INPUT GROUND MOTIONS FOR TALL BUILDINGS WITH SUBTERRANEAN LEVELS

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Abstract

Ground motion prediction equations and seismic hazard analyses are used to evaluate design-basis ground motions that apply for a free-field and ground surface condition. In this article, we address the manner in which those motions can be utilized for the analysis and design of buildings with subterranean levels. Procedures for specifying ground motions in buildings that are utilized in current practice fail to account for kinematic interaction effects, which can cause reductions of ground motion translation and the introduction of rocking. Simple models are reviewed that describe those effects (reduction of translation; introduction of rocking) based on finite element modeling for embedded rigid cylinders. Those models are well validated by recordings from two structures that have very stiff embedded foundations. Most building structures will not have foundations as stiff as those from the validation data set, and hence the degree to which foundation flexibility might affect the motions of embedded foundations was investigated using appropriately instrumented buildings. Preliminary analysis of the data suggests that (1) the foundations are not rigid and rigid body displacements and rotations of the foundation explain only 60-80% of the power of the motion at the ground-level, (2) using available models for kinematic interaction of embedded foundations, translational motions at the ground level of the investigated sites are dominated by base translation with relatively small contributions from base rocking, and (3) for the investigated buildings, base rocking from inertial interaction is much more significant in the frequency range of principal interest than base rocking from kinematic interaction. A number of future research tasks needed to further elucidate the problem are described.

Introduction

When earthquake ground motions are characterized for use in structural design and response simulations, the developed motions generally correspond to a “free-field” and “ground surface” condition. The term *free-field* implies no significant effect of structures on the characteristics of ground shaking. The term *ground surface* indicates that the estimated motions are at the surface of the earth.

Neither the free-field nor the ground surface conditions are generally satisfied for tall buildings in California’s urban centers. Most tall buildings are embedded, having their foundation level at some depth below the ground surface to accommodate subterranean levels. In some cases, the embedded portion of the structure has a different shape in plan view than the above-ground portion. The embedded portion is often much larger in plan, which is referred to as a podium.

A significant problem in earthquake engineering practice concerns the manner by which free-field, ground surface motions should be utilized for the analysis of buildings with subterranean levels. Current practice varies widely, but generally fails to take into account the reduction of translational components of ground motion with depth, the rocking components of base excitation, and the effects of foundation-soil interaction along basement walls and base slabs.
One purpose of this article is to describe modeling procedures that would be expected to realistically simulate this soil-foundation-structure interaction problem and to contrast those procedures with

modeling approaches commonly used in practice. We also present the results of past research investigating ground motions in embedded structures, evaluate the effectiveness of those approaches using available data from instrumented buildings, and identify knowledge gaps that should be addressed by future research. This work was performed by the authors as Task 8 of the Tall Buildings Initiative organized by the Pacific Earthquake Engineering Research Center. An advisory panel of practicing engineers and academics participated in the Task 8 work and provided valuable input for this document. Members of the advisory panel are listed at the end of this article.

Modeling Procedures for Embedded Structures

A schematic illustration of a building with subterranean levels is shown in Figure 1. The actual soil-foundation-structure system is excited by a wave field that is incoherent both vertically and horizontally and which may include waves arriving at various angles of incidence. These complexities of the ground motions cause foundation motions to deviate from free-field motions. This complex ground excitation acts on stiff, but non-rigid, foundation walls and the base slab, which in turn interact with a flexible and nonlinear soil medium having a significant potential for energy dissipation. Finally, the structural system is connected to the base slab, and possibly to basement walls as well.

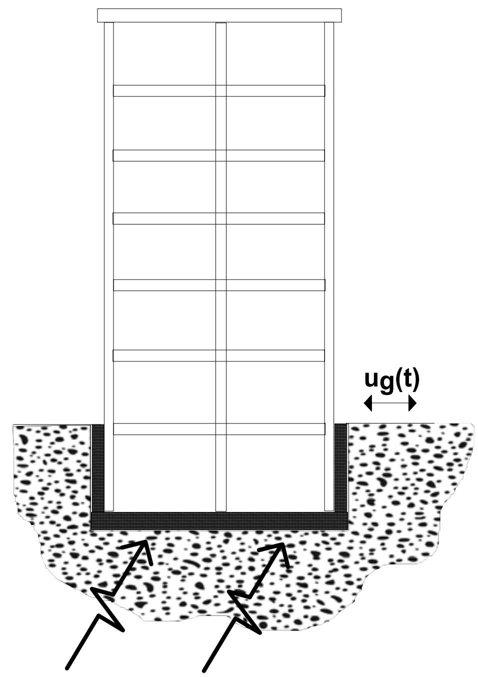

Figure 1. Schematic illustration of soil-foundation-structure system

There are two classical methods for modeling the problem defined in Figure 1. The first is a *direct approach*, in which a computational model of the full structure, foundation, and soil system is set up and excited by a complex and incoherent wave field. An example of a direct model using linear soil and structural elements in the program OpenSees (McKenna and Fenves, 2001) is given in Figure 2. This problem is difficult to solve from a computational standpoint, especially when the system contains significant nonlinearities, and hence the direct approach is rarely used in practice. In the second approach (referred to as the *substructure approach*), the complex soil-foundation-structure interaction problem is divided into three steps as illustrated in Figure 3.

