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STRENGHT REDUCTION FACTORS IN SOIL-STRUCTURE INTERACTION: DAMAGE ANALYSIS

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ABSTRACT :
Strength-reduction factors have been studied for firm ground and soft soils considering site effects. Soil-structure interaction has been recently accounted for by the authors of this work. One of these factors was investigated for a single elastoplastic structure with a flexible foundation excited by vertically propagating shear waves. The concepts for fixed-base yielding systems were extended to account for soil–structure interaction by using the simplified reference model and a nonlinear replacement oscillator proposed by the writers. This work is focused on a simplified procedure for practical damage analysis of structures considering the soil-structure interaction effects. A damage model based on maximum displacement and dissipated energy under monotonic loading is adopted, with the effects of cyclic load reversals being estimated by using a modified Park-Ang index. To simplify the consideration of the soil-structure interaction effects, an equivalent fixed-base oscillator with the same yield strength and energy dissipation capacity as the actual flexible-base structure is applied. Numerical results are presented in terms of dimensionless parameters for their general application, using a set of appropriate earthquake motions to ensure generality of conclusions. The significance of soil-structure interaction in the structural performance is elucidated and the adequacy of the approach proposed is examined

KEYWORDS:
Soil-structure interaction, Damage analysis, strength reduction factor

1. INTRODUCTION

It is a practice to account for nonlinear seismic response of building structures by using strength reduction factors. These factors are the ratio between the structural resistance for elastic behavior and the one required for a given ductility. The most accepted reduction rule is due to Veletsos and Newmark (1960) that was improved by Newmark and Hall (1973) under the assumption that the peak elastic and inelastic displacements are equal in the long-period range. Regarding firm ground and soft soil, many efforts have been made to develop several rules that relate the yielding strength of nonlinear models with the corresponding linear elastic one (Miranda and Bertero, 1994; Miranda and Ruiz-García, 2002). These strength ratios strongly depend on site effects, especially on the ratio between the fundamental period of the structure and the dominant period of the site (Miranda, 1993). There is a site-dependent reduction rule that is more general to be applied to a wide variety of soil conditions (Ordaz and Pérez-Rocha, 1998). According to this rule, when the structure period is close to the site period, the ratio can be significantly higher than the value predicted by the Veletsos-Newmark rule, which is equal to the structural ductility. A work by Avilés and Pérez-Rocha (2005) presented a site-dependent rule that accounts for soil-structure interaction effects. This rule was developed by using a nonlinear replacement oscillator which accounts for a single elastoplastic structure with a flexible foundation excited by shear waves. Typical site effects observed for the rigid-base condition are affected by soil-structure interaction.

In the performance-based seismic design it is not enough to account only for the strengths required for a given ductility. The level of structural damage has been recognized as a measure of cumulative dissipation of energy along with the maximum demand of plastic deformation. Several damage models have been developed to evaluate the structural performance assuming the structure as rigidly supported (Bozorgnia and Bertero, 2003). The most recognized involve the maximum and cumulative demand of plastic deformation without soil-structure interaction effects. According with Fajfar (1992) the structural performance depends not only on the maximum displacement demand, but also on the cumulative damage resulting from low-frequency fatigue, modeled through the plastic hysteretic energy. One of the most recognized damage models based on these two criteria, maximum displacement along with dissipated energy, was proposed by Park and Ang (1985). In spite of this model does not supply correct results when the structure remains elastic and when the ultimate displacement capacity under monotonically increasing load is reached, it has been widely applied in the engineering practice.

