} \)). An approximate equation to the curves in Figure 4 is presented below:

\[
RRS_{bsa} = 1 - \frac{1}{14100} \left( \frac{b_e}{T_{eq}} \right)^{1.2} \tag{5}
\]

Step 3: If the foundation is embedded a depth \( e \) from the ground surface, evaluate an additional RRS from embedment \( (RRS_e) \) at the period of interest using Figure 5. The period that should be used is the same as in Step 2. The equation of the curves in Figure 5 is,

\[
RRS_e = \cos \left( \frac{2\pi e}{T_{eq} V_{s,r}} \right) \leq \cos \left( \frac{10\pi e}{V_{s,r}} \right) \geq 0.454 \tag{6}
\]

where \( e \) = foundation embedment and \( V_{s,r} \) = effective strain-degraded shear wave velocity in the soil. Factors that can be used to estimate \( V_{s,r} \) from small-strain shear wave velocity \( V_s \) are given in Table 1.

---

**Figure 4.** \( RRS_{bsa} \) from simplified model as function of foundation size, \( b_e \)

**Figure 5.** \( RRS_e \) for foundations with variable depths in NEHRP Site Classes C and D
Step 4: Evaluate the product of $RRs_{bsa}$ and $RRs_e$ to obtain the total RRS for the period of interest. The spectral ordinate of the foundation input motion at the period of interest is the product of the free field spectral ordinate and the total RRS.

Step 5: Repeat Steps 2 through 4 for other periods if desired to generate a complete spectrum for the foundation input motion.

3.0 FOUNDATION DAMPING

3.1 OVERVIEW

Inertia developed in a vibrating structure gives rise to base shear and moment at the foundation-soil interface, and these loads in turn cause displacements and rotations of the structure relative to the free field. These relative displacements and rotations are only possible because of compliance in the soil, which can significantly contribute to the overall structural flexibility. Moreover, the difference between the foundation input motion and free field motion gives rise to energy dissipation via radiation damping and hysteretic soil damping, and this energy dissipation affects the overall system damping. Since these effects are rooted in the structural inertia, they are referred to as inertial interaction effects, in contrast to the kinematic interaction effects discussed in Section 2.0.

Previous design documents (FEMA-356, 2000; ATC-40, 1996) contain provisions for evaluating the properties of foundation springs (e.g., Sections 10-3 and 10.4 of ATC-40), and hence this aspect of inertial interaction is not emphasized here. Rather, the ATC-55 project examined the damping component of inertial interaction and the contribution of this damping to the overall system damping.

In the SSI literature, foundation stiffness and damping are often described in terms of an impedance function. The impedance function should account for the soil stratigraphy and foundation stiffness and geometry, and is typically computed using equivalent-linear soil properties appropriate for the in situ dynamic shear strains. Impedance functions can be evaluated for multiple independent foundation elements, or (more commonly) a single 6×6 matrix of impedance functions is used to represent the complete foundation (which assumes foundation rigidity).

A detailed discussion of impedance functions is presented in Appendix 8 of FEMA-440 (2004). In simple terms, impedance functions can be thought of as springs and dashpots at the
base of the foundation that accommodate translational and rotational deformations relative to
the free-field. The coefficients that describe those springs and dashpots are frequency-
dependent. At zero frequency (i.e., static loading), the springs stiffnesses are described by:

\[ K_u = \frac{8}{2-\nu} G_{\text{max}} r_u, \quad K_\theta = \frac{8}{3(1-\nu)} G_{\text{max}} r_\theta^3 \]  

(7)

where subscript 'u' denotes translation and 'θ' denotes rotation in the vertical plane
(sometimes referred to as rocking); \( G_{\text{max}} \) = small-strain soil shear modulus (can be calculated
from \( V_s \) as \( G_{\text{max}} = V_s^2 \rho \), where \( \rho \) = mass density); \( \nu \) = Poisson’s ratio of soil; and radius terms
\( r_u \) and \( r_\theta \) are based on the area and moment of inertia, respectively, of the foundation as
follows:

\[ r_u = \sqrt{A_f / \pi}, \quad r_\theta = \sqrt{4I_f / \pi} \]  

(8)

where \( A_f \) = foundation area and \( I_f \) = foundation moment of inertia. The dashpot coefficients
that describe damping associated with translational and rotational vibrations (\( c_u \) and \( c_\theta \),
respectively) are given by:

\[ c_u = \beta_u \frac{K_u r_u}{V_s}, \quad c_\theta = \beta_\theta \frac{K_\theta r_\theta}{V_s} \]  

(9)

where \( \beta_u \) and \( \beta_\theta \) are functions of frequency as shown in Figure 6. The curves in Figure 6
apply for a uniform soil medium of infinite depth (i.e., a halfspace) and a rigid, circular
foundation at the ground surface.

Figure 6. Foundation stiffness and damping factors for elastic halfspace (dotted line) and viscoelastic
halfspace with 10% hysteretic soil damping. Poisson’s ratio \( \nu = 0.4 \). After Veletsos and Verbic (1973)
The combined effects of translational and rotational dashpots are often expressed by a foundation damping term $\beta_f$. The effects of foundation damping, in turn, on the response of a structure are represented by a modified damping ratio for the overall structural system. The initial damping ratio for the structure neglecting foundation damping is referred to as $\beta_i$, and is generally taken as 5%. The damping ratio of the complete structural system, accounting for foundation-soil interaction, as well as structural damping, is referred to as $\beta_0$. The change in damping ratio from $\beta_i$ to $\beta_0$ modifies the elastic response spectrum. The spectral ordinates are reduced if $\beta_0 > \beta_i$.

The calculation of quantity $\beta_0$ is the objective of a foundation damping analysis. This quantity can be calculated from $\beta_i$ and $\beta_f$ using the following expression, which is modified from Jennings and Bielak (1973), Bielak (1975, 1976), and Veletsos and Nair (1975):

$$
\beta_0 = \beta_f + \frac{\beta_i}{(T_{eq}/T_{eq})^3}
$$

where $T_{eq}/T_{eq}$ represents the period lengthening ratio of the structure in its degraded state (i.e., including the effects of structural ductility). Accordingly, the analysis of $\beta_0$ reduces to the evaluation of foundation damping $\beta_f$ and period lengthening ratio $T_{eq}/T_{eq}$. The evaluation of these two quantities is described in the following sub-sections.

3.2 ANALYSIS OF PERIOD LENGTHENING TERM $\tilde{T}_{eq}/T_{eq}$

The period lengthening can be evaluated using the structural model employed in pushover analyses using the procedure that follows (this procedure remains under investigation, and may deviate slightly from what is ultimately recommended in FEMA 440):

1. Evaluate the first-mode vibration period of the model, including foundation springs. This period is $T$. This period is calculated using initial stiffness values (prior to yield of structural or soil spring elements).

2. Evaluate the first-mode vibration period of the model with the foundation springs removed (or their stiffness and capacity set to infinity). This period is $T$. As before, this period should correspond to pre-yield conditions.
3. Calculate the ratio $\frac{\tilde{T}}{T}$, which is the period lengthening under small-deformation (elastic) conditions.

4. Calculate $\frac{T_{eq}}{T_{eq}}$ using the following equation:

$$\frac{T_{eq}}{T_{eq}} = \left[ 1 + \left( \frac{\Delta_f}{\Delta_s} \right) \left( \frac{\tilde{T}}{T} \right)^2 - 1 \right]^{0.5}$$  \hspace{1cm} (11)$$

where $\Delta_f = \text{average ductility of foundation springs}$ (described further below) and $\Delta_s = \text{target ductility level for design of superstructure}$ (typically 2-4). For structures where the inertial interaction is dominated by rotation (as opposed to foundation translation), $\Delta_f$ can be calculated as the average ductility of the vertical foundation springs. As specified by ATC 40 and FEMA 356, these springs are elastic-perfectly plastic, and the ductility of an individual foundation spring is simply the peak displacement normalized by the yield displacement. Alternatively, $\Delta_f$ can be approximated as $\Delta_f \approx \left( \frac{1}{n_e^2} \right)^2$. In many cases, $\Delta_f \approx \Delta_s$, and hence $\frac{T_{eq}}{T_{eq}} \approx \frac{\tilde{T}}{T}$.

3.3 ANALYSIS OF FOUNDATION DAMPING TERM $\beta_f$

Foundation damping term $\beta_f$ is largest for stiff structures on soft soils, and decreases as the structure/soil stiffness decreases. Other critical factors include the aspect ratio of the structure ($\beta_f$ decreases with the ratio of effective structure height to foundation radius, $h/r$) and the embedment ratio of the foundation ($\beta_f$ increases with the ratio of foundation embedment to foundation radius, $e/r$). These factors that influence $\beta_f$ also influence the period lengthening ratio of the structure. Hence, a convenient way to evaluate $\beta_f$ is through direct relationships with $\frac{T_{eq}}{T_{eq}}$. The $\frac{\beta_f - \tilde{T}_{eq}}{T_{eq}}$ relationship in Figure 7 (left side) was derived for the condition described in Section 3.1, namely uniform soil and rigid, circular foundation at the ground surface. The relationship for $e/r_u = 0.5$ (right side) is a modification for embedment, the basis of which is described below in Section 3.3.2.