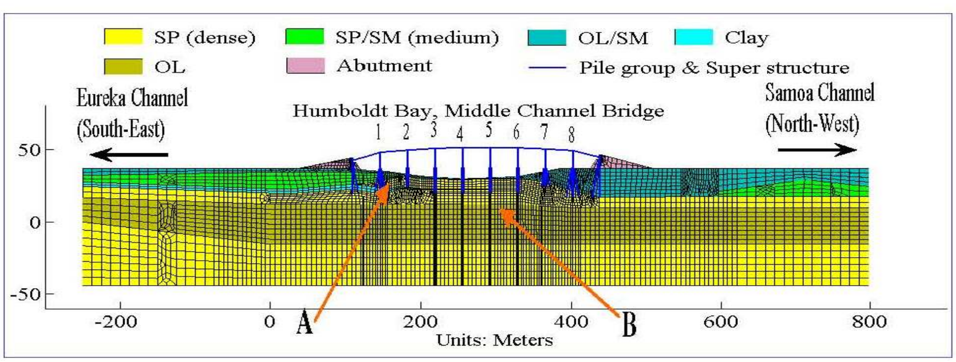

Figure 2. Example of direct model of soil-foundation-structure system using OpenSees (Zhang et al., 2003)

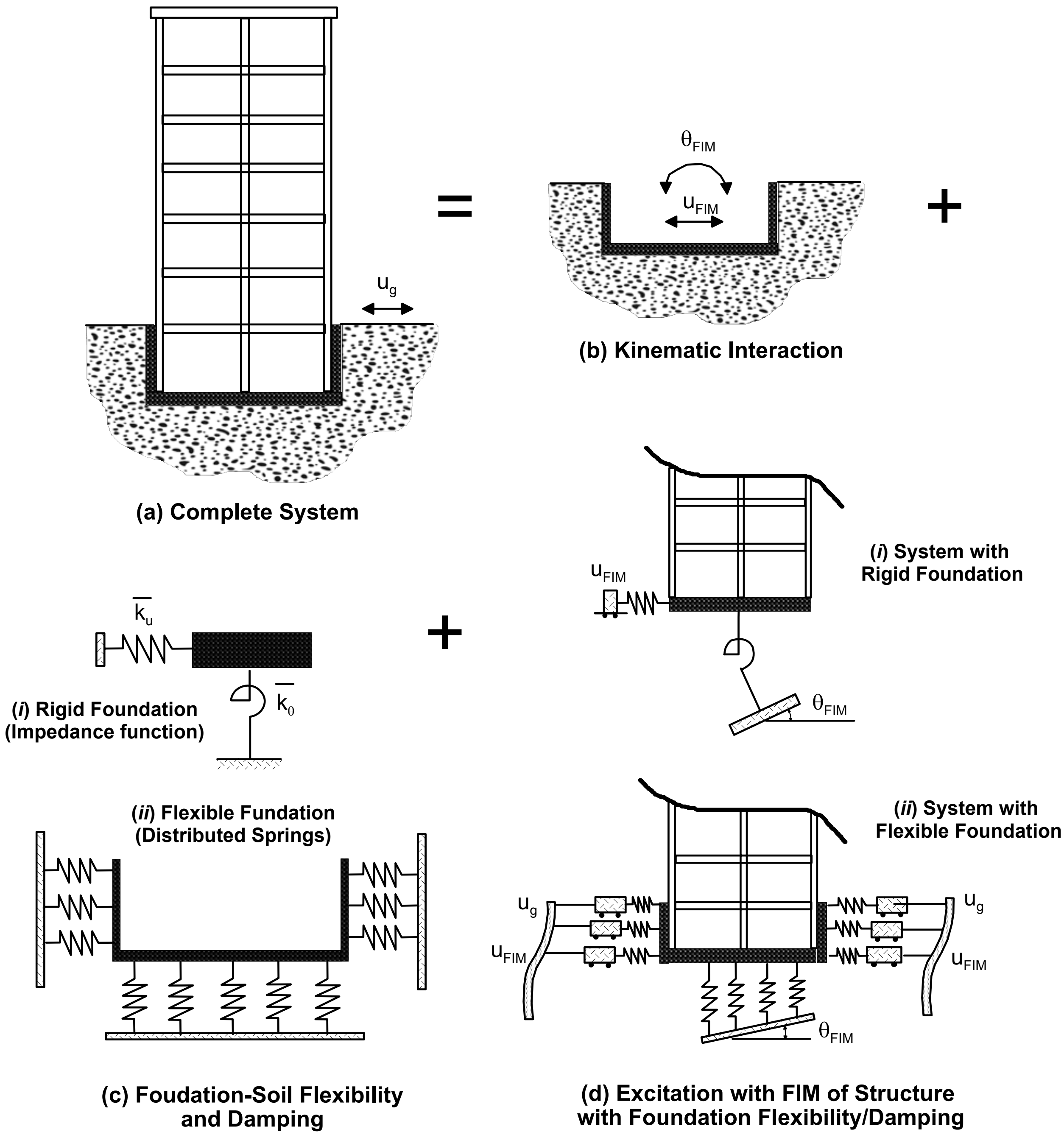

Figure 3. Schematic illustration of substructure approach to solution of soil-foundation-structure interaction problem using either (i) rigid foundation or (ii) flexible foundation assumption

As shown in Figure 3b, the first step in the substructure approach involves evaluating the motion that would be expected to occur on the foundation slab if the superstructure and foundation had no mass. This motion is termed the *foundation input motion*, and it accounts for the complexities of the incident wave field and its interaction with the stiff foundation system. For deeply embedded foundations, the dominant mechanism affecting base slab motions are embedment effects associated with ground motion reductions that occur below the original ground surface. The analysis of the foundation input motion is commonly referred to as a *kinematic interaction* analysis.

In the second step the stiffness and damping characteristics of the foundation-soil interaction are characterized using either relatively simple impedance function models (e.g., Gazetas, 1991) for rigid foundations (illustrated in Figure 3c-i) or a series of distributed springs and dashpots acting around the foundation (illustrated in Figure 3c-ii). Distributed springs are needed for non-rigid foundations and if internal moments and shears and relative displacements of the foundation are a required outcome of the analysis[1]. Only springs are depicted in Figure 2c for simplicity, but dashpots can easily be added in parallel to the springs (or the springs can be visualized as being complex-valued, which accounts for damping).