Soil-structure interaction has been recently accounted for in damage analysis by the authors of this work (Avilés and Pérez-Rocha, 2006). For this purpose, a consistent damage index, which is zero for incipient damage and unitary for potential structural collapse, was provided in a nonlinear replacement oscillator that represents the interacting system. The replacement oscillator is defined by the effective period and damping of the system and its global ductility (Avilés and Pérez-Rocha, 2003). Among other results, strength spectra for constant damage along with energy demand spectra are supplied. These results show that the approximated solution based on the replacement oscillator response is in excellent agreement with the exact solution. On the other hand, it can be pointed out that the yield strengths based on constant damage are greater than those for constant ductility, particularly at the resonance condition of the system. This effect, independent of soil flexibility, decreases as ductility decreases. The influence of soil-structure interaction on both, yield strengths and dissipated energy, is to increase or decrease the values with respect to the fixed-base condition, depending on the contrast between the structure period and the site period. A proper normalization shows that resulting energy is relatively stable in the whole period region and that its ductility dependence can be neglected. In this work, some results of Avilés and Pérez-Rocha (2006) related to strength spectra for constant damage and normalized energy spectra are taken in order to compute strength reduction factors for constant damage that account for soil-structure interaction.

2. SOIL-STRUCTURE SYSTEM

The soil-structure system considered consists of an elastoplastic one-story structure supported by a rigid foundation that is embedded in a viscoelastic layer overlying a homogeneous viscoelastic half-space. 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 condition are

$$T_e = 2\pi\sqrt{M_e/K_e} \tag{2.1}$$

$$\zeta_e = C_e \Big/ 2\sqrt{K_e M_e} \tag{2.2}$$

where C_e and K_e are, respectively, the viscous damping and initial stiffness of the structure when fixed at the base. Kinematic and inertial interactions are considered. The foundation is assumed as a circular mat of radius R and depth of embedment D. It has 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 ν_s, mass density ρ_s, shear wave velocity β_s and hysteretic damping factor ζ_s. The parameters ν_o, ρ_o, β_o and ζ_o are the corresponding properties of the half-space. The soil-structure system is subjected to vertically incident plane shear waves with particle motion in the horizontal plane. The horizontal displacement at the ground surface generated by the free-field motion is denoted by U_g. However, the presence of the foundation modifies the free-field ground motion. This results in a foundation input motion consisting of the horizontal and rocking components denoted by U_0 and F_0, 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_0, and the rocking of the foundation Φ_c relative to the rocking input motion Φ_0.

3. INELASTIC REPLACEMENT OSCILLATOR

The elastic interaction effects are normally expressed by changes in the natural period T_e and damping ratio ζ_e of the structure with rigid base. The resulting parameters $\tilde{T}_e$ and $\tilde{\zeta}_e$ are referred to as the effective period and effective damping ratio of the soil-structure system, respectively. They can be determined using an analogy between the coupled system excited by the foundation input motion and a replacement oscillator excited by the free-field motion on the ground surface. The effective period and damping of the system are obtained such that, under harmonic excitation, the resonant period and the peak restoring force of the actual system are equal to those of the equivalent oscillator. The formulation and validation of this procedure is published elsewhere (Avilés J, Pérez-Rocha, 1996).

According to Avilés and Pérez-Rocha (2003), the nonlinear response of the flexible-base structure is approximately equal to the one of an equivalent fixed-base oscillator characterized by its effective period and damping for the elastic condition and its effective ductility to account for non-linearities. Let U_y and $\tilde{U}_y$ denote the yield deformations of the actual structure and the replacement oscillator, respectively, whereas U_m and $\tilde{U}_m$ indicate the corresponding maximum deformation, and K_e and $\tilde{K}_e$ stand for the initial stiffness. Therefore, the ductility factors are defined in each case as $\mu_e = U_m / U_y$ and $\tilde{\mu}_e = \tilde{U}_m / \tilde{U}_y$. It is assumed that the mass is the same in both systems. Regarding that the yield strength of the equivalent oscillator is the same as that of the given structure, and taking into account eq (2.1), it follows that the yield deformations of both systems are related by

$$U_y = \tilde{U}_y T_e^2 / \tilde{T}_e^2 \tag{3.1}$$

On the other hand, the capacity of plastic deformation of both systems should be identical. Hence, by equating the plastic energy dissipation of both resisting elements, it is obtained that

$$\left(\tilde{U}_m - \tilde{U}_y\right)\tilde{R}_y = \left(U_m - U_y\right)R_y \tag{3.2}$$

where R_y and $\tilde{R}_y$ stand for the yielding strength of the actual structure and the replacement oscillator, respectively. Substituting eq (3.1) into eq (3.2), regarding that $\tilde{R}_y = R_y$ and using the definition of $\tilde{\mu}_e$ and μ_e, it is resulted that

