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SITE- AND INTERACTION- DEPENDENT STRENGTH REDUCTION FACTORS

Luis Eduardo PÉREZ-ROCHA[1] and Javier AVILÉS[2]

SUMMARY

Strength reduction factors have been extensively studied in the past for firm ground, and even for soft soils considering site effects, but always excluding soil-structure interaction. In this work they are investigated for a single elastoplastic structure with flexible foundation excited by vertically propagating shear waves. The concepts developed earlier for fixed-base yielding systems are extended to account for soil-structure interaction. This is done by use of the simplified reference model and a nonlinear replacement oscillator recently proposed by the authors. The latter is defined by an effective ductility together with the effective period and damping of the system for the elastic condition. Numerical evaluations are conducted for typical system configurations, using the great 9 October, 1995 Manzanillo earthquake recorded at the surface of a sand deposit in near field. Results are compared with those corresponding to the fixed-base case. Finally, it is shown how a site-dependent reduction rule proposed elsewhere for fixed-base systems should be adjusted for interacting systems using the information presented.

[1]Researcher, Centro de Investigación Sísmica, Carretera al Ajusco 203, Tlalpan 14200, México, D.F., México
[2]Researcher, Instituto Mexicano de Tecnología del Agua, Paseo Cuauhnahuac 8532, Jiutepec 62550, Morelos, México

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INTRODUCTION

To account that structures experience nonlinear response under strong earthquake motions, the current practice of earthquake resistant design is based on the use of strength reduction factors, R_μ, which relate the structural resistance for elastic behavior to that required for a given ductility. Following this well-established design approach, the yield resistance of nonlinear structures are estimated from the corresponding values of linear structures. Perhaps, the most widely accepted reduction rule for design is the one originally proposed by Veletsos and Newmark [1] and after improved by Newmark and Hall [2], based on the fact that the maximum elastic and inelastic displacements are equal in the long-period region of the response spectrum. As a consequence of site effects, the shape of R_μ for soft soil can be very different from that applicable to firm ground, depending essentially on the ratio between the fundamental period of the structure and the predominant period of the site (Miranda [3]. A recent work by Ordaz and Pérez-Rocha [4] presented a site-dependent reduction rule that is more general than others previously published. It can be applied to a wide variety of soft sites but, as other similar rules, does not account for soil-structure interaction. These authors have shown that, if the structure period is close to the site period, the value of R_μ may be significantly higher than that would be predicted by the Veletsos-Newmark design rule, equal to the structural ductility. For soft soils, a design reduction rule should include the effects of interaction, in addition to those induced by site conditions. Nevertheless, none of the R_μ factors developed so far explicitly account for such effects.

The aim of this paper is at extending the well-known concept of strength-reduction factor developed long ago for fixed-base systems in order to account for soil-structure interaction. To this end, an interaction model formed by a single yielding structure with embedded foundation in a soil layer over elastic bedrock is investigated. To easily assess the R_μ factor with interaction, the solution for a nonlinear replacement oscillator recently proposed by Avilés and Pérez-Rocha [5] is used. This equivalent oscillator is defined by an effective ductility together with the effective period and damping of the system for the elastic condition. These authors have demonstrated that modifying both the ductility factor as well as the relevant natural period and damping ratio of the fixed-base structure is a reliable way of expressing the interaction effects in nonlinear systems. By using this information, the site-dependent reduction rule developed by Ordaz and Pérez-Rocha [4] for fixed-base systems is suitably adjusted for interacting systems. This is a more rational way to assess in practice the yield resistance of flexibly supported structures. Conclusions from this study are expected to be applicable to more complex interacting systems.

