PROPOSAL 3-119 (2009)

SCOPE: Part 2, Commentary Chapter 19

PROPOSAL FOR CHANGE:

Add Chapter 19 to Part 2, of the 2009 Commentary:

Proposed Chapter is attached. Text is not underlined to allow easier review.

REASON FOR PROPOSAL:

One of the basic tasks of the 2009 NEHRP *Provisions* update is to develop a viable commentary to Part 1. Since Part 1 adopts ASCE 7-05 and lists any exceptions to it, the Commentary is developed in accordance with the format and sections of ASCE 7-05.

TS 3 VOTE:

TS 3 developed this commentary chapter and approved for submission. The chapter was edited and accepted by TS 3 .

Chapter19

SOIL STRUCTURE INTERACTION FOR SEISMIC DESIGN

C19.1 GENERAL

The response of a structure to earthquake shaking is affected by interactions between three linked systems: the structure, the foundation, and the geologic media underlying and surrounding the foundation. A seismic Soil-Structure Interaction (SSI) analysis evaluates the collective response of these systems to a specified free-field ground motion. The term "free-field" refers to motions not affected by structural vibrations and represents the condition for which the design spectrum is derived using the procedures given in Chapter 11.

SSI effects are absent for the theoretical condition of rigid foundation and soil conditions. Accordingly, SSI effects reflect the differences between the actual response of the structure and the response for the theoretical, rigid base condition. Visualized within this context, three SSI effects can significantly affect the response of building structures:

1. <u>Foundation stiffness and damping</u>. Inertia developed in a vibrating structure gives rise to base shear, moment, and torsional excitation, and these loads in turn cause displacements and rotations of the foundation relative to the free field. These relative displacements and rotations are only possible because of compliance in the soil-foundation system, which can significantly contribute to the overall structural flexibility in some cases. Moreover, the relative foundation-free field motions give rise to energy dissipation via radiation damping (i.e., damping associated with wave propagation into the ground away from the foundation, which acts as the wave source) and hysteretic soil damping, and this energy dissipation can significantly affect the overall damping of the soil-foundation-structure system. Since these effects are rooted in the structural inertia, they are referred to as inertial interaction effects.
2. <u>Variations between free-field and foundation-level ground motions</u>. The differences between foundation and free-field motions result from two processes. The first is known as kinematic interaction and results from the presence of stiff foundation elements on or in soil, which cause foundation motions to deviate from free-field motion as a result of base slab averaging, wave scattering, and embedment effects. The second process is related to the structure and foundation inertia and consists of the relative foundation-free field displacements and rotations described above.
3. <u>Foundation deformations</u>. Flexural, axial, and shear deformations of foundation elements occur as a result of loads applied by the superstructure and the supporting soil medium. Such deformations represent the seismic demand for which foundation components should be designed. These deformations can also significantly affect the overall system behavior, especially with respect to damping.

Chapter 19 treats only the inertial interaction effects (the first item above). Inertial interaction in buildings tends to be important for stiff structural systems (such as shear walls and braced frames), particularly where the foundation soil is relatively soft (i.e., Site Classes C to E). Kinematic interaction effects are neglected in these provisions. Foundation design is covered in Section 12.13.

In design procedures that utilize response spectra to establish design values of base shear (i.e., force-based methods such as those given in Chapter 12), the effects of inertial SSI on the seismic response of buildings is represented as a function of the ratio of flexible- to fixed-base first-mode natural period,

$\tilde{T}_1 / T_1$, and system damping, β_0, attributable to foundation-soil interaction. The flexible-base first-mode damping ratio, $\tilde{\beta}$, is calculated using Eq. 19-9.

Figure C19-1 illustrates two possible effects of inertial SSI on the peak base shear, which is commonly computed from spectral acceleration at the first-mode. The spectral acceleration for a flexible-based structure ($\tilde{S}_a = \tilde{C}_s / g$) is obtained by entering the spectrum drawn for effective damping ratio, $\tilde{\beta}$, at the corresponding elongated period, $\tilde{T}$. For buildings with periods greater than about 0.5 s, using $\tilde{S}_a$ in lieu of S_a ($=C_s/g$) typically reduces base shear demand, whereas in very stiff structures SSI can increase the base shear. Most equivalent lateral force methods feature a flat spectral shape at low periods that, when coupled with the requirement that $\tilde{\beta} > \beta$, results in modeling of inertial SSI that can only decrease the base shear demand.