$$\tilde{\mu}_e = 1 + (\mu_e - 1) T_e^2 / \tilde{T}_e^2 \tag{3.3}$$

Avilés and Pérez-Rocha (2003) have shown that this expression controls the nonlinear behavior of the replacement oscillator. According to eq (3.3), it is clear that $1 \le \tilde{\mu}_e \le \mu_e$, which implies 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 and equal to unity for infinitely flexible soil. In addition, both resisting elements would experience the same maximum plastic deformation, but different yield deformation due to the reduction from K_e to $\tilde{K}_e$. As the stiffness of the replacement oscillator represents the total stiffness of two springs in series simulating the flexibilities of the structure and foundation, part of its yield deformation is developed in the structural spring and the remainder in the soil spring. By substituting $U_y = U_m / \mu_e$ and $\tilde{U}_y = \tilde{U}_m / \tilde{\mu}_e$ into eq (3.1), it is found that the maximum deformation of the elastically supported structure, U_m, and the one of the replacement oscillator, $\tilde{U}_m$, are related by

$$U_m = (T_e^2 / \tilde{T}_e^2)(\mu_e / \tilde{\mu}_e)\tilde{U}_m \tag{3.4}$$

4. STRENGTH REDUCTION FACTOR

It is common in design criteria the use of strength reduction factors to account for the non-linear structural behavior. They come from the ratio between the strength required for elastic behavior, $S_a(1)$, and the one for which the ductility demand equals the target ductility, $S_a(\mu_e)$, that is

$$R_\mu(T_e, \beta_s) = S_a(1) / S_a(\mu) \tag{4.1}$$

The shape of the $R_\mu(\infty)$ factor, at the fixed-base condition, 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 soils, the strength reduction factor depends on the ratio between the elastic displacement spectrum, $S_d(T_e, \zeta_e)$, and the peak ground displacement, U_g^{max}, in the following way

$$R_\mu(\infty) = 1 + (\mu_e - 1)\left(S_d(T_e, \zeta_e) / U_g^{max}\right)^\alpha \tag{4.2}$$

where $\alpha \approx 0.5$. This expression has correct limits for very short period and long periods of vibration. Note that he values provided by eq (4.2) can be larger than μ_e, specially at the resonant condition. In this reduction rule, the period and damping dependence is properly controlled by the actual shape of the elastic displacement spectrum. The strength reduction factor (eq 4.1) depends on the natural period T_e the ductility factor μ_e, and the soil flexibility given by the shear wave velocity β_s. Avilés and Pérez-Rocha (2005) have proposed a smooth reduction rule based on a work of Ordaz and Pérez-Rocha (1998) and the replacement oscillator approach. This reduction rule may be obtained by replacing eq (3.3) and (3.4) in eq (4.2), with $\alpha \approx 0.5$, that is

$$R_\mu(\beta_s) = 1 + (\tilde{\mu}_e - 1)(\tilde{T}_e/T_e)\left(\tilde{S}_d(\tilde{T}_e,\tilde{\zeta}_e)/U_g^{max}\right)^{1/2} \tag{4.3}$$

Eq. (4.2) will yield the same result as Eq. (4.3) if the elastic displacement spectrum without SSI appearing in the former is replaced by the one with SSI. The two spectra $S_d(T_e,\zeta_e)$ and $\tilde{S}_d(\tilde{T}_e,\tilde{\zeta}_e)$ are used to refer to the actual structure and to the replacement oscillator, respectively. Avilés and Pérez-Rocha (2005) have shown that the use of the factor based on the later provides excellent results. A more accuracy results can be obtained if the reference displacement U_g^{max}, in the fixed-base condition, is the total displacement of the structure discounting the structure deformation. According to eq (3.1), for elastically supported structures, this new reference displacement becomes

$$\tilde{U}_g^{max} = U_g^{max}(\tilde{T}_e^2/T_e^2) \tag{4.4}$$

In view of the many uncertainties involved in the definition of this factor, it is judged that Eqs. (4.2) and (4.4) are appropriate for design purposes.