An approximate equation to the curves in Figure 7 is presented below for PGA $> 0.2$ g:

$$\beta_f = a_1 \left( \frac{\tilde{T}_{eq}}{T_{eq}} - 1 \right) + a_2 \left( \frac{\tilde{T}_{eq}}{T_{eq}} - 1 \right)^2$$  \hspace{1cm} (12)$$

where $\beta_f$ is in percent and
\[ a_1 = c_e \exp(4.5 - h/r_\theta), \quad a_2 = c_e [25 \ln(h/r_\theta) - 22], \quad \text{and} \quad c_e = 1.5(e/r_u) + 1 \] (13)

Figure 7. Foundation damping factor \( \beta_f \) expressed as a function of period lengthening \( \tilde{T}_{eq}/T_{eq} \) for building different aspect ratios \((h/r_\theta)\) and embedment ratios \((e/r_u)\).

Since actual soil/foundation conditions differ from those assumed in the development of Figure 7, guidance is needed in applying this relationship to realistic conditions. This is provided in the following sub-sections.

### 3.3.1 Effect of Non-Uniform Soil

Gazetas (1991) provides solutions for the impedance of rigid foundations overlying soil for which the shear stiffness increases with depth according to prescribed functions. The damping components of these solutions are plotted in Figure 8 in terms of the \( \beta_e \) and \( \beta_\theta \) coefficients defined in Eq. 9. Also plotted for comparative purposes are the halfspace solutions. Damping values for non-uniform profiles are plotted for a zero hysteretic damping condition (radiation damping only). In Figure 8 the normalizing shear modulus and shear wave velocity are the values at the ground surface \((G_0\) and \(V_{s0}\), respectively).

Figure 8 shows that the radiation damping in translation for a non-uniform profile is less than that for a halfspace at low frequencies. For rotation, a small reduction can occur at low frequencies, but the effect is less significant than for translation. At large frequencies, the radiation damping for non-uniform profiles exceeds that for the halfspace.
The low-frequency reduction in damping is due to reflections of body waves emanating from the foundation; the frequency dependence of the reduction is related to the depth over which the shear modulus increases relative to wavelength. For short wavelengths (low $T$) body waves, the non-uniform soil medium is “seen” as being effectively uniform, whereas long wavelength (large $T$) body waves “see” a much more non-uniform medium and wave transmission into the medium is impeded. The increase of radiation damping at high frequencies is due to the higher $V_s$ of the non-uniform profiles at depth as compared to the velocity of the halfspace model (for which $V_s$ was taken as $V_{s0}$).

An extreme case of soil non-uniformity is a finite soil layer of thickness $H$ overlying a rigid base. In this case, soil damping cannot occur for periods larger than the fundamental site period, $T_s = 4H/V_s$.

Figure 8. Foundation damping factors for halfspace with and without hysteretic damping (Veletsos and Verbic, 1973) and for soil profiles with indicated shear modulus profiles and no hysteretic damping (Gazetas, 1991).
Guidelines for practical application of the above results is summarized below:

- For translational damping, profile non-uniformity is not significant for $\omega r/V_s > 1$. Case history studies by Stewart et al. (1999) suggest that inertial soil-structure interaction is generally not important for $h/(V_s T) < 0.1$. Hence, for sites where SSI is important, profile non-uniformity need not be considered if $h/r < 2\pi h/(V_s T)$ or $V_s T/r < 2\pi$. The condition stated by the inequality is generally satisfied for sites having significant inertial SSI if $h/r < 2/3$, which is often the case for short-period buildings. Accordingly, it is often justified to treat the non-uniform soil as a halfspace, taking the halfspace velocity as the in situ value immediately below the foundation.

- Rotational damping for a non-uniform profile can generally be reasonably well estimated by a halfspace model, with the half space velocity taken as the in situ value immediately below the foundation.

- For sites with a finite soil layer overlying a very stiff material, foundation damping should be neglected for periods greater than the site period.

3.3.2 Effect of Embedment

Foundation embedment refers to a foundation base slab that is positioned at a lower elevation than the surrounding ground, which will usually occur when buildings have a basement. The impedance of embedded foundations differs from that of shallow foundations in several important ways. First, the static stiffness of embedded foundations is increased. Secondly, embedded foundations can produce much larger damping due to the greater foundation-soil contact area.

An approximate and generally conservative approach for estimating the damping of embedded foundations consists of using the increased static stiffness terms coupled with ordinary $\beta_u$ and $\beta_0$ factors for surface foundations (i.e., Figure 6). This approach has been found to provide reasonable estimates of observed foundation damping in actual structures for embedment ratios $e/r_u < 0.5$ (Stewart et al., 1999). As short period structures are seldom deeply embedded, this approximate approach will often suffice for practical applications. For more deeply embedded foundations, alternative formulations can be used such as Bielak (1975) or Apsel and Luco (1987). The results shown in Figure 7 (right side) are based on this approximate approach.
3.3.3 Effect of Foundation Shape

The impedance function model described in Section 3.1 is based on representing foundations of arbitrary shape as equivalent circular mats through the use of radius terms $r_u$ and $r_\theta$ (Eq. 8). The adequacy of this assumption for oblong foundations was investigated by Dobry and Gazetas (1986), who found that the use of equivalent circular mats is acceptable for aspect ratios less than 4:1, with the notable exception of dashpot coefficients in the rotation mode. For that condition, the translational damping is underestimated at low frequencies. This effect was neglected in the development of Figure 7, which is conservative.

3.3.4 Effect of Foundation Non-Rigidity

This section addresses flexibility in the foundation structural system (i.e., the base mat, or assemblage of a base mat and grade beams/footings). The foundation flexibility referred to here is not associated with the soil.

Impedance functions for flexible circular foundation slabs supporting shear walls have been evaluated for a number of wall configurations, including: (1) rigid core walls (Iguchi and Luco, 1986), (2) thin perimeter walls (Liou and Huang, 1994), and (3) rigid concentric interior and perimeter walls (Riggs and Waas, 1985). Those studies focused on the effects of foundation flexibility on rotation impedance; the horizontal impedance of flexible and rigid foundations are similar (Liou and Huang, 1994). Foundation flexibility effects on rotation impedance were found to be most significant for a rigid central core with no perimeter walls. For this case, the flexible foundation has significantly less stiffness and damping than the rigid foundation. The reductions are most significant for narrow central cores and large deviations between soil and foundation slab rigidity.

Significant additional work remains to be done on foundation flexibility effects on impedance functions because the existing research generally has investigated wall/slab configurations that are seldom encountered in practice for building structures. Nonetheless, based on the available studies and engineering judgment, the following preliminary recommendations were developed:
• The rigid foundation assumption is probably generally acceptable for the analysis of damping associated with horizontal vibrations of reasonably stiff, inter-connected foundation systems.

• For buildings with continuous shear walls or braced frames around the building perimeter, and continuous footing or mat foundations beneath these walls, the rigid foundation approximation can be used to provide a reasonable estimate of damping from rotation vibrations. In this case, the effective foundation radius \( r_\theta \) would be calculated using the full building dimensions. This recommendation also applies if continuous basement walls are present around the building perimeter. This case is referred to as **stiff rotational coupling**.

• For buildings with a core of shear walls within the building, but no shear walls outside of this core, a conservative estimate of foundation damping can be obtained by calculating the effective foundation radius \( r_\theta \) using the dimensions of the wall foundations (which, in this case, would be smaller than the overall building plan dimensions). This is an example of **soft rotational coupling** between the shear walls and other load bearing elements.

• For buildings with distributed shear walls at various locations around the building plan, the key issues are (1) the rotational stiffness of the building system as a whole (i.e., does the building tend to rotate as a single rigid block due to significant rotational stiffness coupling between adjacent elements, or do individual vertical components such as shear walls rotate independently of each other?), and (2) the degree to which destructive interference occurs between waves emanating from rotation of distinct foundation components.

   In most cases, rotational coupling between vertical components is limited. In such cases, when foundation elements are widely spaced, the destructive interference would be small, and from a conceptual standpoint, it should be possible to evaluate the effective foundation system moment of inertia \( I_{f,\text{eff}} \) by assuming the walls act independently, as follows:

   \[
   I_{f,\text{eff}} = \sum_i I_{f,i} \quad (13)
   \]
where $I_{f,i}$ represents the moment of inertia of an individual wall foundation. The effective foundation system radius ($r_{\text{eff}}$) for rotation would then be calculated using $I_{f,\text{eff}}$ in Eq. 8. This is an example of soft rotational coupling without destructive interference. However, when foundations are more closely spaced, destructive interference will occur and the above formulation may be unconservative. Unfortunately, this topic has not been researched, and thus what footing separation distances constitute “close” and “widely spaced” is unknown, which in turn precludes the development of recommendations for the analysis of rotation damping for distributed walls.