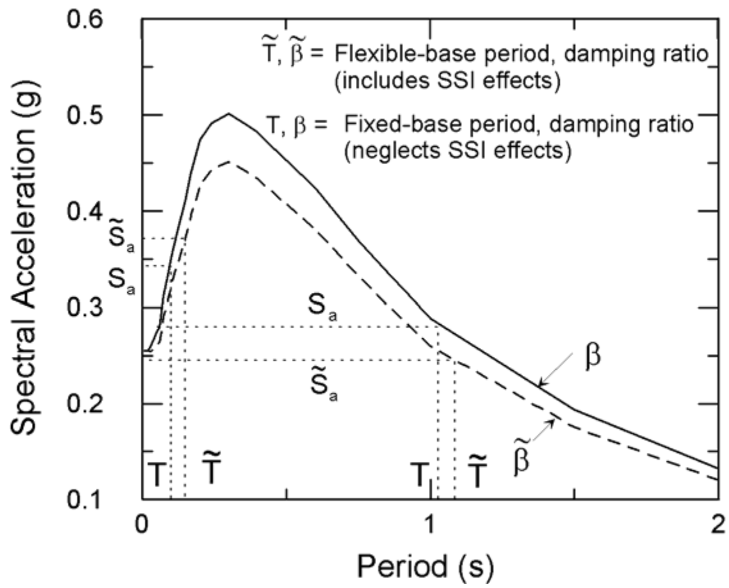

Figure C19-1 Schematic showing effects of period lengthening and foundation damping on design spectral accelerations

The method given in Chapter 19 for evaluating inertial SSI effects is optional and has rarely been used in practice. There are several reasons for this. First, because the guidelines were written such that base shear demand can only decrease through consideration of SSI, SSI effects are ignored in order to be conservative. Second, many design engineers who have attempted to apply the method on projects have done so for major, high-rise buildings for which they felt evaluating SSI effects could provide cost savings. Unfortunately, inertial interaction effects are negligible for these tall, flexible structures, and hence the design engineers realized no benefit for their efforts and thereafter discontinued use of the procedures. The use of the procedures actually yield the most benefit for short-period, stiff structures with stiff, interconnected foundation systems (i.e., mats or interconnected footings) founded on soil.

C19.2 EQUIVALENT LATERAL FORCE PROCEDURE

This procedure considers the response of the structure in its fundamental mode of vibration and accounts for the contributions of the higher modes to story shears implicitly through the choice of the effective weight of the structure and the vertical distribution of the lateral forces. The effects of soil-structure interaction are accounted for on the assumption that they influence only the contribution of the fundamental mode of vibration.

C19.2.1 Base Shear. Base shear is reduced for the effects of SSI as indicated in Eq. 19.2-1 and 19.2-2. As indicated in Eq. 19.2-2, the change in base shear is related to the change in seismic coefficient (or spectral acceleration, as shown in Figure C19-1). The term $\left(0.05/\tilde{\beta}\right)^{0.4}$ in Eq. 19.2-2 represents the reduction in spectral ordinate associated with a change of damping from the fixed base value of $\beta = 0.05$ to the flexible base value of $\tilde{\beta}$.

C19.2.1.1 Effective Building Period. The fixed base period, *T,* is lengthened to the flexible-base period, $\tilde{T}$, using Eq. 19.2-3, which was derived by Veletsos and Meek (1974). Terms K_y and K_θ represent the translational and rocking stiffnesses of the foundation, respectively. The standard does not provide guidance on the evaluation of these stiffness terms. Equations for K_y and K_θ are given by Gazetas (1991), and a number of practical considerations associated with the analysis of these terms are reviewed in FEMA 440 (2005). For convenience, simplified guidelines are presented below for these stiffness terms for use with the standard.

For building foundation systems having lateral continuity, such as mats or footings interconnected with grade beams, stiffnesses K_y and K_θ can often be approximated as:

$$K_y = \frac{8}{2-\nu} G r_a \qquad \text{C19-2}$$

$$K_\theta = \frac{8}{3(1-\nu)} G r_m^3 \alpha_\theta \qquad \text{C19-3}$$

Where:

r_a = an equivalent foundation radius that matches the area of the foundation, A_0 (i.e., $r_a = \sqrt{A_0/\pi}$)

r_m = an equivalent foundation radius that matches the moment of inertia of the foundation in the direction of shaking (i.e., $r_m = \sqrt[4]{4I_0/\pi}$)

G = the strain-dependent shear modulus, as defined in the standard

ν = the soil Poisson's ratio (generally taken as 0.3 for sands and 0.45 for clays)

α_θ = a dimensionless coefficient that depends on the period of excitation, the dimensions of the foundation, and the properties of the supporting medium (Luco, 1974; Veletsos and Verbic, 1973; Veletsos and Wei, 1971). A similar coefficient exists for translation (α_y), but can be taken as 1.0 without introducing significant error, and hence is not shown in Eq. C19-2.