5. DISSIPATED HYSTERETIC ENERGY AND CONSISTENT DAMAGE INDEX

The number of inelastic cycles is reflected in the plastic hysteretic energy. Thus, this cumulative quantity is a reliable indicator for evaluating the damage potential of long-duration intense earthquakes. The plastic strain energy dissipated by the structure under earthquake excitation is computed as the cumulative area of the force-deformation hysteresis loops. For a general elastoplastic cycle, we have that

$$E_H = R_y(U_m - U_y) \tag{5.1}$$

where U_m is the inelastic displacement demand and $U_y = R_y/K_e$ the yield displacement. Considering that $R_y = \tilde{R}_y$, Avilés and Pérez-Rocha (2006) have shown that the dissipated energy is the same in the actual system and in the replacement oscillator, that is $E_H = \tilde{E}_H$, where the dissipated energy by the replacement oscillator is given by

$$\tilde{E}_H = \tilde{R}_y(\tilde{U}_m - \tilde{U}_y) \tag{5.2}$$

Avilés and Pérez-Rocha (2002) proposed a consistent damage index by making a simple adjustment to the Park-Ang index, which traduces in

$$DI = \left(\alpha(\mu_m - 1) + \beta E_H^n\right)/(\mu_u - 1) \tag{5.3}$$

here $\mu_m = U_m/U_y$ and $\mu_u = U_u/U_y$ are the ductility demand and the ductility capacity, whereas U_m is the maximum displacement demand during earthquake excitation and U_u is the ultimate displacement capacity under monotonic loading. At the same time, $E_H^n = E_H^m/F_yU_y$ is the so-called normalized hysteretic energy, where $E_H^m = F_y(U_m - U_y)$ is the hysteretic energy demand during earthquake excitation. The coefficients α and β, which satisfy the condition $\alpha + \beta = 1$, control the strength deterioration in terms of the maximum displacement and dissipated energy, respectively. They depend on the characteristics of both the structural system and earthquake excitation. It will be assumed that $\alpha = 0.8$ and $\beta = 0.2$. It should be noted that these rough values are used only to illustrate the implementation of the damage model. A rational procedure to estimate similar coefficients can be found in Bozorgnia and Bertero (2003) by correlations with the predicted values of Park-Ang index in its intermediate range. For the failure condition, where $DI = 1$, and considering that μ' is the equivalent ductility to account for the cumulative damage due to cyclic load reversals in the inelastic range, a relation between the ensuing reduced ductility and the ultimate monotonic ductility can be obtained from eq. (5.3), as follows

$$\mu'_m = \left(\mu_u - \beta[1 + E_H^n(1)]\right)/\alpha \tag{5.4}$$

6. NUMERICAL RESULTS

Throughout the paper, it will be assumed that $M_c/M_e = 0.2$, $J_c/M_e(H_e + D)^2 = 0.05$, $M_e/\rho_s \pi r^2 H_e = 0.15$, $\zeta_e = \zeta_s = 0.05$ and $\nu_s = 0.45$. These values are intended to approximate typical building-foundation-soil systems. In this work we use the relative stiffness of the structure and soil, $\sigma = H_e/T_e\upsilon_s$ for measuring the importance of SSI. If T_e is proportional to H_e, then σ measures purely the soil flexibility. Herein calculations were performed for $\sigma = 1/3$, corresponding to say $H_e/T_e = 25\,\mathrm{m/s}$ and $\upsilon_s = 75\,\mathrm{m/s}$. The former value is valid for many building-type structures and the latter for soft sites found in the lakebed zone of Mexico City. One typical interacting system with $H_e/r = 3$ for the slenderness ratio and $D/r = 0.5$ for the embedment ratio was examined. The next basic results were taken from Avilés and Pérez-Rocha (2006) in order to show the R_μ ratios of constant damage strength spectra.