As shown in Figure 3d, the final step involves placing the superstructure atop the foundation and exciting the system through the foundation by displacing the ends of the springs using the rocking and translational components of the foundation input motion. Note that in the case of the distributed spring/flexible foundation model, differential ground displacements over the height of the basement walls (= depth of embedment) should strictly be applied given the vertical variability of ground motion.

Members of the Task 8 advisory panel have extensive experience working with structural design engineers in the western U.S. The panel members were asked about their experience with respect to the modeling of structures with subterranean levels. It was found that models of the type depicted in Figure 3 are not being used. Figure 4 shows the most common modeling approaches. In all cases, the foundation input motion is assumed equal to the free-field motion (i.e., $u_{FIM} = u_g$; $\theta_{FIM} = 0$), meaning that kinematic interaction effects are ignored. The stiffness and flexibility of the soil-foundation interaction are also often ignored by assuming a fixed base, as illustrated in Models 1 and 2 in Figure 4. The only distinguishing factor between Models 1 and 2 is the height of the structure, which is taken as the height above ground level for Model 1 and as the height above the foundation level for Model 2. Model 3 includes the flexibility of the soil-foundation interaction through the use of springs (damping effects are ignored) and excites the structure using the free-field motion applied at the ends of the horizontal springs. While better than Models 1 and 2, Model 3 fails to account for kinematic interaction effects on the base excitation and fails to account for ground excitation entering the system through stresses applied on the basement walls.

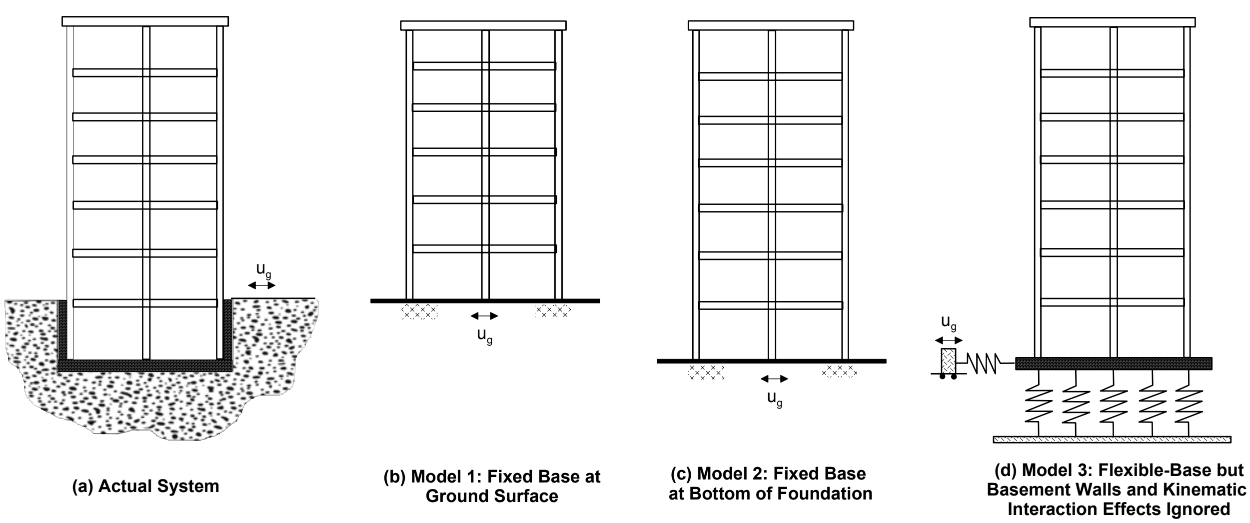

Figure 4. Schematic illustration of three models used in engineering practice to represent the actual soil-foundation-structure interaction problem

[1] Procedures for evaluating the depth variation of spring stiffness for distributed springs are not well established.

Theoretical Models for Ground Motions at the Base of Rigid Embedded Foundations

Solutions exist in the literature for most of the critical components of the substructure analysis depicted in Figure 2 (e.g., Kramer and Stewart, 2004). Most critical for the subject of this paper, solutions are available that describe the foundation input motion for embedded foundations as a function of the free-field motion. The available solutions apply for rigid massless cylinders embedded in a uniform soil of finite or infinite thickness (halfspace). When subjected to vertically propagating coherent SH waves, the embedded cylinders experience a reduction in base-slab translational motion relative to the free-field due to ground motion reductions with depth and wave scattering effects. In addition, rotations in the vertical plane are introduced.

