

FULL-SCALE SIMULATION OF SEISMIC STRUCTURE-FOUNDATION-SOIL INTERACTION

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Introduction

Structure-foundation-soil interaction (SFSI) can significantly affect the seismic performance of building and bridge structures. Engineering models of these effects are required for rational evaluations of seismic *demand* placed on the structure-foundation-soil system, and for evaluations of the deformation *capacity* of such systems. Existing models for SFSI effects related to both demand and capacity are based on simplifying assumptions that cannot be applied with confidence to realistic situations without calibration. Unfortunately, the present inventory of SFSI performance data from full-scale structures is very limited, and consequently improving the present state of model development and calibration is an important research need.

We begin by reviewing SFSI effects from both demand and capacity perspectives. Emphasis is placed on the role of SFSI in existing simplified analysis procedures used in practice so that its importance can be readily understood. Shortcomings of the existing field performance database needed for calibration of such procedures are outlined. While discussed in the context of existing analysis procedures, these shortcomings of the empirical data set are equally, if not more, vexing for the calibration of more sophisticated, next-generation analysis procedures. We conclude the paper by describing characteristics of a new mobile field laboratory under development at UCLA that is designed to enable collection and rapid dissemination of high quality performance data from vibration testing and aftershock monitoring of structures. Through use of this equipment, it will be possible to develop an inventory of field test results that will provide new insights into SFSI effects and enable analysis procedures for these effects to be developed and calibrated.

SFSI from Seismic Demand Perspective

Role of SFSI in Demand Analysis Procedures

Most U.S. seismic design practice for new buildings uses a force-based representation of seismic demand (e.g., ICBO, 1997; ICC, 2000; BSSC, 2000). Common features of these procedures for demand specification include: (1) a base shear force is calculated as the product of the building mass and a response spectral ordinate at the fundamental-mode building period, (2) this base shear force is reduced using a response reduction factor, R_w , and (3) a vertical distribution of this reduced base shear force is applied over the height of the building to enable an equivalent static analysis of building response to determine member forces.

Conceptually, SFSI can be thought to affect the base shear demand through two distinct mechanisms – inertial and kinematic interaction. Inertial interaction results from the flexibility of the soil-foundation system; it occurs because the base shear and moment induce foundation translation and rocking deformations relative to the free field. These effects can be quantified by the ratio of flexible- to fixed-base first-mode natural period (\tilde{T}/T) and by additional structural damping ($\tilde{\zeta}_0$) attributable to foundation-soil interaction (Jennings and Bielak, 1973).

Figure 1 schematically illustrates two possible effects of inertial interaction on the first-mode spectral acceleration. One spectrum in Figure 1 corresponds to the fixed-base structural damping ζ and the other to the effective structural damping $\tilde{\zeta}$. We assume that $\tilde{\zeta} > \zeta$, which is the typical condition. The spectral acceleration for a given flexible-based structure is obtained by entering the spectrum drawn for effective damping ratio $\tilde{\zeta}$ at the corresponding elongated period \tilde{T} . This spectral acceleration will be smaller than that for the corresponding fixed-base structure for medium- to long-period buildings (i.e., $T >$ about 0.5 sec), because spectral acceleration tends to decrease with period across that period range (unless there is a significant long-period ground response effect). However, spectral acceleration for the flexible-base condition can exceed that for fixed-base in very stiff structures because spectral accelerations tend to ramp up at low period from the peak ground acceleration value.

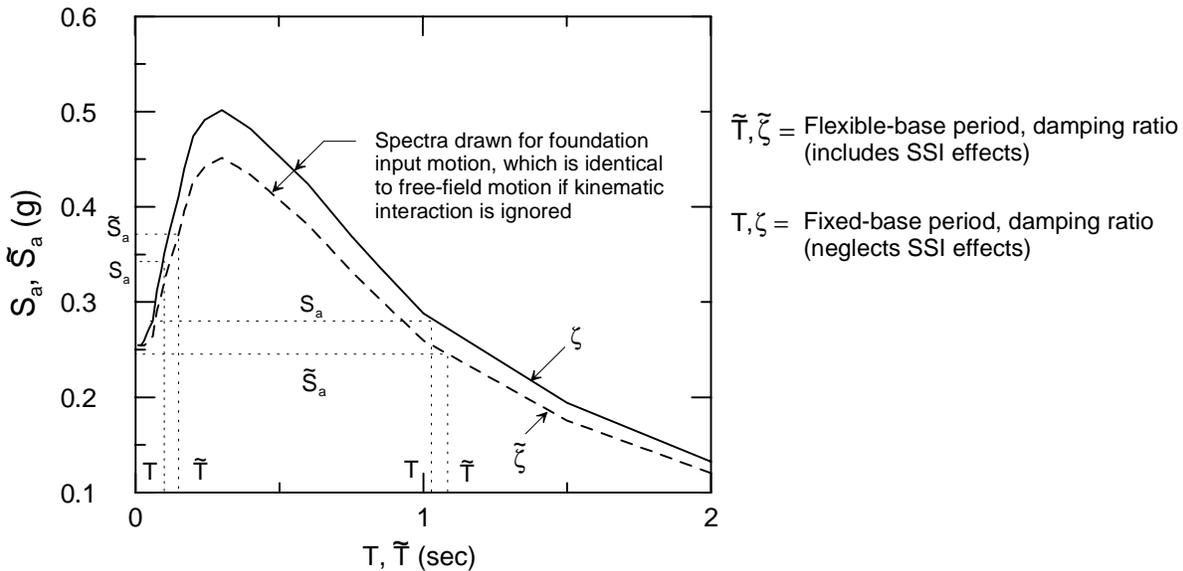


Figure 1. Schematic showing effect of SFSI on base shear demand

Kinematic interaction effects reduce foundation motions relative to the free-field due to the differences in stiffness between the foundation and surrounding soil, mainly as a result of base-slab averaging, embedment effects, and wave scattering (e.g., Luco and Mita, 1987; Veletsos and Prasad, 1989; Elsabee and Morray, 1977). These effects are most pronounced at high frequencies of excitation. Returning to the schematic in Figure 1, kinematic effects would enter the analysis as a period-dependant modification of the free-field design spectrum.