To identify general tendencies of results, a statistical study was performed by using 90 horizontal ground motions recorded on 15 free-field stations in Mexico City from 3 distant subduction earthquakes, detected as far as 250-300 km away. The average response spectra with and without SSI obtained from these motions are depicted in figure 2, for elastic and inelastic behavior, in terms of spectral amplification. This results, depicted with thin lines, correspond to constant ductility $\mu_u = 2$ and 4. The averaging process was carried out by proper normalization of the dominant excitation period at each site (defined as the period where the 5% acceleration spectrum attains its maximum) for not eliminating the characteristic peaks typical of narrow-band response spectra (Mylonakis and Gazetas, 2000). Also amplitudes were scaled with the peak ground acceleration. The fixed-base elastic spectrum exhibits two resonant peaks, one at $T_e/T_s = 1$ associated with the first mode of vibration of the soil and other at $T_e/T_s \approx 0.35$ associated with the second mode. These peaks tend to disappear for inelastic spectra. The response spectra with SSI shift towards shorter periods, the consequences of which depend primarily on the period ratio T_e/T_s. Unlike what happens with the fixed-base case, resonance in SSI occurs for a structure period significantly shorter than the site period.

Strength spectra are determined by iteration on the yield resistance F_y until the μ_m and E_H^n demands satisfy equation (5.3), for given values of damage index corresponding to desired performance levels. For the failure limit state, $DI = 1$, average strength spectra are displayed in figure 2 with thick lines. Results are given with and without consideration of SSI and, in each case, the spectrum for constant damage is compared with that for constant ductility used in conventional design practice. The strengths demands based on constant damage are greater than those for constant ductility. This is to compensate for the cumulative damage due to multiple cyclic inelastic actions. The effects of structural damage on the strengths required for very short and long periods of vibration are negligible, particularly for the lower ductility. Conversely, the constant-damage and constant-ductility spectra separate greatly from each other at their resonant peaks. Figure 3 shows the elastic displacement spectra, without and with SSI. They are normalized with the peak ground displacement. The former tends to the peak ground displacement, the latter tends to this value affected by the factor $T_e^2/\tilde{T}_e^2$.

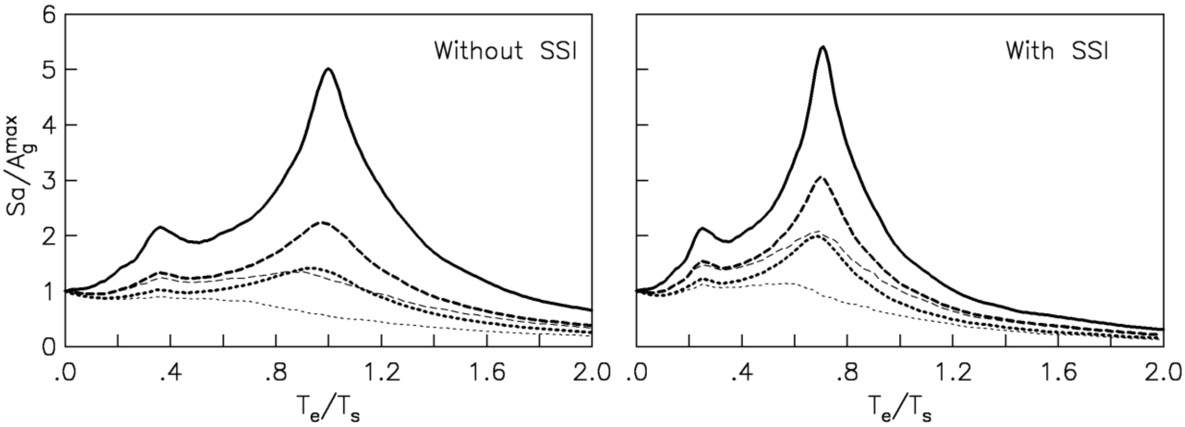

Figure 2. Strength spectra without (left part) and with (right part) soil-structure interaction. Thin and thick lines indicate constant ductility and constant damage (for ID=1) strength spectra. Dashed and dotted lines stand for $\mu_u = 2$ and 4, respectively. (After Avilés and Pérez-Rocha, 2006)