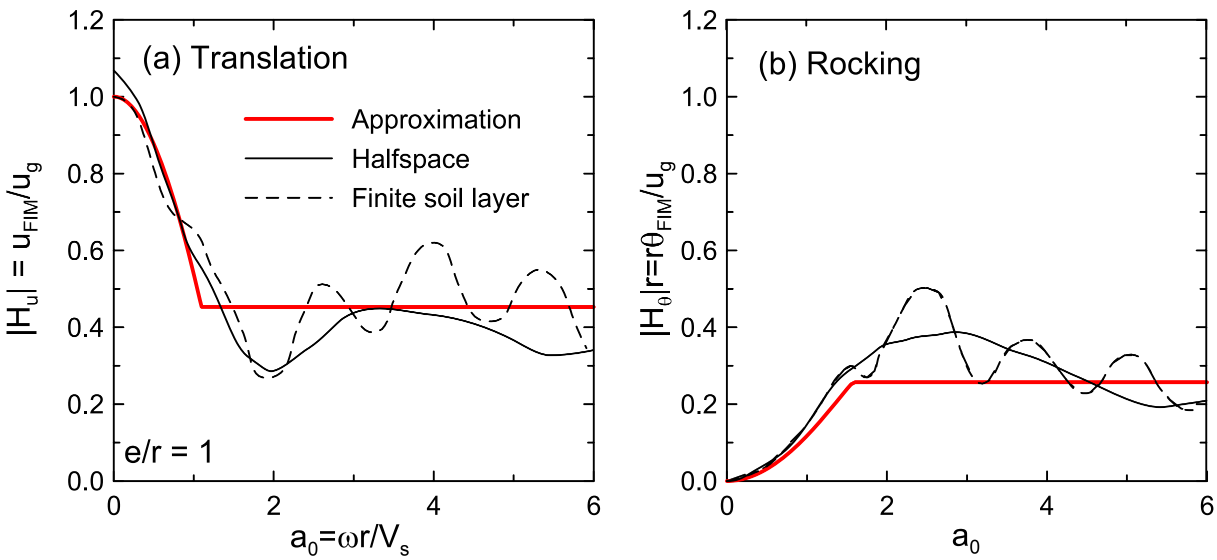

Figure 5. Solutions for the transfer functions between foundation input motions and free-field motions for translation and rocking of embedded rigid cylinders. Halfspace solution is from Day (1978) and finite soil layer case is from Elsabee and Morray (1977).

Elsabee and Morray (1977) and Day (1978) developed analytical transfer functions relating base-slab translational and rotational motions to free-field translations for an incident wave field consisting of vertically propagating, coherent SH waves. Day (1978) used finite element analyses to evaluate the motions at the base of a rigid, massless cylindrical foundation embedded in a uniform elastic half space (hysteretic soil damping $\beta = 0$, Poisson's ratio $\nu = 0.25$). Elsabee and Morray (1977) performed similar studies but for the case of a visco-elastic soil layer of finite depth over a rigid base ($\beta = 0.05$ and $\nu = 0.33$). The amplitude of the halfspace and finite soil layer transfer functions are shown together in Figure 5a 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 5a is the following approximate transfer function amplitude model developed by Elsabee and Morray (1977):

$$\left|H_u(\omega)\right| = \cos\left(\frac{e}{r}a_0\right) = \cos\left(\frac{e\omega}{v_s}\right) > 0.453 \tag{1}$$

where $\left|H_u\right|$ indicates the FIM/free-field transfer function (i.e., ratio of the Fourier amplitudes of the two motions), e = foundation embedment, r = effective foundation radius (selected to match the actual foundation base slab area), ω = circular frequency in rad/sec, V_s = soil shear wave velocity, and $a_0 = \omega r/V_s$.

Figure 5b shows similar results for the rocking component of the foundation input motion. In this case, the approximate transfer function is given by:

$$\left|H_\theta(\omega)\right| r = 0.257\left[1-\cos\left(\frac{e}{r}a_0\right)\right] < 0.257 \tag{2}$$

The amount of rocking increases with frequency up to a limiting value of 0.257 as shown in Figure 5b.

These results for an embedded rigid cylinder subjected to vertically incident coherent SH waves have been extended for cases of (1) soil properties varying with depth (Elsabee and Morray, 1977), (2) horizontally propagating coherent SH waves (Day, 1977), and (3) non-circular foundations (Mita and Luco, 1989) as follows:

- For soil properties which vary with depth, Elsabee and Morray found that the approximate transfer functions represented by Eq. 1-2 remain valid provided an averaged V_S across the embedment depth is used.
- For the case of horizontally propagating coherent SH waves, Day found that the base rocking was practically negligible, the filtering of horizontal motions was significant but was relatively insensitive to *e/r*, and a significant torsional response was induced at high frequencies (a_0 > 1.5). It should be noted, however, that horizontally propagating shear waves are generally of negligible engineering significance in SSI problems because components of ground motion with frequencies above about 1 Hz tend to attenuate rapidly with distance (Chen et al., 1981).
- Mita and Luco found that an embedded square foundation could be replaced by an equivalent cylinder without introducing significant error. The radius of the equivalent cylinder was defined as the average of the radii necessary to match the area and moment of inertia of the square base.

Validation of Theoretical Models

The analytical model presented in Eqs. 1-2 has been verified with respect to two sites that have deeply embedded circular foundations. One is referred to as Site A3, and consists of a deeply embedded nuclear containment structure at Humboldt Bay California. This site has a ratio of embedment to foundation radius of *e/r* = 26.2/9.0 m = 2.9. The second site is referred to as Site A46, and consists of a model of an embedded nuclear containment structure in Lotung, Taiwan. The site has a ratio of embedment to radius of *e/r* = 4.6/5.0 m = 0.9). Kim (2001) computed transmissibility function amplitudes |*H*| for these sites, which are shown in Figure 6 along with the simplified analytical transfer functions in Eqs. 1-2. The simplified transfer functions for translation are generally consistent with the observations. Only Site A46 has sufficient instrumentation to enable an evaluation of rocking. The rocking transfer functions for Site A46 are generally consistent with the data, but show some errors at low frequencies. These errors are likely due to significant rocking induced by inertial interaction, which is pronounced for this structure. Overall, the approximate procedure for estimating FIMs of embedded foundations appears to be well verified by the data from these deeply embedded circular structures. The large spikes in the transfer functions results from frequency intervals with low coherency associated noise in the data, and in general are not physically meaningful.