If rotational stiffness coupling between vertical elements is large (i.e., they tend to rotate as a rigid unit, e.g., because of deep spandrel beams between adjacent shear walls), but the vertical elements have independent footings, then the building has what is referred to as intermediate rotational coupling. In this case, the moment of inertia of the coupled elements can be estimated as

$$I_{f,j} = \sum_{j=1}^{M} A_{j} y_{j} + \sum_{j=1}^{M} I_{f,j} \quad (14)$$

where $I_{f,j}$ = effective moment of inertia of $j=1$ to $M$ coupled elements, $A_{j}$ = area of footing $j$, $y_{j}$ = normal distance from the centroid of the $jth$ footing to the rotational axis of the coupled elements, and $I_{f,j}$ = moment of inertia of footing $j$. If the vertical elements for the entire building have intermediate rotational coupling, then $I_{f,i}$ from Eq. 14 is the effective moment of inertia for the foundation system as a whole. If the intermediate rotational coupling only occurs between selected vertical elements, then $I_{f,i}$ from Eq. 14 represents one contribution to the overall effective foundation moment of inertia, which can be calculated in consideration of all of the elements using Eq. 13 (with due consideration of potential destructive interference effects).

For buildings with only moment resisting frames (no walls or braced frames), foundation rotation is not likely to be significant, and hence foundation flexibility effects on rotation damping are also likely insignificant.

### 3.4 RECOMMENDED PROCEDURE

**Step 1**: Evaluate effective foundation radii using Eq. 8. The foundation area ($A_{j}$) for use in Eq. 8 is the full plan area if foundation elements are interconnected. The evaluation of an
appropriate moment of inertial \((I_j)\) is discussed in Section 3.3.4. Determine the foundation embedment, \(e\), if applicable.

**Step 2:** Evaluate effective structure height, \(h\), which is taken as the full height of the building for one story structures, and as the vertical distance from the foundation to the centroid of the first mode shape for multi-story structures. In the later case, \(h\) can often be well approximated as 70% of the total structure height.

**Step 3:** Evaluate the period lengthening ratio for the structure using the site-specific structural model developed for nonlinear pushover analyses. See Section 3.2 for details.

**Step 4:** Evaluate the initial fixed base damping ratio for the structure \((\beta_i)\), which is often taken as 5%.

**Step 5:** Using Figure 7 and the guidelines in Section 3.3, estimate foundation damping \((\beta_f)\) based on \(\frac{T_{eq}}{T_{eq}}\), \(e/r_u\), and \(h/r_\theta\).

**Step 6:** Evaluate the flexible base damping ratio \((\beta_o)\) from \(\beta_f\), \(\beta_i\), and \(\frac{T_{eq}}{T_{eq}}\) using Eq. 10.

**Step 7:** Evaluate the effect on spectral ordinates of the change in damping ratio from \(\beta_i\) to \(\beta_o\) using established models (e.g., Eq. 8-10 of ATC-40; model for use in FEMA 440 remains under investigation).

### 4.0 CONCLUSIONS

In this paper, we have presented sets of recommendations for incorporating the effects of kinematic soil-structure interaction and foundation damping into assessments of seismic demand for use in nonlinear static analysis procedures for building structures. These effects generally decrease the seismic demand relative to what would be used in current practice, which is based on 5% structural damping and equivalent foundation and free-field motions. The demand reduction is greatest at short periods. The recommended procedures are summarized in Sections 2.4 and 3.4, respectively.

### ACKNOWLEDGEMENTS

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REFERENCES

Design and actual performance of a super high R/C smokestack on soft ground

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Dynamic analysis considering soil-pile-structure interaction was adopted for the seismic design of a super high reinforced concrete smokestack supported by pile foundation on a reclaimed soft ground in Osaka, Japan. After construction, a series of vibration tests and microtremor measurements were conducted. Moreover, the seismic array observation was carried out to successfully obtain the records during various earthquake events including the 1995 Kobe earthquake. The objectives of this paper are to show the modeling and features of the dynamic analysis model for the seismic design, and to verify the model based on the vibration tests and the seismic observation. The seismic design model is a coupled lumped-mass model consisting of pile-structure and soil systems combined with springs representing the effect of interaction. The analytical results show very good match with the observation results, in terms of transfer functions, time histories and shifted natural periods due to irreversible nonlinear behavior.

INTRODUCTION

A 200 m high reinforced concrete smokestack supported by a long-pile foundation system was constructed on a reclaimed soft ground in Osaka, Japan. Prior to the seismic design of the smokestack, the predominant periods of the ground and the structure were estimated to be long and have been found to become longer under severe seismic motions. It was worried that some transient resonance between the ground and the structure might occur and due to excited nonlinear behavior of soils, unexpected stresses might be produced by the amplified ground displacement. Therefore, it was necessary to study the inertial and kinematic interactions between the soft ground, the pile foundation and the smokestack well during the seismic design. The main objective of this paper is to verify the dynamic nonlinear soil-structure interaction (SSI) analysis model for the seismic design based on the vibration tests and seismic observations. First, an overview of the seismic design of the smokestack and the models of the dynamic analyses for the seismic design is described featuring their

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dynamic characteristics. Second, vibration tests carried out after the construction are discussed especially from the standpoint of the transfer functions between the ground and the structure under very small amplitude vibration. Third, seismic observation results including those during the 1995 Hyogo-ken Nanbu earthquake (also known as the Kobe earthquake) are discussed from the standpoint of the nonlinear behaviors of the ground and the smokestack. Finally, the design models are verified by comparing the results in terms of the time histories of responses of the ground and the smokestack and the transfer functions (i.e., spectral ratios) obtained from the analysis and actual measurements.

In particular, the effectiveness of a simple lumped mass-beam-spring model for the nonlinear SSI analysis in practice is emphasized. So, it may be appropriate to review some typical previous research literatures. Uchida et al. (1980, 1981) analyzed the strong motion records obtained during two big earthquakes in 1978 on an 18-story building made of steel and reinforced concrete, and conducted the elasto-plastic dynamic analysis so as to clarify the change of its natural period during the two earthquakes. Likewise, for Onto Millikan Library in California, four series of vibration experiments were conducted from 1966 to 1969 prior to the 1971 San Fernando earthquake, and another five series of experiments were also carried out from 1971 to 1975 after the earthquake (Luco et al., 1987). Foutch and Jennings (1978), based on their experimental results, clarified the fact that the resonant frequency of a hospital building decreased while the displacement amplitude of its top increased. Moreover, they asserted that this was because of the degradation of the rigidity related to soil-structure interaction during the earthquake motion. In contrast, Luco et al. (1987) asserted that the change of the resonant frequency was not because of the degradation of the rigidity related to the soil-structure interaction but because of the degradation of the rigidity of the super-structure. Celebi (1997) examined the behavior of the six-story Olive View Hospital during the 1987 Whittier earthquake and the 1994 Northridge earthquake using the acceleration records. He clarified that the fundamental frequency of the hospital decreased more significantly during the 1987 earthquake (0.91g in PGA) than the 1994 earthquake (0.061g in PGA). Li and Mau (1997), on the other hand, conducted a system identification analysis and discussed the change of the natural frequencies of 21 buildings where the 1987 Whittier and the 1989 Loma Prieta earthquake records were obtained. Ohba and Hamakawa (1997) also attempted to find the cause of changes of natural periods of buildings that suffered damage to the superstructure or the foundation piles due to the excitation of the Kobe earthquake based on a series of microtremor measurements at the buildings. The
researches mentioned above are thus limited to studying the change in natural period of the structures after experiencing strong earthquake motions. However, investigations related to the change in natural period of the structures occurring right before and after the strong motions are lacking.

OVERVIEW OF THE SEISMIC DESIGN OF THE SMOKESTACK

The 200 m high smokestack for Nanko LNG thermal power plant of Kansai Electric Power Company was erected on a soft manmade island, often known as Osaka Nanko, in 1990. The Nanko Power Plant site can be characterized as a manmade island reclaimed on a very soft thick deposit requiring a long-pile foundation for the structure and having a predominance of longer period components during the earthquakes due to deep Osaka basin. Therefore, the soil-structure interaction between the thick soft soil layer and the super-high smokestack supported on the pile foundation and the pile stress due to ground displacement required careful study and evaluation during the seismic design (Kida et al. 1992). The author worked in the seismic design of the smokestack as a leading engineer from 1987 to 1989. A bird’s eye view of the smokestack is shown in Photo 1.