Strength-reduction factors were computed using the results given in figure 2. The shapes of R_μ are shown in figure 4. 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 demands if soil–structure interaction plays an important role. Note that, as required by structural dynamics, $R_\mu \to 1$ for $T_e = 0$ and, for the fixed-base condition, $R_\mu \to \mu_e$ as $T_e \to \infty$. For the elastic embedment condition, the ratios tend to this value affected by the factor $T_e^2/\tilde{T}_e^2$. In some period ranges, the values of these factors are larger for rigid- than for flexible-base structures, but in others are smaller. Despite this irregular behavior, one can conclude that site effects, reflected in that $R_\mu > \mu_e$ around the site period, are counteracted by soil–structure interaction. This means that, at extreme interaction conditions, it will have that $R_\mu = \mu_e(T_e^2/\tilde{T}_e^2)$ for medium and long natural periods. The reason for this is that, if the interaction effects were so large, the structure period would shift to the long-period spectral region, for which the equal displacement rule is applied. It should be noted that the R_μ factor is to be used in combination with flexible-base elastic spectra which, in turn, can be derived from rigid-base elastic spectra using the values of $\tilde{T}_e$ and $\tilde{\zeta}_e$ previously defined. By this way, the yield resistance and maximum deformation of interacting inelastic systems are estimated from the corresponding values of fixed-base elastic systems.

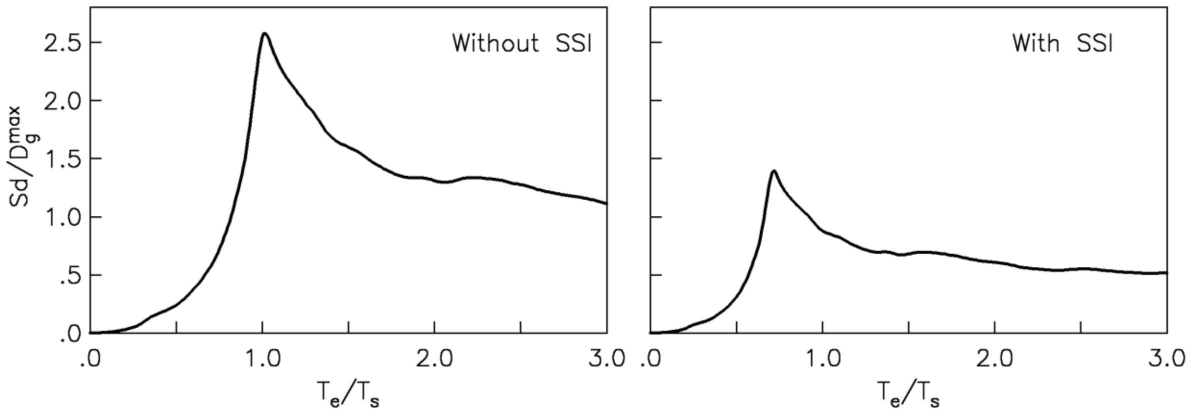

Figure 3. Elastic displacement spectra without (left part) and with (right part) soil-structure interaction.