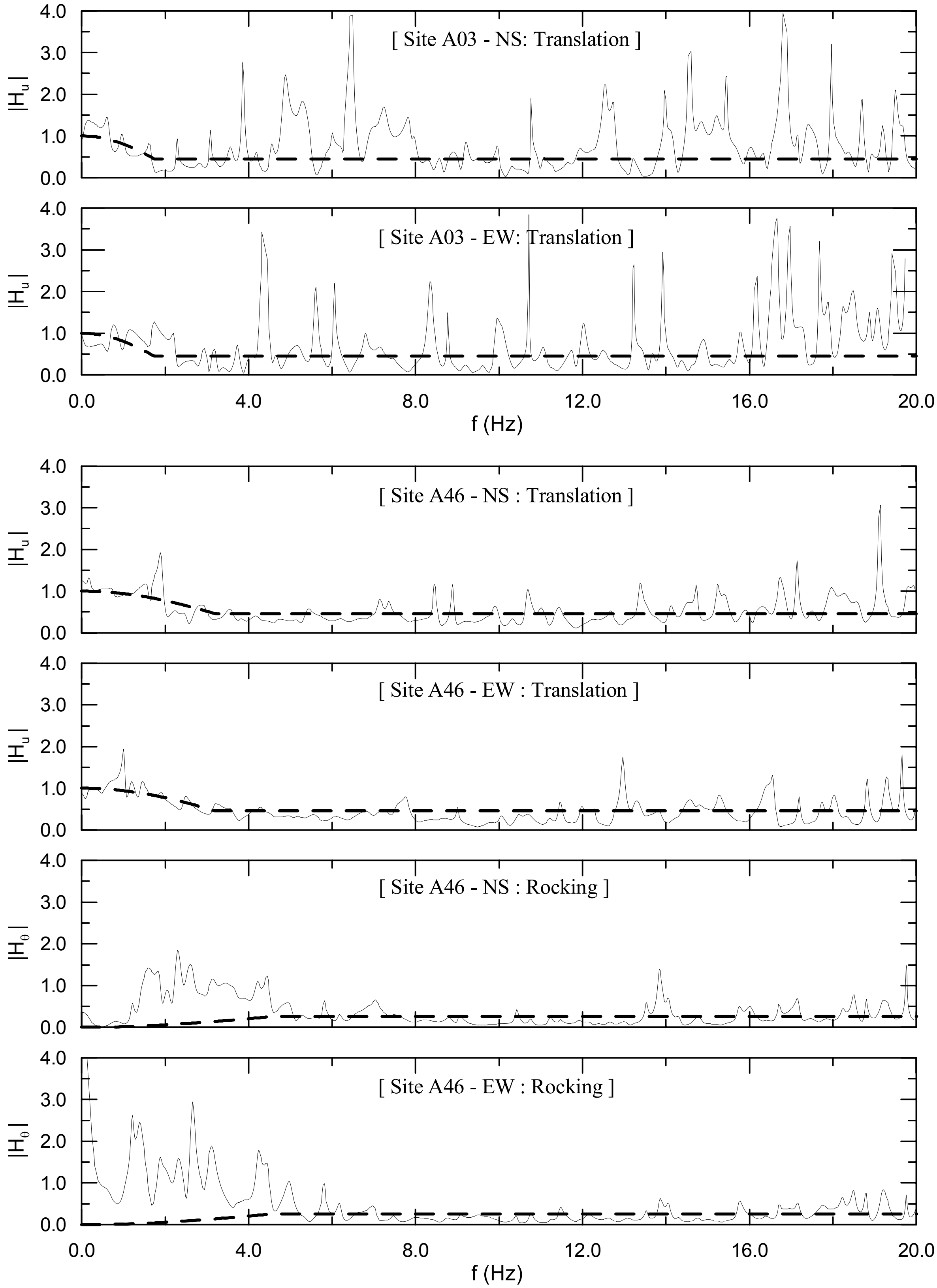

Figure 6. Comparison of simplified model for FIM with transmissibility function amplitudes for two sites with deeply embedded foundations (figure from Kim, 2001)

Discussion

The validation presented in Figure 6 provides confidence that the characteristics of foundation input motions can be estimated with reasonable accuracy for embedded, stiff circular foundations using Eq. 1-2. However, a number of issues remain unresolved with respect to the specification of input motions for buildings with subterranean levels. One of the key issues involves the degree to which transient soil displacements over the depth of embedment affects the foundation response through interaction with the basement walls.

These foundation wall-soil interaction issues are accounted for by the simplified model (Eqs. 1-2) because the finite element analyses used to develop the transfer functions shown in Figure 5 include this interaction. However, the results only apply for a *rigid* embedded structure. As the basement walls become relatively flexible, the transfer functions would be expected to change, especially for rocking. To help visualize how flexible basement walls would affect foundation response, consider a pile embedded in a soil profile as shown in Figure 7. Bending of the pile occurs because of the transient displacements of the adjacent soil; similarly, bending of basement walls is also possible if they are relatively flexible. It is obvious that for any given pile (or wall) depth, the displacements of the flexible pile at that depth would differ from those for a rigid pile. In the extreme case of a perfectly flexible pile, there is no rotational component to the FIM and the translation is similar to the soil column response at the foundation elevation.

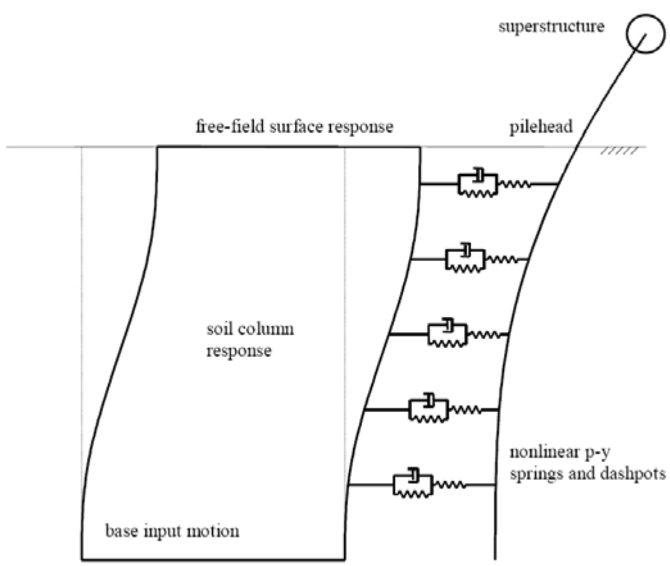

Figure 7. Schematic of flexible pile foundation subject to bending as a result of vertical variability of ground motion. Although an extreme case with respect to subterranean levels of buildings, the schematic is intended to show how free-field ground response can affect the bending of flexible basement walls, which in turn affects the characteristics of shaking in the subterranean levels of the structure (figure from Wilson, 1998)