As noted in the standard, shear modulus G is evaluated from small-strain shear wave velocity as $G = \left(G/G_o\right)G_o = \left(G/G_o\right)\gamma v_{so}^2/g$ (all terms defined in the standard). Shear wave velocity, v_{s0}, should be evaluated as the average small-strain shear wave velocity within the effective depth of influence below the foundation. The effective depth should be taken as $0.75r_a$ for horizontal vibrations of the foundation and $0.75r_m$ for rocking vibrations (Stewart et al., 2003). Methods for measuring v_{s0} (preferred) or estimating it from other soil properties are summarized elsewhere (e.g., Kramer, 1996).

The dynamic modifier for rocking, α_θ, can significantly affect the computed response of some building structures. In the absence of more detailed analyses, for ordinary building structures with an embedment ratio $d/r_m < 0.5$ (where d = depth of embedment, measured from ground surface to base of foundation), the factor α_θ can be estimated as follows (Stewart et al., 2003):

$r_m/(v_{s0}T)$	α_θ
< 0.05	1.0
0.15	0.85
0.35	0.7
0.5	0.6

Foundation embedment has the effect of increasing the stiffnesses K_y and K_θ. For embedded foundations for which there is positive contact between the side walls and the surrounding soil, K_y and K_θ may be determined from the following approximate formulas (Kausel, 1974):

$$K_y = \frac{8Gr_a}{2-\nu}\left[1+\left(\frac{2}{3}\right)\left(\frac{d}{r_a}\right)\right] \qquad \text{C19-4}$$

$$K_\theta = \frac{8Gr_m^3}{3(1-\nu)}\left[1+2\left(\frac{d}{r_m}\right)\right] \qquad \text{C19-5}$$

Experimental studies and field performance data (Stokoe and Erden, 1975; Stewart et al., 1999) indicate that the effects of foundation embedment are sensitive to the condition of the backfill and that judgment must be exercised in using Eq. C19-4 and C19-5. For example, if contact is lost between the soil and basement walls, stiffnesses K_y and K_θ should be determined from the formulas for surface-supported foundations. More generally, the quantity d above should be interpreted as the effective depth of foundation embedment for the conditions that would prevail during the design earthquake ground motion.

The formulas for K_y and K_θ presented above can be applied to most soil profiles in which soil shear wave velocity, v_{s0}, changes with depth. However, if the soil profile consists of a surface stratum of soil underlain by a much stiffer deposit with a shear wave velocity more than twice that of the surface layer, K_y and K_θ may be determined from the following two generalized formulas in which G is the shear modulus of the surface soil and D_s is the total depth of the stratum:

$$K_y = \frac{8Gr_a}{2-\nu}\left[1+\left(\frac{2}{3}\right)\left(\frac{d}{r_a}\right)\right]\left[1+\left(\frac{1}{2}\right)\left(\frac{r_a}{D_s}\right)\right]\left[1+\left(\frac{5}{4}\right)\left(\frac{d}{D_s}\right)\right] \qquad \text{C19-6}$$

$$K_\theta = \frac{8Gr_m^3}{3(1-\nu)}\left[1+2\left(\frac{d}{r_m}\right)\right]\left[1+\left(\frac{1}{6}\right)\left(\frac{r_m}{D_s}\right)\right]\left[1+0.7\left(\frac{d}{D_s}\right)\right] \qquad \text{C19-7}$$

The above formulas are based on analyses of a stratum supported on a rigid base (Elsabee et al., 1977; Kausel and Roesset, 1975) and apply for $r/D_s < 0.5$ and $d/r < 1$ (r taken as either r_a or r_m). The applicability of those rigid base solutions to practical situations (non-rigid geologic media) was evaluated by Stewart et al. (2003), resulting in the recommendations provided above.

For buildings supported on footing foundations, the above formulas can generally be used with r_a and r_m calculated using the full building footprint dimensions, provided that the footings are interconnected with grade beams. An exception can occur for buildings with both shear walls and frames, for which the rotation of the foundation beneath the wall may be independent of that for the foundation beneath the column (this is referred to as weak rotational coupling). In such cases, r_m is often best calculated using the dimensions of the wall footing. Very stiff foundations, which provide strong rotational coupling, are best described using an r_m value that reflects the full foundation dimension. Regardless of the degree of rotational coupling, r_a should be calculated using the full foundation dimension if foundation elements are interconnected or continuous. Further discussion can be found in FEMA (2005). The use of discrete (non-interconnected) spread footing foundations in seismic regions is not recommended.