Photo 1. Bird’s eye view of the smokestack of Nanko Thermal Power Plant of KEPCO (By courtesy of KEPCO)
The location of the site is such that it lies on a reclaimed land near the Osaka Bay area and at almost the center of the basin surrounded by the mountains, as shown in Figure 1. From the viewpoint of earthquake ground motion, 3 to 4 period components tend to be predominant in Osaka City area, which is considered to be due to the surface wave induced by the basin effect of the Osaka Basin. The strongest ground motion experienced at the site during the 1995 Kobe earthquake with an epicentral distance of about 22 km will be discussed later.

Figure 1. Location of the site and the epicenter of the 1995 Kobe earthquake

Figure 2 shows the front and side views of the smokestack together with the arrangement of the accelerometers on the smokestack as well as the ground. The smokestack consists of three internal cylinders made of FRP and an external hexagonal reinforced concrete structure supporting the cylinders. The height of the internal cylinders is 200m and that of the external structure is 194m. The cross section of the external structure is a hollow hexagon with two different sides attached with three pairs of straight wings, and the outline of vertical figure follows a straight line and a parabolic curve. The width of the wings decreases gradually from 8.0 to 3.1 m toward the top of the smokestack while the maximum internal width of the hexagon decreases from 18.5 to 13.1 m, and the wall thickness decreases from 100 to 30 cm. Near the bottom of the smokestack, three openings exist for horizontal penetration of the smoke pipes connected to the internal cylinders. Each opening has a length of 10 m and a width of 8m, and is located at 5 m above the base of the smokestack. The external structure was constructed by slip-form concrete method such that it supports the internal FRP cylinders at 18 and 177 m of elevation points.
The accelerometers, as indicated in Figure 2, consist of two horizontal component accelerometers fixed at 65, 131, and 193.5 m height on the smokestack, and a three component accelerometer on the smokestack base. The aim of fixing the accelerometers is to measure the third vibration mode of the smokestack. Two additional vertical accelerometers on the base aimed for extracting two-directional rocking of the base were incorporated with one on it. A pair of three component accelerometers was also installed at the depths of 1 m and 70 m below the ground so as to measure the principal behavior of a free field 100m away from the smokestack.

The plan and elevation of the smokestack base along with the arrangement of 273 piles are show in Figure 3. The piles are seen concentrated near the edges of the basement with at least 2-meter interval for effectively resisting the rocking of the base. Each pile is made of five segments consisting four pre-stressed high strength concrete piles (PHC piles) and one

Figure 2 Front and side views of the smokestack and the arrangement of accelerometers on it and on the ground
steel reinforced PHC pile (B-type: the second strongest) on the top with an external diameter of 80 cm. Each pile has a length of 65 m and reaches the pile base layer composed of second dilluvial gravely sand at a depth of 72 m. All the piles penetrate the soft and hard layers, so there was an anxiety during the construction that seismic ground displacement could induce some stress concentration in the piles near the boundaries between the soft and hard layers. The base of the smokestack measures 6.5 m deep and 51 m wide having sufficient rigidity against rocking.

The detailed soil profile at the site is shown in Figure 4 together with the SPT N-values and shear wave velocity profiles. The soil profile consists of different layers, which include 6.5 m thick banked layers, 10.7 m thick filled layer, 25.5 m thick alluvial clay-silt-sand layer (Ma13), 8 m thick dilluvial gravel layer (Temma Layer), 14.5 m thick dilluvial clayey layer (Ma12), and 11 m thick second dilluvial gravely sand layer. The base for the piles was selected to be 11 m thick second dilluvial gravely sand layer. The reclamation work was carried out in 1972 through 1980. The banked layers were filled with the material exploited from the mountains and waste soil from the construction sites after the lower layer was dredged and sand drains were applied for consolidation of alluvial clayey layer.

The performance requirements for the seismic design of the smokestack were specified
for two levels of earthquake motion: 1) serviceability during and after the Level 1 motion and 2) safety during and after the Level 2 motion. The maximum velocities of ground surface for the Level 1 and 2 motions were evaluated to be 25 and 50 cm/s respectively. The input
ground motions for the dynamic response analysis for the seismic design were basically defined as surface ground motion of the free field because a fixed base model was adopted as a basic model of dynamic analysis as per the Japanese conventions. Basically, four strong motion acceleration time histories including El Centro (NS), Taft (EW), and Hachinohe (NS) records, as per the conventions, were adopted as input motions with scaled amplitude in the previously mentioned velocities. The major performance criteria were the ductility factor and allowable shear stress for the smokestack, the allowable tensile stress for the internal FRP cylinders, and the allowable stresses for the foundation base and the piles. The criterion of the ductility factor with regard to the Level 2 motion was 2.0, which resulted in a ductility factor of 1.45 in the maximum response for the Level 2 motion with Hachinohe record, which is rich in longer period components. The ground was assumed to be a horizontally layered system during the analysis, and three different ground models were prepared in terms of the degree of non-linearity of soils, as shown in Figure 5.

The basic ground model, which is assumed to have a set of rigidity determined based on

![Figure 5](image_url)

**Figure 5** Three different ground models depending on the degree of non-linearity of soil
the PS logging at the site, is regarded as an elastic linear ground, and is used in the analysis for the ground motions with small amplitude. The other two models have different values of rigidity and damping ratio under the Level 1 and 2 motions. The values were determined as an average of three convergent values in the cases of equivalent analyses with El Centro, Taft, and Hachinohe as the input motions by using SHAKE program. Each analysis was repeated until the surface velocity response converged to the given magnitude of Level 1 or 2 motions. Figure 5 shows the shear wave velocity and damping ratio profiles of these three different models of the ground. The magnitudes of predominant strain in the seismic grounds ranged from 0.03 to 0.2 % for the Level 1 motion and from 0.05 to 5 % for the Level 2 motion.

MODELS OF DYNAMIC ANALYSES FOR THE SEISMIC DESIGN

Generally, a fixed base model is adopted for seismic design when the effect of soil-structure interaction is considered negligible. When this effect is taken into account, the use is commonly made of a sway rocking model for the structures with spread or short-pile foundation. The SSI effect on the structure as well as piles, however, is considered significant in case of long piles in soft ground. In such a case, some coupled system should be adopted for the seismic design model. A finite element model would be powerful in evaluating the SSI effect if it were linear. Unfortunately, however, it was not available for the structural and geotechnical nonlinear models at that time. So, a lumped mass-spring-beam model was

![Figure 6 Models of preliminary analyses as candidates for the seismic design](image-url)
considered appropriate for taking into account the major SSI effect as well as the material non-linearity in the structure and the ground. As a result, the author and one of his colleagues developed and proposed the lumped mass-spring-beam model (Mori et al. 1992; Mori 2000). Figure 6 shows the models mentioned above.

In order to evaluate the appropriateness of the above-mentioned models in the seismic design, preliminary analyses employing these models with the soil properties in the Level 1 motions were conducted, and the results from the different models were compared. The three of the four actual models employed in the preliminary analyses are shown in Figure 7 for reference. In addition, the modified FLUSH model was studied.

![Figure 7](image)

(a) Proposed model    (b) 2D FEM(FLUSH) model   (c) Fixed base model

**Figure 7** Three of four actual models for the preliminary analyses

As the basic information, the predominant periods of the smokestack obtained from the above-mentioned models are given in Table 1. In 2D-FEM model, the predominant periods are specified by its transfer function of the top of smokestack to the base of the ground. In the rest of the models, however they are specified according to the result of eigenvalue analysis.

<table>
<thead>
<tr>
<th>Type of model</th>
<th>1st</th>
<th>2nd</th>
<th>3rd Rocking</th>
<th>3rd Sway</th>
<th>4th Rocking</th>
<th>4th Sway</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fixed base model</td>
<td>2.22</td>
<td>0.49</td>
<td>0.20</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sway-rocking model</td>
<td>2.33</td>
<td>0.54</td>
<td>0.21</td>
<td>0.25</td>
<td>0.13</td>
<td></td>
</tr>
<tr>
<td>Lumped mass-spring-beam model</td>
<td>2.33</td>
<td>0.54</td>
<td>0.21</td>
<td>0.29</td>
<td>0.12</td>
<td>0.11</td>
</tr>
<tr>
<td>2D-FEM model</td>
<td>2.38</td>
<td>0.58</td>
<td>0.23</td>
<td></td>
<td></td>
<td>0.12</td>
</tr>
</tbody>
</table>

Note1: SSI models are based on the soil properties assumed under Level 2 earthquake motions
Note2: The detail of the smokestack for preliminary analyses was different from the final structure.
The transfer functions in the amplitude, i.e., Fourier spectral ratios, of the top of the smokestack and its base to the ground surface of the free field and to the ground surface with regard to horizontal movement as obtained from the proposed model and the two dimensional FEM model are compared in Figure 8 (a) and (b). As for the transfer functions of the top of the smokestack, the predominant periods of the first and second modes in these two models match well. However, the amplification near the second predominant period for the smokestack from the proposed model is much greater than that from the FEM model, whereas it is only slightly greater over the frequency ranges beyond the second predominant period. The 2D FEM model may overestimate the dissipation damping of the smokestack, so the proposed model was considered more appropriate in the practical seismic design. As for the transfer functions of the smokestack base, the input loss effects due to kinematic interaction are simulated in the similar manner, and the results in terms of phase differences are shown in Figure 9. The results from both the models are seen to match well as a whole.