The effects of foundation flexibility on the kinematic response of embedded foundations has not been investigated to the knowledge of the authors and the Task 8 advisory panel. Foundation flexibility effects on impedance functions have been investigated (Iguchi and Luco, 1982; Riggs and Waas, 1985; Liou and Huang, 1994; Todorovska et al. 2001), but for the kinematic problem all of the available theoretical solutions are for rigid foundations (Elsabee and Morray, 1977; Day, 1978; Mita and Luco, 1989). In the following section, we examine data from instrumented buildings to provide some preliminary insights into possible foundation flexibility effects on foundation motions.

Empirical Evaluation of Foundation Flexibility Effects on Foundation Motions

In this section, we examine data from structures with embedded foundations having the instrumentation shown in Figure 8. If the foundation is rigid, the motion at ground level (denoted u_{fg}) should be perfectly explained by the foundation base motion (u_{FIM}) and the foundation rocking ($\theta = (v_1 - v_2)/2r$). As depicted in Figure 8, that relationship would be as follows for a rigid foundation:

$$(u_{fg})_{rigid} = u_{FIM} + e\theta \tag{3}$$

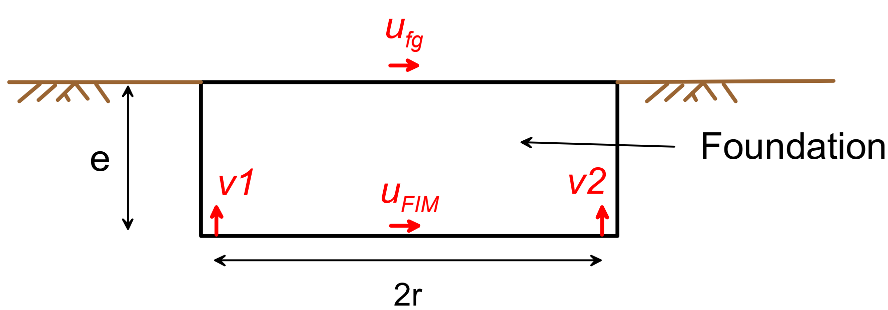

Figure 8. Schematic of minimal level of instrumentation needed to evaluate the effects of foundation flexibility on the motions at the ground level of the structure.

The data are used to evaluate the extent to which the real foundation motion at ground level (u_{fg}) is consistent with $(u_{fg})_{rigid}$. We have reviewed sites from the database of Stewart et al. (1998), and identified the sites in Table 1 as having the required instrumentation. As can be seen in Table 1, of the five sites with the required instrumentation, only three sites can be used because of missing data from specific instruments.

Table 1. Sites having required instrumentation for evaluating foundation flexibility effects (data from Stewart et al., 1998)

	SITE ATTRIBUTES						
No	Name	T (s)[1]	e (m)[2]	r_θ (m)[3]	v_s (m/s)[4]	Earthquakes Recorded	Data Utilization
A 23	LA 6 Story Office Building	0.9	4.3	14.1	152	1994 Northridge	Recording ufg not available; data not used
B3	San Francisco 47 story Office	5.3	8.2	24.1	174	1989 Loma Prieta	Recording ufg not available;
B4	San Francisco Transamerica	3.3	12.8	30.3	293	1989 Loma Prieta	Data used
B12	LA 32 Story Office	1.9	13.0	27.8	421	1994 Northridge	Data used
B13	LA 54 Story Office	5.9	14.0	26.4	436	1994 Northridge	Data used

[1] T = fixed base period of structure
[2] e = embedment of foundation (measured from ground surface to bottom of base slab)
[3] r_θ = fndn radius selected to match moment of inertia of base of foundation (trans. direction)
[4] v_s = average soil shear wave velocity to depth of r_θ below base slab

A representative example of data from one of the sites is given in Figure 9. The right side of Figure 9 shows acceleration histories for u_{fg}, $(u_{fg})_{rigid}$ (calculated per Eq. 3), u_{FIM}, and $e\theta$. The left side of Figure 9 shows power spectral density functions with 4X smoothing using the method of Welch (1967) for each of the acceleration histories. As can be seen from Figures 9a and 9b, ground level motion u_{fg} is not perfectly described by $(u_{fg})_{rigid}$, although the differences appear to be relatively small. The ground level motion derived from foundation rotation ($e\theta$) is small compared to the motion derived from foundation translation (u_{FIM}), particularly at low frequencies. The recording instruments used for these analyses are not located at the centroid of the foundation, and some of the effects observed here could result from torsion. Further discussion of possible causes of the discrepancies is given subsequently.