DESCRIPTION OF MODEL AND BASIC EQUATIONS

The effects of kinematic and inertial interaction in nonlinear systems are evaluated by use of the relatively simple axisymmetric model shown in Fig. 1. The former effects are produced by the scattering and diffraction of the incident waves from the foundation, while the latter are caused by the inertial forces generated in the structure and foundation. This model is similar to that formulated by Avilés and Pérez-Rocha [5] to investigate the effects of interaction on the structural ductility. It consists of a single elastoplastic structure placed on a rigid foundation that is embedded in a homogeneous viscoelastic layer of constant thickness overlying a uniform viscoelastic half-space. The structure represents either a one-story building or, more generally, the first-mode approximation of a multi-story building. In the latter case, the parameters H_e and M_e are the effective height and mass of the structure when vibrating in its rigid-base fundamental mode. The natural period and damping ratio of the structure for the elastic and rigid-base conditions are given by

$$T_e = 2\pi \sqrt{\frac{M_e}{K_e}} \tag{1}$$

$$\zeta_e = \frac{C_e}{2\sqrt{K_e M_e}} \tag{2}$$

where C_e and K_e are the viscous damping and initial stiffness of the structure when fixed at the base. The foundation is assumed perfectly bonded to the surrounding soil; it is defined by the radius r, depth of embedment D, mass M_c, and mass moment of inertia J_c about a horizontal centroidal axis at the base. The stratum of thickness H_s is characterized by the Poisson's ratio ν_s, mass density ρ_s, shear wave velocity β_s, and hysteretic damping ratio ζ_s. Similarly, the corresponding material properties of the underlying half-space are defined by ν_o, ρ_o, β_o and ζ_o.

The interacting system is excited by vertically incident shear waves with particle motion parallel to the *x*-axis, as illustrated in Fig. 1. The horizontal free-field displacement at the ground surface is denoted by U_g. In view of the characteristics of this wave excitation, the input motion for the foundation consists of the horizontal component U_o at the center of the base and the rocking component Φ_o about the *y*-axis. The response of the building is described by the relative horizontal displacement U_e at the center of the deck, whereas the response of the foundation is described by the horizontal displacement U_c at the center of the base and the rocking Φ_c about the *y*-axis, both measured with respect to the corresponding horizontal and rocking input motions. Providing the foundation is rigid and the soil behaves linearly, the simplified analysis of interaction can be performed in three steps as follows: (1) elastic determination of the motion of the massless foundation when subjected to the seismic excitation, resulting in the horizontal and rocking input motions at the base; (2) elastic determination of the springs and dampers by which the supporting soil is replaced for the horizontal, rocking and coupling modes of the massless foundation; and (3) non-linear analysis of the structure, including the mass of the foundation, supported on the springs and dampers of step 2 and excited by the input motions of step 1. In accordance with this substructure technique, the governing equations of motion in the time domain for the coupled soil-structure system are given by

$$\begin{bmatrix} M_e & M_e & M_e(H_e+D) \\ M_e & M_e+M_c & M_e(H_e+D)+M_cD/2 \\ M_e(H_e+D) & M_e(H_e+D)+M_cD/2 & M_e(H_e+D)^2+J_c \end{bmatrix} \begin{Bmatrix} \ddot{U}_e(t) \\ \ddot{U}_c(t) \\ \ddot{\Phi}_c(t) \end{Bmatrix} + \begin{Bmatrix} P_e(t) \\ P_s(t) \\ M_s(t) \end{Bmatrix} =$$

$$-\ddot{U}_o(t) \begin{Bmatrix} M_e \\ M_e+M_c \\ M_e(H_e+D)+M_cD/2 \end{Bmatrix} - \ddot{\Phi}_o(t) \begin{Bmatrix} M_e(H_e+D) \\ M_e(H_e+D)+M_cD/2 \\ M_e(H_e+D)^2+J_c \end{Bmatrix} \tag{3}$$

where the overdot denotes differentiation with respect to time t. Also, $P_e(t) = C_e \dot{U}_e(t) + V_e(t)$ is the internal force of the structure, $V_e(t)$ being the restoring force. The interaction force $P_s(t)$ and moment $M_s(t)$ of the soil acting on the foundation are defined by a convolution integral discussed by Avilés and Pérez-Rocha [5]. In this work, the authors explain their scheme of solution of the equation system given in (3).