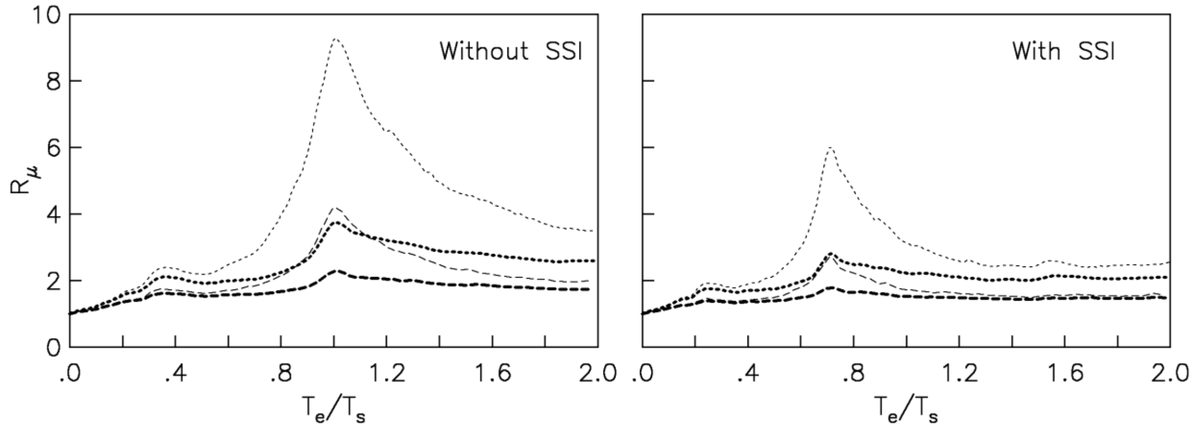

Figure 4. Variations against period of average strength-reduction factors without (left part) and with (right part) nteraction for $\mu_e = 2$ (dashed lines) and 4 (dotted lines). Thin and thick lines stand for constant ductility and constant damage computations, respectively. (After Avilés and Pérez-Rocha, 2006)

The mean value of $E_H^n(1)$ is needed to estimate the equivalent ductility for the failure condition. Figure 5 (left part) exhibits the average hysteretic energy spectra for $\mu_u = 2$ and 4, with and without regard to SSI. As $\mu_u - 1$ is the minimum value of $E_H^n(1)$ expected for very short period, the square root of the ratio $E_H^n(1)/(\mu_u - 1)$ is plotted for convenience. Regarding that $(E_H^n(1))^{1/2}$ is linearly proportional to the excitation amplitude, this quantity was in reality averaged in place of $E_H^n(1)$. We see that this re-normalized hysteretic energy varies relatively little in the whole period region, ranging from 1 at $T_e = 0$ to less than 2 at its maximum and approaching an intermediate value as $T_e \to \infty$. It is clear that ductility has little influence on results, which increase slightly with increasing its value. The energy curves for both ductilities are uniformly separated from one another, except for very short period. Equivalent ductility computed by using eqn (5.4) is shown in figure 5 (right part). It can be notice that the effective ductility is smaller that the ultimate ductility for all structural periods and that this ductility reaches its minimum value at the resonance condition.
Finally, the strength reduction factors for the failure condition showed in figure 4 (with and without SSI) are

compared with those computed using eqn (4.3), (4.4) and (5.4) in figure 6. It is seen that, although the representation is not perfect, the approximate rule satisfactorily reproduces the tendencies observed in reality. Note that these ratios tend to the equivalent ductility when SSI is disregarded and to the equivalent ductility affected by SSI when SSI is regarded. The following steps involved in the application of Eq. (4.3) can help to understand this concept:

1. By use of the expression proposed by Avilés and Pérez-Rocha, compute the modified period $\tilde{T}_e$ and damping $\tilde{\zeta}_e$. The rigid base properties of the structure are assumed as T_e, ζ_e and μ_e

2. Given the structural ductility at the fixed base condition μ_e, and the variation of the normalized hysteretic energy (figure 5, left part) compute the equivalent ductility to account for constant damage following eqn (5.4).

3. By using the equivalent ductility μ' in eqn (3.3) compute de effective ductility $\tilde{\mu}_e$

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

5. The value of R_μ is then estimated by application of Eq. (9), provided the peak ground displacement U_g^{max}, affected by the factor $\tilde{T}_e^2/T_e^2$, is known. For $\sigma = 1/3$, in the examined example, $\tilde{T}_e^2/T_e^2 \approx 2$ for all structural periods.