On the other hand, the sway-rocking model for the smokestack was not adopted for the dynamic analysis model for the seismic design, because of great discrepancy with the FEM

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**Figure 8** Transfer functions of the top of the smokestack and of the basement to the ground surface of the free field with regard to horizontal movement

**Figure 9** Transfer functions in phase difference of the top of the smokestack and of the basement to the ground surface of the free field with regard to horizontal movement
model near and beyond the second predominant frequency in the above-mentioned transfer functions both in amplitude and phase difference.

In order to understand the natural modes in the proposed model, some lower vibration modes of the proposed model together with those of the fixed base model are shown in Figure 10. As shown in the figure, the modes of sway and rocking of the foundation are clearly recognized and excited modes and amplitudes of the smokestack in the periods of predominant or natural ground vibration can be quantitatively understood (Mori 2000).

**Figure 10** Lower vibration modes of the proposed model, which can be identified to the specific modes of the ground, the structure, or the foundation
VERIFICATION OF THE MODELS BASED ON VIBRATION TESTS

A series of vibration tests including microtremor measurements and artificial excitation (i.e., by the use of human power) tests were carried out just after the construction in August 1990 (Kida et al. 1992). The artificial excitation for the first natural period of the smokestack was produced by a cyclic movement of individual centers of gravity of 27 persons on top of the smokestack, while that for the second natural period was produced by applying a joint push of 12 persons on the wall at the top. In order to find a point that could be regarded as the free field for the smokestack-ground system, array observation of microtremor was also carried out. Figure 11 shows the arrangement of the sensors (velocity meters) for the ground and the smokestack during the tests. Figure 12 shows the Fourier spectra of the ground surface and the base of the smokestack, which indicates a gradual decrease of the amplitude.

Figure 11 Arrangements of sensors for the ground and the smokestack

Figure 12 Fourier spectra of the ground surface and the basement
of the base in higher frequency compared with the ground surface. Moreover, the spectral ratios of the base to the ground surface obtained from the microtremor measurements as well as the analysis by the proposed method are shown in Figure 13(a). This figure clearly shows the input loss effect due to kinematic interaction, which can be successfully simulated by the analysis using the proposed model. Moreover, the transfer functions with regard to rocking of the smokestack base to the horizontal ground surface motion, as obtained from the measurement and the analysis are compared in Figure 13(b). This analytical model may underestimate the amplification of the rocking effect due to SSI, especially around the second predominant period.

![Figure 13](image)

**Figure 13** Spectral ratios of the basement to the ground surface both by microtremor measurement and the analysis by the proposed method

Next, the transfer functions of the top of the smokestack to the ground surface of the free field as obtained from the microtremor measurement and the analysis using the proposed model with the soil properties under the Level 2 earthquake motions are compared in Figure 14. Two distinctive features can be seen in this figure. First, the predominant frequencies of

![Figure 14](image)

**Figure 14** Transfer functions of the top of the smokestack to the ground surface of the free field by microtremor measurement and analysis with the proposed model
the measurement are obviously observed to be greater than those of the analysis around the first and the second predominant frequencies, and the ratios of the measured values to the analytical results are almost the same. Second, the shapes of these two transfer functions are almost proportional. These two features are considered to be due to the difference in dynamic properties of the analytical model and the actual structure. The first predominant frequency does not seem to be strongly influenced by the SSI, which means that the difference may be due to the difference of the flexural rigidity of the smokestack.

Furthermore, a comparison of the first and second vibration modes of the smokestack as obtained from the measurement and the analysis is made in Figure 15. Both the microtremor measurement and the artificial excitation test are seen to have resulted in the same vibration mode except for a slight difference of the predominant frequency of the second mode.

![Figure 15](image-url) Comparison of the first and second vibration modes of the smokestack between the measurement and the analysis

Table 2 shows a summary of the predominant periods of the first and second modes as obtained from the analysis and the measurement. The fundamental periods of the smokestack are found to be different. Accordingly, such differences are hereinafter going to be studied

**Table 2** Summary of predominant periods of the first and the second mode by the analysis and the measurement

<table>
<thead>
<tr>
<th>Model and measurement</th>
<th>Top/Free field</th>
<th>Top/Basement</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1st 2nd</td>
<td>1st 2nd</td>
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Unit: second
from the standpoint of Young’s modulus of concrete. The design value of the Young’s modulus of concrete, $E_c$ used in the smokestack was $E_c=2.3 \times 10^5$ kgf/cm$^2$, which was estimated by using an empirical relationship between Young’s modulus and the strength of concrete considering the design strength of concrete, $F_c=240$ kgf/cm$^2$.

The strengths of the actual concrete used in the smokestack are statistically shown in Figure 16. The average compressive strength of the concrete measured at the construction site was 410 kgf/cm$^2$, and the average Young’s modulus can be estimated to be $E=3.3 \times 10^5$ kgf/cm$^2$, which was to be applied in the analysis for vibration experiment. The natural periods according to the proposed model with such modification was almost the same as the measured ones.

![Figure 16 Histogram of the strength of concrete for the smokestack](image)

The damping ratio was measured based on the free damped vibration after the artificial excitation. Figure 17 shows the time history of displacement at the top during the free damped vibration after the artificial excitation. The damping ratio was measured to be approximately 1.1% at all the heights of sensors from the free vibration. Additionally, the damping ratio estimated by the half power method with the microtremor measurement varied from 1.1 to 1.5%, which is almost the same as the value estimated from the artificial excitation.

![Figure 17 Time history of displacement at the top during the free damped vibration](image)
excitation. The design value of the damping ratio was 2%, which was considered appropriate taking into account its dependency on the strain.

**ACTUAL SEISMIC BEHAVIOR OF THE GROUND AND THE SMOKESTACK**

The earthquake observation for the smokestack and the ground was carried out right after the completion of the construction in March 1990. The arrangement of the seismometers has already been mentioned elsewhere. The observations were made until the end of 1997 during twelve earthquakes, the epicenters of which are shown in Figure 18 (Kowada et al. 1997). The ten of these earthquakes were the main event and the aftershocks of the 1995 Kobe earthquake, as mentioned also in conjunction with Figure 1.

![Figure 18 Location of the epicenters of the earthquakes observed at Nanko site](image)

For grasping the overall amplification or de-amplification through the ground, the foundation, and the smokestack, the relationship of the maximum accelerations among the base layer, the ground surface, and the basement and the top of the smokestack are shown in Figure 19. The amplification factor through the subsurface ground is not so great, varying mostly from one to two. From the relationship between the ground surface and the basement of the smokestack, de-amplification can be found especially in the range of low amplitude. This de-amplification can be understood as the effect of input loss due to the kinematic interaction between the foundation and the ground, whereas this effect is negligible in the case of the main event of the Kobe earthquake. The amplification through the smokestack varies from two to five times, and the factor seems to be greater when the amplitude of the ground surface acceleration is smaller.

The predominant frequencies of the ground and the smokestack were determined based
on the predominant peaks in the spectral ratios of the ground surface to the base layer and in those of the top to the base of the smokestack, respectively. Figure 20 shows the relationship between the predominant frequencies and the magnitude of the input in the systems of the ground and the smokestack. The dependency of the predominant frequency on the input to the systems, such as the ground and the smokestack, can not be clearly seen.

Figure 19 Relationship of the maximum accelerations among the base layer, the ground surface, and the basement and the top of the smokestack

In order to study the change of the first and second predominant frequencies along the progress of the time, the change of the predominant periods in the order of the time of the earthquakes is shown in Figure 21. As for the predominant periods of the ground, those during the Kobe earthquake are the longest both in the first and second ones, and those after the Kobe are longer than those before the Kobe in the first predominant period, while the

Figure 20 Relationship between the predominant frequencies and the magnitude of the input in the systems of the ground and the smokestack
reverse relation can be recognized in the second ones. The second predominant period can be considered influenced by the soil properties in relatively shallower soils. Accordingly the excitation by the Kobe earthquake might have densificated the shallower sandy soil deposits. On the other hand, a slight change of the first predominant period toward the longer side can be supposed to be due to the effect of softening in clayey soil deposits in deeper location. As for the predominant periods of the smokestack, the first one during the main event was significantly long, and those after the Kobe are longer than those before the Kobe. The averaged values of the first predominant period before and after the Kobe earthquake are 1.94 and 2.09 seconds, respectively. Moreover, the averaged values of the second predominant period before and after the Kobe are 0.48 and 0.56 seconds, respectively. These irreversible changes of the predominant periods are considered to mean that the stresses in the smokestack had gone far beyond the elastic limit or the cracking limit (Kowada et al., 1998).