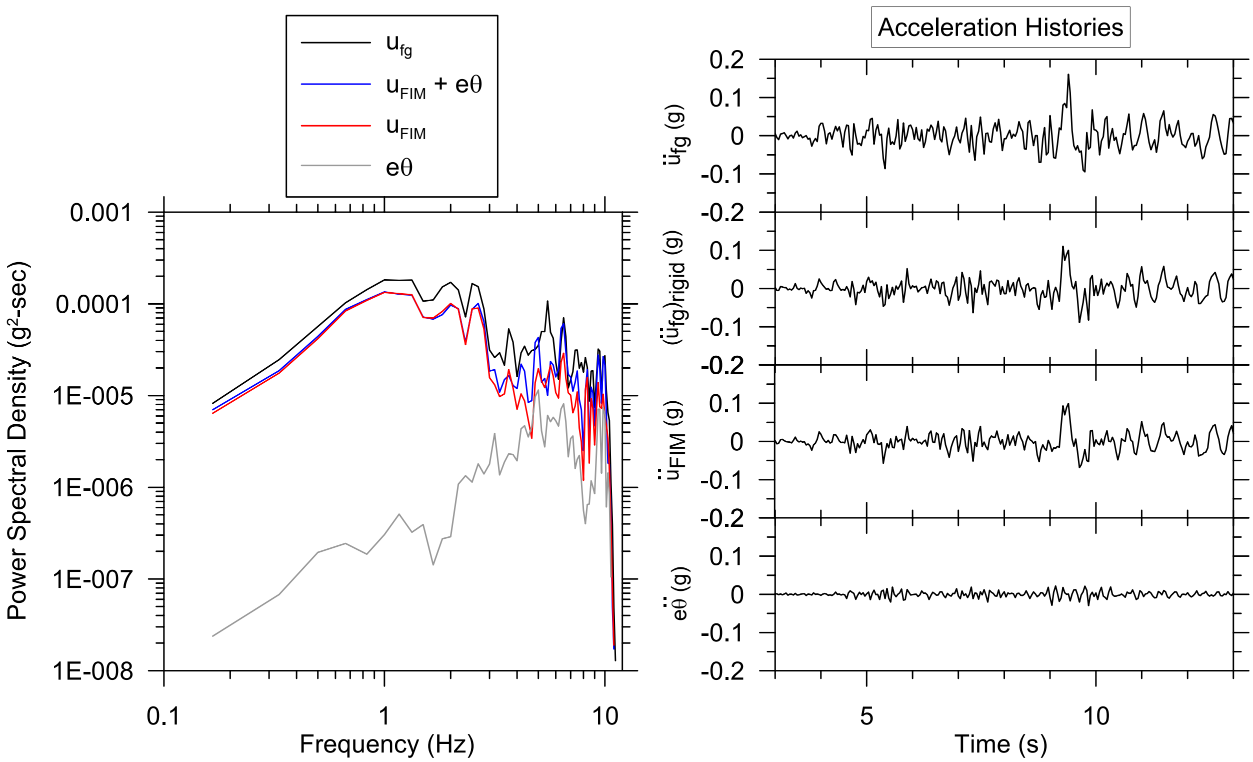

Figure 9. Power spectral density functions and acceleration histories for Site B12, showing contribution of base translation (u_{FIM}) and rotation ($e\theta$) to motion at top of foundation and also showing contribution of rigid body motions ($u_{FIM} + e\theta$) to top of foundation motion (u_{fg}). Note that the Nyquist frequency for the selected records after decimation is 12.5 Hz.

The power of each signal from Figure 9 is calculated as follows:

$$P_x = \int_{\omega_1}^{\omega_2} S_{xx}(\omega)d\omega \tag{4}$$

Where $S_{xx}(\omega)$ is the power spectral density function for signal x, ω denotes circular frequency (in rad/sec), and ω_1 and ω_2 denote the lowest and highest useable frequency per the signal filtering criteria. Table 2 lists for all the sites with available data the power ratios for the following motions: $(u_{fg})_{rigid}/u_{fg}$, u_{FIM}/u_{fg}, $e\theta/u_{fg}$. Also shown in Table 2 are the limiting frequencies used in the calculations (expressed in Hz, e.g. $f_l=\omega_l/2\pi$).

Table 2. Power ratios and amplitude ratios for buildings with instrumented foundations

Site	f_1 (Hz)	f_2 (Hz)	Power Ratios from Data			Weighted Amplitude Ratios			
			$(u_{fg})_{rigid}/u_{fg}$	u_{FIM}/u_{fg}	$e\theta/u_{fg}$	Data: u_{FIM}/u_{fg}	Theory: $u_{FIM}/(u_{fg})_{rigid}$	Data: $e\theta/u_{fg}$	Theory: $e\theta/(u_{fg})_{rigid}$
B4	0.17	12.5	0.60	0.58	0.01	0.74	0.97	0.06	0.03
B12	0.13	12.5	0.63	0.54	0.05	0.72	0.91	0.17	0.09
B13	0.66	50	0.84	0.77	0.05	0.88	0.96	0.16	0.04

As can be seen from the column of Table 2 labeled $(u_{fg})_{rigid}/u_{fg}$, 60-80% of the power of the ground level motion can be explained by rigid-body motion. Moreover, most of the rigid body motion (85 – 95%) can be explained by base translation. This suggests that base rocking is not a significant contributor to the motion at the ground level of the selected buildings. The rocking contributions may become more significant further up in the superstructure.

To facilitate comparisons of the data to the theoretical model for rigid foundations (represented by Eq. 1-2), we compute transfer function amplitudes for the ground motion ratios listed under the "Weighted Amplitude Ratios" portion of Table 2. Those ground motion ratios represent the relative contributions of foundation base translation and base rocking on the top-of-foundation translations. Example results are shown in Figure 10 for Site B12. The transfer functions marked as "data" are calculated as the square root of the corresponding power spectral ratios, the square root being necessary because the power spectrum corresponds roughly to the square of the Fourier amplitude spectrum (Pandit, 1991). The corresponding model predictions are obtained from the functions given in Eq. 1-2 as:

$$\frac{u_{FIM}}{(u_{fg})_{rigid}} = \frac{1}{1+\frac{e}{r}\left(\frac{|H_\theta|}{|H_u|}\right)}, \quad \frac{e\theta}{(u_{fg})_{rigid}} = \frac{1}{1+\frac{r}{e}\left(\frac{|H_u|}{|H_\theta|}\right)} \tag{5}$$