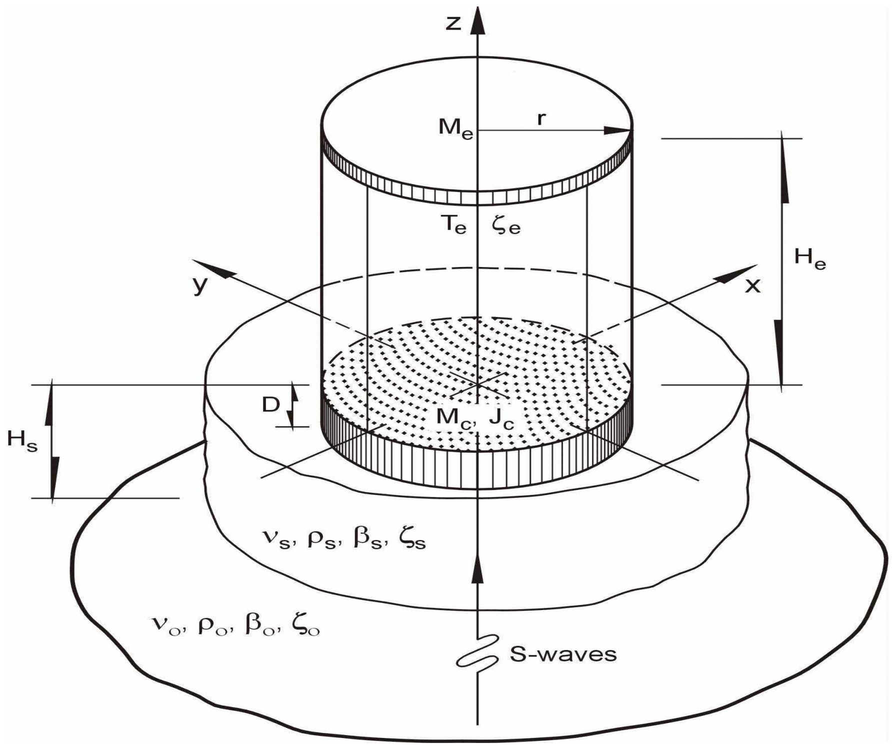

Fig. 1. Single nonlinear structure placed on a rigid foundation that is embedded in a stratum overlying a half-space, under vertically propagating shear waves.

NONLINEAR REPLACEMENT OSCILLATOR

Let us call $\tilde{T}_e$ and $\tilde{\zeta}_e$ to the effective period and damping of the system. They 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. The mass of this equivalent oscillator is taken to be equal to that of the actual 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. Introducing some permissible simplifications, Avilés and Suárez [6] have deduced the following expressions:

$$\tilde{T}_e = \left(T_e^2 + T_h^2 + T_r^2\right)^{1/2} \tag{4}$$

$$\tilde{\zeta}_e = \frac{1}{\left|Q_h + (H_e + D)Q_r\right|}\left(\zeta_e \frac{T_e^3}{\tilde{T}_e^3} + \frac{\zeta_h}{1+2\zeta_h^2}\frac{T_h^2}{\tilde{T}_e^2} + \frac{\zeta_r}{1+2\zeta_r^2}\frac{T_r^2}{\tilde{T}_e^2}\right) \tag{5}$$

where $T_h = 2\pi(M_e/K_{hh})^{1/2}$ and $T_r = 2\pi(M_e(H_e + D)^2/K_{rr})^{1/2}$ are the natural periods if the structure were rigid and its base were only able either to translate or to rock, and $\zeta_h = \tilde{\omega}_e C_{hh}/2K_{hh}$ and $\zeta_r = \tilde{\omega}_e C_{rr}/2K_{rr}$ 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 effective frequency of the system, $\tilde{\omega}_e = 2\pi/\tilde{T}_e$, an iterative process is required for calculating the system period from (4). Once this is done, the system damping is directly calculated also from (5). It should be mentioned that the factor $\left|Q_h + (H_e + D)Q_r\right|$ 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.

To account for the inelastic interaction effects, an equivalent ductility factor that fully characterizes the replacement oscillator 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. By equating the yield strengths and maximum plastic deformations developed in both systems under monotonic loading, it has been found that (Avilés and Pérez-Rocha, [5])

$$\tilde{\mu}_e = 1 + (\mu_e - 1)\frac{T_e^2}{\tilde{T}_e^2} \tag{6}$$

Note that the values of $\tilde{\mu}_e$ vary from 1 to μ_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 μ_e for infinitely-rigid soil (for which $\tilde{T}_e = T_e$) and to unity for infinitely-flexible soil (for which $\tilde{T}_e = \infty$). It has been demonstrated by Avilés and Pérez-Rocha [5] that, under seismic excitation, the yield strength of the replacement oscillator for the effective ductility $\tilde{\mu}_e$ remains in satisfactory agreement with that required by the interacting system for the allowable ductility μ_e.