**VERIFICATION OF THE MODELS BASED ON STRONG MOTION RECORDS**

The ground motion observed during the Kobe earthquake is approximately equal to the magnitude of the Level 1 design motions in terms of velocity of the ground surface. In
addition, the behavior of the ground and the smokestack has been clarified to be strongly nonlinear in the previous section. Consequently, the numerical simulation with the record during the Kobe earthquake by the seismic design models including the conventional fixed base model and the proposed model can be suitable verification of the seismic design of the smokestack. Figures 22 and 23 show the time histories of the north-south components of the acceleration and displacement of the ground and the smokestack, respectively. The duration of the principal motions of the ground approximately begins at 16 seconds and ends at 28 seconds, whereas the significant amplitude of vibration when four-second component is highly predominant takes place in the later stage just after the principal motion continues up to 120 seconds in the top of the smokestack. In particular, the maximum displacement of the top takes place in the later stage. This four-second predominant component is also significant.

**Figures 22** Acceleration time histories of the north-south components of the ground and smokestack
in the displacements at the two depths of the ground with almost the same amplitude, which is hence considered to be a kind of surface wave, probably the Love wave.

A numerical analysis was conducted by the proposed model used in the seismic design with the recorded acceleration at the base at a depth of 70 m as the input motion for the models. The analysis by the fixed base model was conducted with the recorded ground surface motion as the input motion. Figure 24 shows the acceleration response time histories of the top of the smokestack, the basement of the foundation, and the ground surface together with the measured ones. The response of the top of the smokestack is focused on the time range of the principal motion because this is thought to be dominated by vertically incident shear wave.
In the response of the ground surface, a good agreement is seen in Figure 24. Accordingly, the difference of the ground surface motion between the two models may not be so influential to the response of the smokestack. Comparing the waveform of the basement with that of the ground surface, it is clearly understood that the short period components are obviously reduced. As for the degree of this reduction, the measured one is greater than the analytical one, which means the reduction may not be only due to kinematic interaction along the depth but also due to that in the horizontal plain.

Figure 24 Acceleration response time histories of the top of the smokestack, the basement of the foundation, and the ground surface together with the measured ones, which is focusing on the time range of the principal motion.
As for the response of the top of the smokestack, both the analytical results roughly match the measured one in terms of amplitude and phase; however, the phase in the response of the fixed base model slightly advances more than the measured one, while that of the proposed model well matches the measured one. This means that the response of the smokestack from the motion directly transmitted through the pile foundation is presumably dominant in the entire response of the smokestack and the proposed model is able to simulate this mechanism.

Next, the spectral ratios of the top of the smokestack to the ground surface for the north-south component are shown in Figure 25 in order to discuss soil-structure interaction effect from the viewpoint of the transfer function between them. For the first predominant period, the two analysis results match the measurement. For the second predominant period, the result of the proposed model well matches the measured one, but that of the fixed base model is shorter than the measured one. Additionally, a small peak around 1.2 to 1.3 seconds, which may be due to the effect of the first predominant mode of the ground, can be simulated only by the proposed model but not by the fixed base model.

![Figure 25 Spectral ratios of the top of the smokestack to the ground surface and to the base layer for the north-south component](image)

Figure 25 shows various kinds of the spectral ratios of the proposed model and the measurement. That of the top of the smokestack to the base layer, which represents the overall dynamic characteristics of the coupled model, matches the measurement result. That of the ground surface to the base layer, which represents the dynamic characteristics of the
ground, also matches the measurement result. As for that of the basement to the ground surface, which represents the effect of input loss in the foundation, both the shapes are roughly the same, whereas the detailed shapes are different.

Figure 27 shows the maximum responses of bending moment and curvature of the smokestack on the skeleton curves with regard to the direction when the north side of the smokestack is in tension and with regard to the east-west direction. In this figure, the two breaking points of each skeleton curve correspond to the cracking and the yielding surfaces. The yielding surface is defined as a situation when the most outer reinforcement bars start to yield. According to the figure, the response of the smokestack went beyond the cracking surface in the range of heights from 20 to 120 m in case of the proposed model, while such
situation occurred in a range of heights from 30 to 90 m. The values of residual rigidity of the smokestack estimated from its changed predominant periods, which was mentioned earlier, might correspond to those in case of the proposed model.

Based on the comparison of the results of the analyses using the proposed model and the measurement with regard to the time histories of accelerations of the ground and the smokestack, the transfer function of the smokestack, and the relationship between the change of the predominant periods and the nonlinear response of stress-strain of the smokestack, the appropriateness and effectiveness of the proposed model can be verified.

CONCLUSIONS

The soil-structure interaction model proposed by Mori et al. (1992) and Mori (2000) was adopted in the seismic design of a 200 m high reinforced concrete smokestack constructed on a soft ground in Osaka, Japan. The fundamental dynamic characteristics of the smokestack were examined by microtremor measurements and manpower excitation tests with regard to its elastic behavior. Moreover, the earthquake observations on the ground and the smokestack were carried out, and the strong motion records during the 1995 Kobe earthquake were analyzed and used in the numerical analyses with the seismic design models. The results of a series of tests on the elastic behavior and the numerical simulation on the nonlinear behavior of the ground and the smokestack could verify the appropriateness of the proposed model as the seismic design model and its effectiveness as a nonlinear SSI analysis model qualitatively as well as quantitatively.

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REFERENCES


An Investigation on Aspects of Damage to Precast Concrete Piles Due to the 1995 Hyogoken-Nambu Earthquake

Yoshihiro Sugimura, a) Madan B. Karkee, b) and Kazuya Mitsuji c)

Aspects of typical damage at deeper underground part of precast concrete piles, referred to as the K-shaped (in Japanese character pronounced "ku") failure in this paper, have been investigated. Such failures are attributed to lateral flow at liquefied reclaimed land, such as Port Island and Rokko Island in Kobe city, during the 1995 Hyogoken-Nambu earthquake. A simple static analysis proposed earlier by the first author and referred to as the load distribution method has been utilized to evaluate the horizontal resistance of piles and to investigate the cause of this type of failure. Results of the analysis show that bending stresses in piles exceed pile strength. In addition, the shearing stress at slip surface formed by liquefied soil layer is close to the ultimate strength. It is shown that the effects of lateral flow on piles may be effectively represented by the combination of distributed load arising from active earth pressure due to separation of frontward soil from piles and the concentrated load corresponding to the slip force in soil behind the piles.

INTRODUCTION

The 1995 Hyogoken-Nambu earthquake resulted in damage examples in no small numbers of not only superstructures but also pile foundations, especially precast concrete piles such as prestressed concrete (PC) piles, supporting residential buildings (AIJ, 1996). In general, the damage aspects may be classified into two categories, consisting of failure close to the pile top and at deeper parts of long piles. The former is the bending-shear or shear failure type of failure caused mainly by inertia force of superstructure, which the authors have pointed out repeatedly as the repetition of the nature of damage noted during the 1978 Miyagiken-oki earthquake (Sugimura et al, 1998, Ishii et al, 2002). The latter refers to damage or breaking of pile member at relatively deeper part usually due to lateral

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flow at the boundary or within a liquefied layer. This may be considered to be a damage aspect peculiar to the Hyogoken-Nambu earthquake, and is referred to as the K-shaped (in Japanese character pronounced "ku") failure. This paper is concerned with the investigation of this type of failure based on a newly proposed approach of hypothetical failure mechanism.

OUTLINE OF THE RESEARCH

Of the various research attempts to clarify the occurrence mechanism of K-shaped failure in piles, a committee report culminating in the symposium held in May 1998 by the Japanese Geotechnical Society concerning the flow and permanent deformation of the ground and soil structures during earthquakes is noteworthy. The initiative may be considered a concerted such effort, because it resulted in an inclusive summary (JGS, 1998). According to the report, methods of evaluating the effect of lateral flow of fluid ground on piles may be broadly classified into the following three groups: (i) earth pressure method, (ii) fluid pressure method and (iii) response displacement method.

Characteristically all the three methods above involve consideration of the effect of pile being pushed forward by the flowing ground mass behind the pile. However, close observation of actual cases such as shown in the photos in Figures 1 and 2 indicate cracks in the ground extending from the edge of quay wall to the toe of sloping ground. In fact, based on the simple experimental and numerical studies on another damage example of ground fissures radiating from the footing of a bridge pier, Tazo et al. (1999) have pointed out that the ground fissures appeared not because of the spreading ground pushing from behind the piles and footings, but by movement toward the river (or sea) of the ground behind the quay wall. The observation and explanations give the impression that the effect of lateral spreading emerged in a manner similar to what is known as slump in the field of landslide, as shown in Figure 3(a). The similarity of the image of the occurrence of lateral spreading reconstructed detailed field observation with the slump in landslide is seen to be remarkable in Figure 3. This interpretation that the fluid soil layer does not push the piles from the back but leaves from in front of the piles is worth noting as a unique idea. This in turn reminds one of Nojiri's work (1997) emphasizing the removal of earth support on the excavated side as the ‘mechanism of unloading’ in excavation process.