For buildings supported on pile foundations, lateral stiffness, K_y, can be taken as the sum of the lateral head stiffnesses of the supporting piles. These stiffness values are generally calculated using a beam on Winkler foundation model, which is discussed in detail elsewhere (e.g., Salgado, 2006). Rotational stiffness, K_θ, can be calculated from the vertical stiffness of the individual piles, k_{zi}, as follows:

$$K_\theta \approx \sum_i k_{zi} y_i^2 \qquad \text{C19-8}$$

where y_i = horizontal distance from the foundation centroidal axis to pile i measured in the direction of shaking. The approximation in Eq. C19-8 assumes an infinitely rigid pile cap and neglects the rotational stiffness of individual piles, which is typically a small contribution. Quantity k_{zi} can be calculated for an individual pile using well-established methods, such as discrete element modeling with *t-z* curves (e.g., Salgado, 2006).

The alternate approach in the standard, represented by Eq. 19.2-5, was derived using Poisson's ratio ν = 0.4, and is generally sufficient for non-embedded foundations that are laterally continuous across the building footprint and for which there is no "rigid" layer at depth in the profile (which would require the use of Eq. C19-6 and C19-7 to calculate foundation stiffness). The value of relative weight parameter, α (defined in the standard), can be taken as approximately 0.15 for typical buildings.

C19.2.1.2 Effective Damping. Bielak (1975, 1976) and Veletsos and Nair (1975) expressed the flexible-base first-mode damping ratio, $\tilde{\beta}$, as indicated in Eq. 19.2-9. This equation is based on analyses of the harmonic response of single-degree-of-freedom oscillators supported on a visco-elastic medium with hysteretic damping. Foundation damping factor β_0 incorporates the effects of energy dissipation into the soil due to radiation damping and hysteretic damping in the soil.

Figure 19.2-1 shows β_0 as a function of period lengthening ratio and was derived from the analytical solution presented in Veletesos and Nair (1975) for the condition of zero foundation embedment. Additional damping can be realized for embedded foundations, and the use of damping values from Figure 19.2-1 is conservative for such conditions. More exact solutions can be obtained using procedures given in FEMA (2005).

Eq. 19.2-9, in combination with the information presented in Figure 19.2-1, may lead to damping factors for the soil-foundation-structure system, $\tilde{\beta}$, that are smaller than the fixed base structural damping, β (assumed to be 0.05). However, it is recommended that $\tilde{\beta}$ never be taken as less than 0.05 for design applications. The use of values of $\tilde{\beta} > \beta$ is well-justified from field case-history data (Stewart et al., 1999, 2003).

The presence of a stiff layer at depth in the soil profile can impede radiation damping, rendering the values in Figure 19.2-1 too high. If a site consists of a relatively uniform layer of depth, D_s, overlying a very stiff layer with a shear wave velocity more than twice that of the surface layer, damping values should be reduced as indicated by Eq. 19.2-12.

C19.2.2 Vertical Distribution of Seismic Forces. The vertical distributions of the equivalent lateral forces for flexibly and rigidly supported structures are similar, and it is recommended that the same distribution be used in both cases, changing only the magnitude of the forces to correspond to the appropriate base shear. A greater degree of refinement in this step would be inconsistent with the approximations embodied in the requirements for rigidly supported structures.

With the vertical distribution of the lateral forces established, the overturning moments and the torsional effects about a vertical axis are computed as for rigidly supported structures. The above procedure is applicable to planar structures and, with some extension, to three-dimensional structures.

C19.2.3 Other Effects. In addition to its effect on base shear, inertial SSI also can increase the horizontal displacements of the structure relative to its base (because of rocking). This can increase the required spacing between structures and secondary design forces associated with *P*-delta effects. Such effects can be significant for stiff structural systems (e.g., walls and braced frames).

C19.3 MODAL ANALYSIS PROCEDURE

The procedure outlined above in Section C19.2 is applicable to a modal analysis by adjusting the modal period and damping ratio of the fundamental mode only. Higher modes are relatively unaffected by SSI (e.g., Bielak, 1976; Chopra and Gutierrez, 1974; Veletsos, 1977). Hence, the contributions of higher modes are computed as if the structure were fixed at the base, and the maximum value of a response quantity is determined as for fixed-base structures but with the adjusted first-mode responses.

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