STRENGTH-REDUCTION FACTOR

Contemporary design criteria admit the use of strength reduction factors to account for the nonlinear structural behavior. It is indeed common practice to make use of these factors for estimating inelastic design spectra from reducing elastic design spectra. For the interacting system subjected to a given earthquake, let us call the strength reduction factor, $R_{\mu-\beta}$, to the ratio between the strength required to have elastic behavior, $V_m(1)$, and the strength for which the ductility demand equals the target ductility, $V_y(\mu_e)$:

$$R_{\mu-\beta}(T_e) = \frac{V_m(1)}{V_y(\mu_e)} \tag{7}$$

It should be noted that this factor depends not only on the structural period T_e, but also on the ductility factor μ_e and the soil flexibility measured by the shear wave velocity β_s. To a lesser degree, this factor is also influenced by the structural damping ζ_e. It is evident that determination of $R_{\mu-\beta}$ allows estimation of inelastic strength starting from their elastic counterpart. Next we are to show the extent to which the $R_{\mu-\beta}$ factor are influenced by soil-structure interaction.

RESULTS

A free-field control motion, defined by the 9 October, 1995 Manzanillo earthquake recorded at the surface of a sand deposit in near field, was used for computations. In left side of Fig. 2 are depicted the normalized strength ($V_y/M_e g$) spectra for constant ductility ($\mu_e = 1$, 2 and 4) and 5% of critical damping, at the exclusion of soil-structure interaction. Here g is the acceleration of gravity. Also shown in this figure are the corresponding strength-reduction ($R_{\mu-\infty}$) factor. According to the one-dimensional wave propagation theory, the predominant period of the site is given by $T_s = 4H_s/\beta_s$, where $H_s = 15$ *m* and $\beta_s = 80$ *m/s* for the site considered. So, we have that $T_s = 0.75$ *s*, equal to the second resonant period observed at the elastic acceleration spectrum. The first one is related to the source in near field. It can be seen that the values of $R_{\mu-\infty}$ are close to μ_e, the value predicted by Veletsos and Newmark's rule. To show the influence of foundation flexibility on the $R_{\mu-\beta}$ factor, the $\beta_s = 80$ *m/s* shear wave velocity value was considered. For a representative story height of 3.6 *m*, the ratio H_e/T_e is approximately equal to 25 *m/s*, assuming the effective height as 0.7 of the total height and the translational period as 0.1 *s* of the number of stories. By considering this empirical relationship, the relative stiffness of the structure and soil takes values within the range $0.1 \le H_e/\beta_s T_e \le 0.5$. Note that for a given value of T_e, the value of H_e is obtained from the constant ratio H_e/T_e. With this, the foundation radio is determined from a fixed value of the ratio $\delta_r = H_e/r = 3$, and then the foundation embedment is determined from a fixed value of the ratio $\delta_d = D/r = 0.5$. This implies that the structure considered changes in height as a function of the period but has a constant slenderness ratio. At the same time, the foundation dimensions vary when the structure height changes, as happens with many types of buildings. The remaining system parameters were fixed constant at typical values for building structures: $M_c/M_e = 0.25$, $J_c/M_e(H_e + D)^2 = 0.05$ and $M_e/\rho_s \pi r^2 H_e = 0.15$. In right side of Fig. 2 are depicted the normalized strength spectra for constant ductility ($\mu_e = 1$, 2 and 4) and 5% of critical damping, and the corresponding strength-reduction factor, for this interaction scenario. The validity of (4), (5) and (6) is also verified. It is clear that the strength spectra obtained for the interacting system are well predicted by using the replacement oscillator.