There is another research work drawing the interest of authors. Nakazawa et al. (1996) inferred the occurrence of a slip surface above which the lateral flow of the liquefied reclaimed layer had occurred, as shown in Figure 7(a). They have explained that failure of
piles under northern footings occurred due to inertia force of the superstructure while the failure of the piles under southern footings occurred due to slippage within the ground. Subsequently, Nakazawa et al. (1999) have presented a more detailed report on this building and ground conditions and reconfirmed the difference of failure type between northern and southern pile group noted earlier. That is, the damage to piles under southern footings was caused by lateral flow of the liquefied reclaimed layer, referred to as the K-shaped failure in this paper as an attempt to identify it as a new type of failure mode.

Figure 1. Photograph showing overall view of the building looking from the west side, with blocks A, B and C from left to right, where the damage to quay wall to the west can be noticed.

Figure 2. Photograph showing closer view of the south-west corner, where the widened gap due to bulging out of the quay wall towards the sea can be clearly seen.

Figure 3. The schematic illustration of the K-shaped failure mechanism indicating the similarity of (a) slump during landslide with the (b) lateral spreading around quay wall due to liquefaction.
THE MECHANISM OF K-SHAPED FAILURE

The photograph in Figure 2 shows the bulging and southward movement of the quay wall. As a result, several cracks parallel to the building had appeared on the hardtop of parking area to the west and there were vertical cracks on the quay wall to the east as shown in the photographs in Figures 4 and 5 respectively. On the north side of the building, cracks on the ground parallel to the building and vertical gap at plinth level were observed. The photograph in Figure 6 is a closer view showing a large vertical gap under the north corridor and the crack parallel to the building about 5m in front. The interval of comparatively large cracks along the longitudinal direction of the building can be estimated to about 5 to 7m.

Figure 4. Photograph showing cracking at parking lot and fence parapet on the west side.

Figure 5. Photograph showing cracking on the quay wall to the east of the building.

From the basic observations described above concerning the behavior of building and ground, authors have attempted to develop a simple analytical technique to depict the occurrence mechanism by combining the ideas of Tazo et al. (1999) and Nakazawa et al. (1996). That is, in consideration of the development of slip surface as illustrated in Figure 7(a), the ground in front of the piles recedes towards the sea caused by movement quay wall. As a result, an unbalance of earth pressure between the front and the back of the pile
group occurs. It can be considered to resemble the situation of excavation in the front such that load distribution regarded as active earth pressure from the back may be assumed. However, the lateral pressure was considered instead of the active earth pressure, because the ground would have been in fluid state making it difficult to distinguish earth pressure from hydraulic pressure, and a coefficient of lateral pressure of 0.7 was assumed.

**Figure 6.** Photograph as seen from the north side of block A, where gap developed under the corridor and the crack parallel to the building about 5m in front may be noticed.

**Figure 7.** (a) Slip surface and displaced shape of the damaged pile based on the investigation by Nakazawa et al. (1996 & 1999) and (b) hypothetical load distribution considered in the analysis.

Next, as a result of the development of slip surface in the ground behind pile group, certain addition demand on pile resistance may be expected. It is regarded as the slip force in this paper and represented by a concentrated load located at a depth where the slip surface interfaces the pile group. Based on the investigation of Nakazawa et al. (1999) as illustrated in Figure 7(a), the slip surface along the pile groups to the south occurs at a depth of about 4.1m below the ground surface. Accordingly, the slip force is assumed to
act behind the pile at a depth of 4.1m as shown in Figure 7(b). The magnitude of the slip force was estimated from the condition that the slippage occurs when the frictional resistance limit of slip surface is reached. Considering the possibility of slippage as a block of ground, three different slip lengths of 2m, 4m and 6.7m, with longest one corresponding to half width of the building, were considered for comparative analysis.

APPLICATION OF THE LOAD DISTRIBUTION METHOD

Authors have proposed and elaborated over the years the concept of load distribution method as a way of accounting for the response displacement effect in piles. They have also discussed comparison of such simple analytical results with more sophisticated analysis consisting of one and two dimensional time history analysis in accounting for the effects of ground response on stresses developed in piles during earthquakes (Sugimura et al, 1997, Karkee et al, 1999). Load distribution method is based on the fundamental equation of response displacement method as follows.

\[ EI \frac{d^4 y}{dx^4} + k_h B \{ y - f(x) \} = 0, \]  

(1)

The notations used in equation 1 are as follows:

- \( EI \): Bending rigidity of pile
- \( k_h \): Coefficient of subgrade reaction
- \( B \): Breadth of pile
- \( y \): Horizontal displacement of pile
- \( f(x) \): Horizontal displacement of the ground
- \( x \): Depth

If we put \( f(x) = 0 \) in equation 1, it becomes simply the ordinary fundamental equation representing horizontal resistance of pile subjected horizontal force at the top. Transforming equation 1, we have:

\[ EI \frac{d^4 y}{dx^4} + k_h By = k_h B f(x), \]  

(2)

Because the term in the right hand side can be regarded as apparent external force, replacing \( k_h B f(x) \) with an arbitrary external force distribution \( p(x) \), we have:

\[ EI \frac{d^4 y}{dx^4} + k_h By = p(x), \]  

(3)
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$k_a$: Coefficient of subgrade reaction

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While the external force distribution restricted to $k_a Bf(x)$ in equation 2, the term $p(x)$ in equation 3 may be selected arbitrarily, such that any general load condition, such as shown in Figure 7(b), may be considered in arriving at the solution.
Assuming the pile to have infinite length, general solutions for displacement or stresses for predefined load distributions, such as concentrated load, uniform load, triangular load, can be readily obtained from Hetenyi (1946) as shown in Table 1. It may be noted from the table that when the solutions for the cases of triangular and inverse triangular load distributions are added, the result logically corresponds to the solution for uniform load. Authors have also utilized this approach and prior examples of application can be found in Sugimura and Karkee (2001) and Karkee and Sugimura (2002). In order to consider the situation of the nonexistence of subgrade reaction at the upper part above the slip surface, as may be considered to be the actual condition, the effect of drastic reduction in the coefficient of subgrade reaction above the slip surface was evaluated.

ANALYTICAL EXAMPLE

Nakazawa et al. (1999) have provided details of the building and foundation system. According to them, the building consisted of 11 storied SRC structure with span between columns in longitudinal direction of 6.3m. Accordingly, the width of the slip surface corresponding to a pile group was considered to be 6.3m. The foundation beams were 0.4m in width and 1.8m in depth, 1.5m of which is embedded. There were six prestressed concrete (PC) piles of grade A (effective prestress $\sigma_e=3.92\text{N/mm}^2$) in the group supporting the target foundation slab of thickness equal to depth of foundation beam. The PC piles had outer diameter of 50cm, wall thickness of 9cm and length of 26m consisting of two 13m segments joined together. The design bearing capacity of single piles was 764kN.

The reclaimed layer (fill material) in Figure 7 consists of sandy material containing some gravel, the unit weight of which is assumed as $\gamma=18\text{kN/m}^3$. Based on the available information, the average value of standard penetration test blow count $N$ is estimated to be about 15. The shear strength $s$ of the fill material may then be evaluated as 50kN/m$^2$ based on the empirical relation in Japanese practice of $s=10N/3$ (kN/m$^2$). As mentioned above, the shear resistance of sandy material may be considered as resisting slip, and when the slip occurs slip force equivalent to the frictional resistance over the slip surface may be regarded to have mobilized. Based on AIJ (2001), the coefficient of subgrade reaction may be estimated from the following equation:

$$k_{h0} = \alpha \xi E_0 B^{-3/4}$$

(4)

Here,

$k_{h0} = \text{Normalized coefficient of subgrade reaction (kN/m}^3\text{)}$
\( E_0 = \) Modulus of deformation (kN/m\(^2\)), which for the N-value works out to be
\( E_0 = 700 \) (the coefficient \( \alpha = 80 \) in this case)
\( \xi = \) Influence factor for pile group = \(0.15\kappa + 0.10\) for the case \( \kappa \leq 6 \)
\( \kappa = \) Ratio of center to center interval of piles to pile diameter (2.5 in this case, such that \( \xi = 0.475 \))
\( B = \) Non-dimensional coefficient of pile diameter (50 in this case)

Substituting appropriate values in equation 4, the value of \( k_{s0} = 2.122 \times 10^4 \) (kN/m\(^3\)) was obtained. Calculation of response under trapezoidal lateral and the concentrated force at the slip surface acting on the upper part of pile as shown in Figure 7 can be undertaken. The trapezoid load can be divided into uniform and triangular distributions as illustrated in Figure 7(b). Displacement and stresses in pile were calculated at the 86 points from pile top at GL-1.5m to the depth of GL-10m at 0.1m interval and the results are shown in Figures 8 and 9.