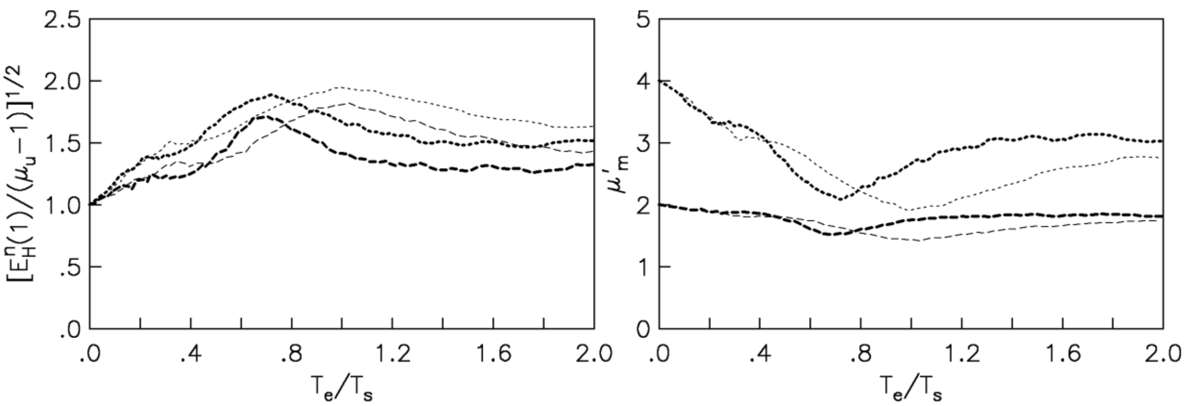

Figure 5. Average hysteretic energy spectra for the failure condition (left part) and equivalent ductility (right part) without (thin lines) and with (thick lines) soil-structure interaction, considering $\mu_u = 2$ (dashed line) and 4 (dotted line). (After Avilés and Pérez-Rocha, 2006)

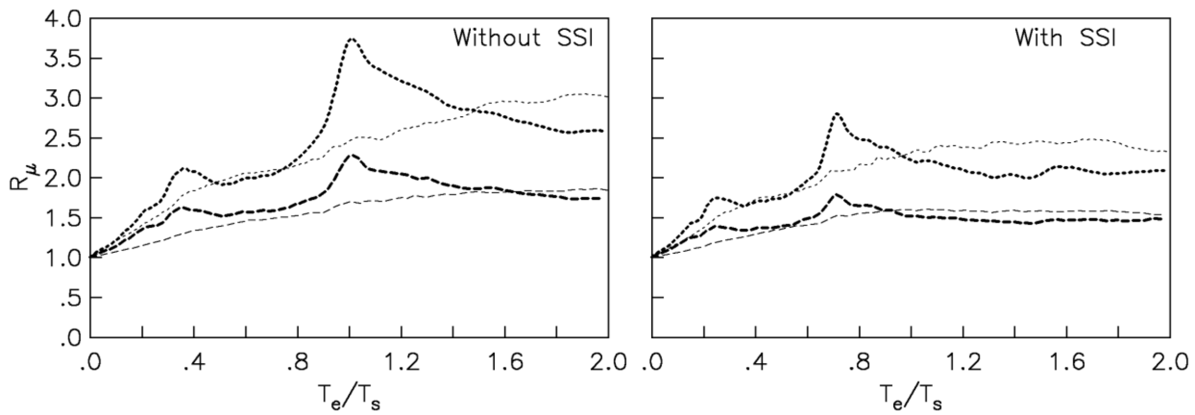

Figure 6. Comparisons of real strength-reduction factors (thick line) with those obtained by the proposed reduction rule (thin line) for μ_e =2 (dashed line) and 4 (dotted line).

7. CONCLUSIONS

A simplified energy-based approach for damage analysis of structures with flexible foundation has been reviewed. This approach is based on the solution of a non-linear replacement oscillator with the same yield strength and energy dissipation capacity as the actual structure. For evaluation of the structural performance, a consistent damage index that is zero for incipient damage and unity for potential collapse was introduced. A set of earthquake ground motions collected from Mexico City was used for calculations. Both strength and damage spectra were computed for flexible- and rigid-base conditions. Equivalent ductilities to account for low-cycle fatigue were also calculated. Based on these results, a simplified procedure for practical damage analysis of structures considering the soil-structure interaction effects, is used to compute this R_μ factor. It has been found that the shapes of these factors are primarily a function of the period ratio of the structure and site. This is in agreement with earlier findings by other writers 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 cancelled by soil–structure interaction. These results are strongly modified when constant damage is assumed. When constant damage is accounted for, the reduction factor tends to the well known rule due to Veletsos and Newmark. As a result, a period-damping-ductility-normalized energy-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 and maximum deformation of flexible-base inelastic structures from the corresponding values of rigid-base elastic structures.

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