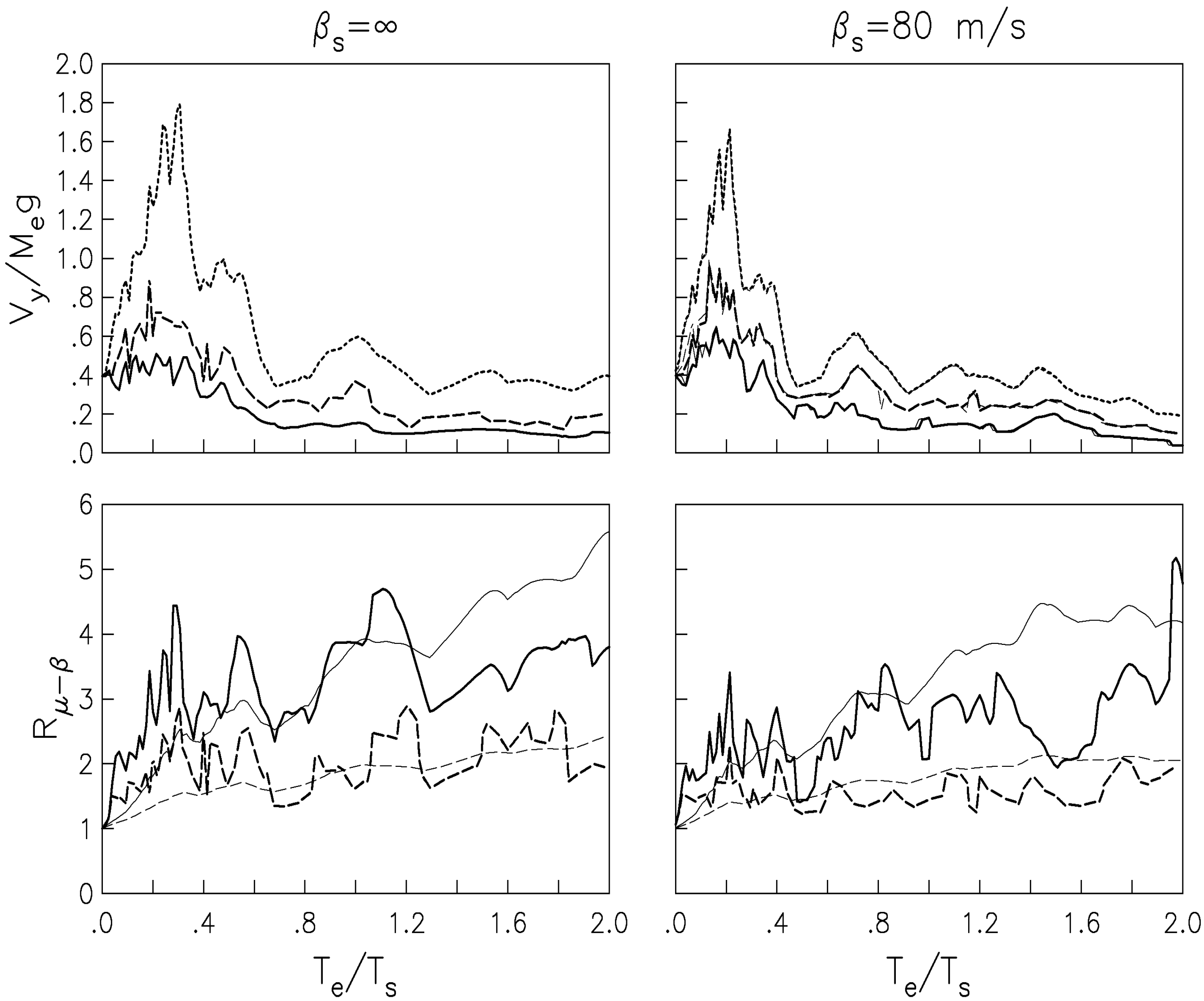

Fig. 2. Top: normalized strength spectra without (left) and with (right) interaction for $\mu_e = 1$ (dotted line), 2 (dashed line) and 4 (solid line), considering a 9 October, 1995 Manzanillo earthquake near field record. Exact solution for the interacting system (thick line) and approximate solution for the replacement oscillator (thin line). Bottom: Comparisons of real strength-reduction factors (thick line) for $\mu_e = 2$ (dashed line) and 4 (solid line) with those obtained by the proposed reduction rule (thin lines) for $\mu_e = 2$ (dashed line) and 4 (solid line). Results on right column correspond to an interacting system with $H_e/r = 3$, $D/r = 0.5$ and $H_s/r = 3$.