**DISCUSSION OF THE ANALYTICAL RESULTS**

Analytical results shown in Figure 8 include displacement, bending moment and shear force in piles for various lengths of slip surface, where the concentrated load \( P \) of 105.0kN, 210.0kN and 351.8kN correspond to slip surface lengths of 2m, 4m and 6.7m respectively. Similarly, Figure 9 shows the case of \( P=351.8kN \) in Figure 7(b) with the coefficient of subgrade reaction reduced to 10\% and 1\% of the normal value obtained from equation 4 for the portion of fill material above the slip surface. As mentioned above, the drastic reduction attempts provide analogy to the case of practically nonexistent subgrade reaction due to liquefaction.

It may be noted that all the Figures 8 and 9 show the respective response curves corresponding to concentrated load, distributed load, and triangular load, separately as well as the synthesis (total) of those denoted by the thicker solid line. For comparison, the ultimate bending resistance \( M_u \) (kNm) and ultimate shear resistance \( Q_u \) (kN) of the pile member is shown by vertical dashed lines in the respective diagrams in Figures 8 and 9. Ultimate bending moment \( M_u \) was determined from the relation of the ultimate axial force and bending moment (\( M-N \) interaction curve) corresponding to the design bearing capacity of piles, which is 764kN as noted above. \( Q_u \) was determined by comparing the results by equation 5 (AIJ, 1990) and equation 6 proposed by Kishida et al. (2000).
\[
Q_u = \frac{2tI}{S_0} \frac{1}{2} \sqrt{(\sigma_g + 2\phi \sigma_s)^2 - \sigma_g^2} - \sigma_g^2
\]

\[
\sigma_g = \sigma_c + \frac{N}{A_c}
\]

\[\text{(5)}\]

**Figure 8.** Variation of the extent of pile response obtained from the load distribution method depending on changes in length of slip surface, and hence the magnitude of concentrated load P.

In equation 5, the notations are as follows:

- \( t \) = Thickness of the hollow circular pile section
- \( I \) = Second moment of area of the section
- \( S_0 \) = First moment of area of the section
$\sigma_e =$ Effective prestress

$\sigma_t =$ Tensile strength of concrete (about equal to 7% of compressive strength)

$N =$ Axial load

$A_e =$ Equivalent transformed section area of concrete

$\phi =$ Correction coefficient corresponding to experimental results (usually taken as 0.5)

$Q_u = \tau_{\text{max}} b_c j$  \hspace{1cm} (6)

In equation 6, the notations are as follows:

$b_c = \{-1.24(t/D) + 1.19\} A_e / D$

$j = \frac{7}{8} d = \frac{7}{8} \left( D - \frac{t}{2} \right)$

$\tau_{\text{max}} = \tau_1 + \tau_2 + \tau_3$  \hspace{1cm} (For design of pile members)

$\tau_1 = \frac{0.115k_w k_p (\sigma_b + 17.7)}{(M/Qd) + 0.115}$

Figure 9. The effect of drastic reduction in coefficient of subgrade reaction above the slip surface
\[ \tau_2 = 0.657 \left( 0.785 p_{w - w} \sigma_y \right) \quad \left( 0.785 p_{w - w} \sigma_y \leq 7.4 \right) \text{ (N/mm}^2) \]

\[ \tau_3 = 0.102 \left( \sigma_e + \sigma'_e \right) = 0.102 \left\{ \sigma_e + \frac{N'}{b_e j} \right\} \quad \left( \sigma_e + \sigma'_e \leq 27.4 \right) \text{ (N/mm}^2) \]

Where,

- \( D \) = Outer diameter of pile (mm)
- \( A_e = \pi (D - t) \) = Section area of pile (mm\(^2\))
- \( \tau_1 \) = Shear stress shared by concrete (N/mm\(^2\))
- \( \tau_2 \) = Shear stress shared by spiral hoop (N/mm\(^2\))
- \( \tau_3 \) = Shear stress shared by axial force (N/mm\(^2\))
- \( \sigma_c \) = Compressive strength of concrete (N/mm\(^2\))
- \( N' \) = Axial force (N)
- \( d \) = Effective depth (mm)
- \( \sigma_y \) = Strength at yield point (N/mm\(^2\))
- \( k_u \) = Modification factor for section size (\( k_u = 0.72 \) in this case)
- \( k_p \) = Modification factor depending on tension reinforcement \( p_t \) as shown in the following equation,

\[ k_p = 0.82 \left( 100 p_t \right)^{0.23} \quad p_t = p_y / 4 \quad \text{(7)} \]

- \( p_g \) = Main reinforcement ratio (\( p_g = A_s / b_e j \))
- \( p_w \) = Spiral hoop ratio (\( p_w = a_w / b_e s \))
- \( A_s \) = Total area of main reinforcement (mm\(^2\))
- \( a_w \) = Area of spiral shear reinforcement (mm\(^2\))
- \( s \) = Interval of spiral hoop (mm)

\( M/Q_d \) = Shear span ratio (Since based on the experience of authors (Sugimuta et al., 1984), it is clarified that a shear span ratio of nearly equal 1.0 lies in the border between bending failure and shear failure of PHC piles type A, the value of 1.0 was used in the analysis.)
Considering the axial force to be equal to design bearing capacity of single pile (764kN), the value of $Q_u$ were obtained as 277kN and 340kN respectively from equation 5 and equation 6. The result seems to correspond well with the recent thinking that equation 5 agrees with shear crack capacity while equation 6 tends to conform to the ultimate shear capacity. Accordingly, equation 6 was adopted for estimation of $Q_u$ for the pile member.

Figure 8 shows the effect of concentrated load dominates the response of pile. As such the bending moment as well as the shear force at the slip surface at GL-4.1m increases with the length of slip surface. The ratio of the maximum load effect to the ultimate capacity is about 80% in bending moment and about 60% in shear force.

On the other hand, the situation is seen to change completely in Figure 9. When the coefficient of subgrade reaction is reduced to 10% for the case of slip surface length of 6.7m, the maximum shear force becomes a little larger and distribution above the slip surface changes a little, but the change is limited over a small range on the whole. Distribution of bending moment at the slip surface, however, becomes much larger and the maximum bending moment exceeds the ultimate capacity. And displacement also becomes discontinuous in distribution at the slip surface, which is extremely different from usual case. When the coefficient of subgrade reaction reduced further to 1%, these tendencies become more prominent.

Based on the analytical results, damage features observed in piles may be explained analogically as follows. The piles are subject to forced deformation, which tends to be discontinuous at the slip surface. Accordingly, not only the bending moment reaches easily the ultimate capacity but also the effect of direct shear failure contributes to overall response. The authors are aware that the several assumptions considered in the process of analysis by load distribution method may not provide a solid basis for quantitative aspects of the problem, such as the aspects of whether the stresses in pile exceed ultimate capacity or not. However, the authors wish to emphasis that the consideration of comparatively unique loading condition, such as the introduction of a combination of concentrated and distributed load discussed above, is effective and necessary to introduce in order to clarify the aspects of damage actually observed. Alternatively, while attempting to investigate pile behavior by response displacement method, it is necessary to consider the condition of deformation discontinuity at locations where a slip surface may be expected to develop in the ground under the action of severe earthquakes.
CONCLUSIONS AND FUTURE RESEARCH

Attempt was made to clarify the mechanism of K-shaped failure at deeper part of piles, observed during the 1995 Hyogoken-Nambu earthquake. This involved review of existing relevant reports and selection of a particular structure for analytical interpretation by applying the load distribution method proposed earlier by the authors. The analysis was based on the hypothesis that damage of the ground due to lateral flow resembles occurrence of slump in landslide. That is, considering the effect of unbalanced horizontal stress due to lateral flow of liquefied soil layer, while the horizontal earth pressure on one side of the pile is absent as if removed by excavation. The analysis is based on the assumption of an infinitely long pile subjected to a trapezoidal load representing lateral pressure and the concentrated load representing the action at the slip surface. The analysis also include the cases with drastically reduced coefficient of subgrade reaction for the part of soil above the slip surface as a way to analogically represent the nonexistence of subgrade reaction in liquefied soil.

The analytical results indicate general tendency that the pile would be subjected to displacement discontinuity at the slip surface. Bending moment easily reaches ultimate capacity of pile member, particularly at the slip surface. In addition, the effect of relatively large shear force is likely to contribute to failure. In conclusion, it may be emphasized that the consideration of a comparatively unique loading condition to represent ground condition and its possible failure mechanism is necessary to realistically represent K-shaped failure of piles. In addition, consideration of such unique loading condition is expected to be an effective approach in design practice.

In order to represent the actual condition more realistically, it is necessary to introduce the boundary condition at pile top, effect of axial force, reduction in strength of the ground due to liquefaction, the elasto-plastic theory or large deformation theory of piles etc. in the analysis. These aspects remain to be investigated and evaluated in future.

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(15)