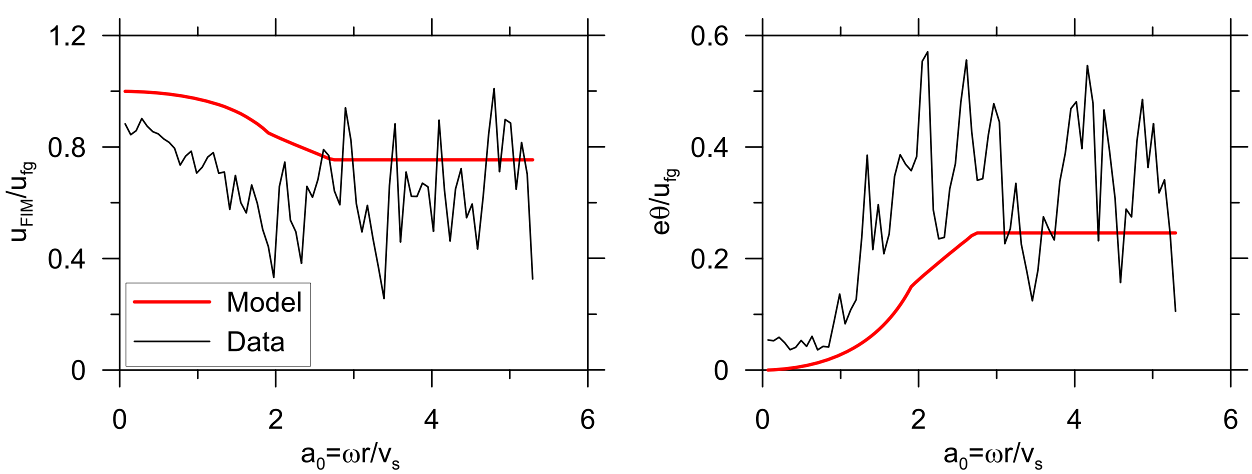

Figure 10. Comparison of transfer functions from rigid foundation model to data for transfer functions representing the relative contributions of base-of-foundation translation (u_{FIM}) and rocking (θ) on top-of-foundation displacement (u_{fg}). Data is from Site B12.

The results in Figure 10 show that the data generally follow the trends predicted by the rigid foundation model. However, for translation the data generally lies below the model, which we attribute to the relatively large ground level motion used for the data versus the model (i.e., $u_{fg} > (u_{fg})_{rigid}$). On the other hand, for rotation the data lies above the model, which likely results from additional rotations introduced at the foundation level by rocking associated with inertial interaction.

Lastly, we calculate a weighted average of the transfer function amplitudes across the frequency range of the motions, in which the weight for a given frequency is the corresponding power spectral density ordinate normalized so that the sum of the weights across all frequencies is unity. This effectively establishes the average value of the transfer function amplitudes, but weighted according to the dominant frequency band of the ground motion. The results are shown on the right side of Table 2 in the columns under the heading "Weighted Amplitude Ratios." The results in the "data" columns are very similar to the square root of the corresponding power ratios, as expected. The ratios from theory exceed those from data for translation and are smaller for rotation, which is consistent with the aforementioned trends from Figure 10.

The principal findings from this data analysis are as follows:

- The rigid foundation models for kinematic interaction of embedded foundations produce estimates of top-of-foundation ground motions that are biased on the low side. There are a number of possible explanations for this difference, including amplification through the subterranean portion of the structure (which is affected by the mass and flexibility of those portions of the structure), effect on demand introduced to the system of foundation non-rigidity between the base and top of the foundation, possible torsional effects on the ground-level motions, and potential effects of surface waves (stronger at the ground surface than at depth).
- For the structures investigated here, the top-of-foundation motion estimated by the rigid foundation models are dominated by base translation; base rocking could be neglected without any significant loss of accuracy.
- The base rocking for these tall buildings, as established from data, is considerably larger than what is predicted by rigid foundation models. This suggests that base rocking associated with inertial interaction is far more significant than base rocking associated with kinematic interaction.

Conclusions and Needed Future Research

In this article we have described on a schematic level how the soil-foundation-structure interaction problem for tall buildings with subterranean levels can be solved using a substructure approach (Figure 3). The modeling approaches used in current practice, as represented by the diagrams in Figure 4, fail to capture many of the key attributes of soil-structure interaction. One of the key missing attributes is the effects of kinematic interaction on the motions of embedded foundations. We review the attributes of an existing model for the reduction of translation and the introduction of kinematic rocking, and show the model to provide good results when compared to recordings from actual structures with very stiff cylindrical foundations.

Recordings from tall buildings with subterranean levels are then examined to evaluate the degree to which the foundations are truly rigid and to further test the aforementioned model. The main results of this work are summarized in the bullets above, and are not repeated here.

There are several research needs that have been identified by the authors and the Task 8 advisory panel to provide needed further insight into the problem of ground motion specification for buildings with subterranean levels. Those research needs are outlined in the following:

- Analytical studies of foundation flexibility effects on foundation input motions are needed. Results of those studies should be used to see if they can explain the misfit between the top-of-foundation motions and base-of-foundation motions described in the previous section. These studies should also investigate the effect on response of various load paths in subterranean levels (e.g., core walls that continue to base mat versus load transfer near ground level to perimeter basement walls).
- Additional ground motion data recorded on embedded foundations is needed, as the existing data covers a limited range of structural conditions, and the data is of limited resolution. This could be accomplished with high-resolution sensors used in a temporary monitoring operation, e.g., with the nees@UCLA equipment.
- Using the best available information on kinematic interaction for embedded foundations and foundation impedance functions, simulations should be performed to investigate the importance of soil-structure interaction on structural performance for tall buildings (collapse, serviceability, etc.).
- Additional simulations should be performed to investigate the adequacy of various options for foundation modeling, with the objective of identifying simplifications that could be applied in practice to the full model shown in Figure 4d (e.g., how do the results of simulations performed using the models in Figures 4b and 4c differ from those in Figure 4d?).

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