DESIGN REDUCTION RULE

Ordaz and Pérez-Rocha [4] observed that, for a wide variety of soft sites, the shape of $R_{\mu-\infty}$ depends on the ratio between the elastic displacement spectrum and the peak ground displacement as:

$$R_{\mu-\infty} = 1 + (\mu_e - 1)\left(\frac{U_m(T_e, \zeta_e)}{U_g}\right)^{\alpha} \tag{8}$$

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 (8) can be larger than μ_e, which indeed occurs if $U_m > U_g$. Following the replacement oscillator approach, this reduction rule may be readily implemented for elastically supported structures by merely replacing in (8) the relationships $\mu_e - 1 = (\tilde{\mu}_e - 1)\tilde{T}_e^2 / T_e^2$ and $U_m = (T_e^2 / \tilde{T}_e^2)\tilde{U}_m$, with which we have:

$$R_{\mu-\beta} = 1 + (\tilde{\mu}_e - 1)\frac{\tilde{T}_e}{T_e}\left(\frac{\tilde{U}_m(\tilde{T}_e, \tilde{\zeta}_e)}{U_g}\right)^{\alpha} \tag{9}$$

It should be pointed out that (8) will yield the same result as (9) if the elastic displacement spectrum without interaction is replaced by that with interaction. The two spectra $U_m(T_e, \zeta_e)$ and $\tilde{U}_m(\tilde{T}_e, \tilde{\zeta}_e)$ are used to emphasize the fact that the former corresponds to the actual structure, whereas the latter to the replacement oscillator. Comparisons are made in Fig. 2 (bottom) between real strength-reduction factor and that obtained with the proposed reduction rule, for $\mu_e = 2$ and 4. It is seen that, although the representation is not perfect, the approximate rule reproduces satisfactorily the tendencies observed in reality. In view of the many uncertainties involved in the definition of $R_{\mu-\beta}$, it is judged that such an approximation is appropriate for design purposes. The differences between the results with and without interaction are noticeable, specially for $\mu_e = 4$. It is apparent that structures on soft soil designed assuming rigid base may experience significant changes in their intended strength if soil-structure interaction plays an important role. Note that, as required by structural dynamics, $R_{\mu-\beta} = 1$ for $T_e = 0$ and $R_{\mu-\beta} \to \mu_e$ as $T_e \to \infty$, irrespective of the foundation flexibility. For other natural periods, there are no theoretical indications regarding the values of this factor. The steps involved in the application of (9) can be summarized as follows:

1. By use of (4), (5) and (6), compute the modified period $\tilde{T}_e$, damping $\tilde{\zeta}_e$ and ductility $\tilde{\mu}_e$ of the structure whose rigid-base properties T_e, ζ_e and μ_e are known.

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

3. The value of $R_{\mu-\beta}$ is then estimated by application of (9), provided the peak ground displacement U_g is known.

CONCLUSIONS

The influence of foundation flexibility on strength-reduction factor has been investigated, using a simplified reference model representative of code-designed buildings. Results were given for an earthquake characteristics and geotechnical conditions prevailing Mexican subduction zone. It has been found that the shape of this factor is primarily a function of the period ratio of the structure and site. This is in agreement with earlier findings by other authors for the fixed-base case. The main differences between the factors with and without interaction arise when the structure period is close to the site period. Furthermore, the site effects observed for the rigid-base condition tend to be canceled by soil-structure interaction. Therefore, the use of factors derived assuming rigid base may lead to strength considerably different from that actually developed in structures with flexible foundation.

Based on the solution for a nonlinear replacement oscillator, an available site-dependent reduction rule has been adjusted to include soil-structure interaction. As a result, a period-, damping- and ductility-dependent rule was implemented, which permits the use of standard free-field elastic spectra. The efficiency of this approximation was validated by comparison with results obtained rigorously. The new rule should be useful to assess, in the context of code design of buildings, the yield resistance of flexible-base inelastic structures from the corresponding values of rigid-base elastic structures. There is still a practical necessity of modifying this rule to take into account the multi-degree-of-freedom effects and the uncertainties involved in real buildings.

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