Models for SFSI Effects and Calibration Data Needs

Inertial and kinematic interaction effects can be quantified in terms of foundation impedance functions and transfer functions, respectively. Impedance functions represent the complex-valued and frequency-dependant relationship between forces and moments applied to the foundation and the corresponding displacements and rotations of the foundation relative to the free-field. Transfer functions represent the frequency-dependant variation between foundation and free-field ground motions.

Existing analytical models for impedance and transfer functions are generally based on a linear representation of the soil-foundation-structure response. While clearly an idealization of the true (nonlinear) behavior, linear SFSI models are generally acceptable for force-based demand characterizations (such as described above), which implicitly assume visco-elasticity through their use of response spectral ordinates. In the general case, a foundation impedance function model should consist of a 6×6 matrix for each element on the foundation (three translation and three rotation degrees-of-freedom per element, with the collection of elements representing a discretization of the foundation). Transfer functions for any degree-of-freedom would similarly have to be defined for each foundation element. However, foundation rigidity is commonly assumed so that the impedance of the entire foundation can be reduced to a single 6×6 matrix and a single transfer function can be used for a particular degree-of-freedom. Other simplifying assumptions include homogeneous, isotropic and visco-elastic soil and simple foundation shapes (typically rectangle or circle).

The degree to which these assumptions are (or are not) acceptable in practice is best assessed through careful analysis of field performance data. To evaluate impedance functions from seismograms, recordings of shaking in the structure, on the foundation, and in the free-field are required. The system identification procedures required to evaluate impedance functions from such recordings are presented by Kim (2001), but essentially consist of taking the ratio of the excitation force (or moment) to the relative foundation/free-field displacement (or rotation). Transfer functions are simply evaluated directly as the ratios of foundation/free-field motions (usually evaluated using non-parametric system identification techniques).

While simple in concept, the calculation of impedance from seismograms is a challenging system identification problem. Difficulties arise because signal noise is often of the same order (or larger) than the relative foundation/free-field displacements. Perhaps for this reason, few previous studies have attempted to infer impedance functions from earthquake data [i.e., Moslem and Trifunac's (1986) study of the Millikan library building, two earthquakes; Kim's (2001) investigation of five buildings, one earthquake each]. The identification of impedance functions from forced vibration testing is significantly more convenient because free-field motions are very small (ambient vibrations only). Investigators that have evaluated impedance functions from vibration testing include Lin and Jennings (1984), Luco et al. (1988), Wong et al. (1988), Crouse et al. (1990), and DeBarros and Luco (1995).

The above experimental studies are few in number and do not provide an adequate data inventory to calibrate existing impedance function models for the wide parametric variations in soil conditions, foundation flexibility, and foundation geometry encountered in practice. For example, the above-cited system identification and experimental studies focused on calibrating rigid foundation models of impedance due to limited numbers of foundation instruments. What is needed is a database of test results covering a wide parameter space, with each experiment ideally utilizing a large number of sensors on the foundation so that possible spatial variations of impedance and transfer functions can be evaluated from the data. Development of this database requires a field testing system that includes a large number of high-resolution sensors that can be efficiently installed. The UCLA NEES program is developing equipment to meet this demand, and this equipment is described below.

The evaluation of transfer functions is only possible for earthquake excitation (i.e., such data cannot be obtained from forced-vibration testing). Identifications of transfer functions from strong motion data have been performed by Kim (2001), but again the models calibrated from these data were developed using a rigid foundation assumption. What is needed in this case is instrumentation that can be rapidly installed on foundations and in the free-field after major earthquakes for aftershock monitoring. This instrumentation need is also being addressed by the UCLA NEES program.

SFSI from Capacity Assessment Perspective

Role of SFSI in Capacity Analysis Procedures

U.S. seismic design practice for existing buildings uses a displacement-based representation of structural capacity (ATC 40, FEMA 273). The system performance is represented by a lateral force-displacement relationship calculated using a so-called “pushover analysis.” As illustrated in Figure 2, in a pushover analysis a prescribed vertical distribution of static lateral load is applied to a structure, and the nonlinear deformation response of the structure-foundation-soil system is evaluated based on an appropriate system model. The cumulative lateral load (i.e., base shear) can be plotted against a control point displacement to provide a concise representation of the nonlinear system behavior. A point on this curve defines a damage state in the building, since the deformation of all of the structural components can be related to the control point displacement.

The expected seismic structural performance is assessed by combining this capacity curve with an Acceleration-Displacement Response Spectra (ADRS) as illustrated in Figure 3. The ADRS represents the seismic demand, and consists of a plot of spectral acceleration (S_a) vs. spectral displacement (S_d). The demand spectrum is reduced, as appropriate, to account for inelastic deformation of the structure. The ADRS is compared to the so-called capacity spectrum, which is the capacity curve normalized by building mass and corrected for higher-mode effects. The capacity and demand spectra meet at the performance point, which represents the expected structural performance given the seismic demand.

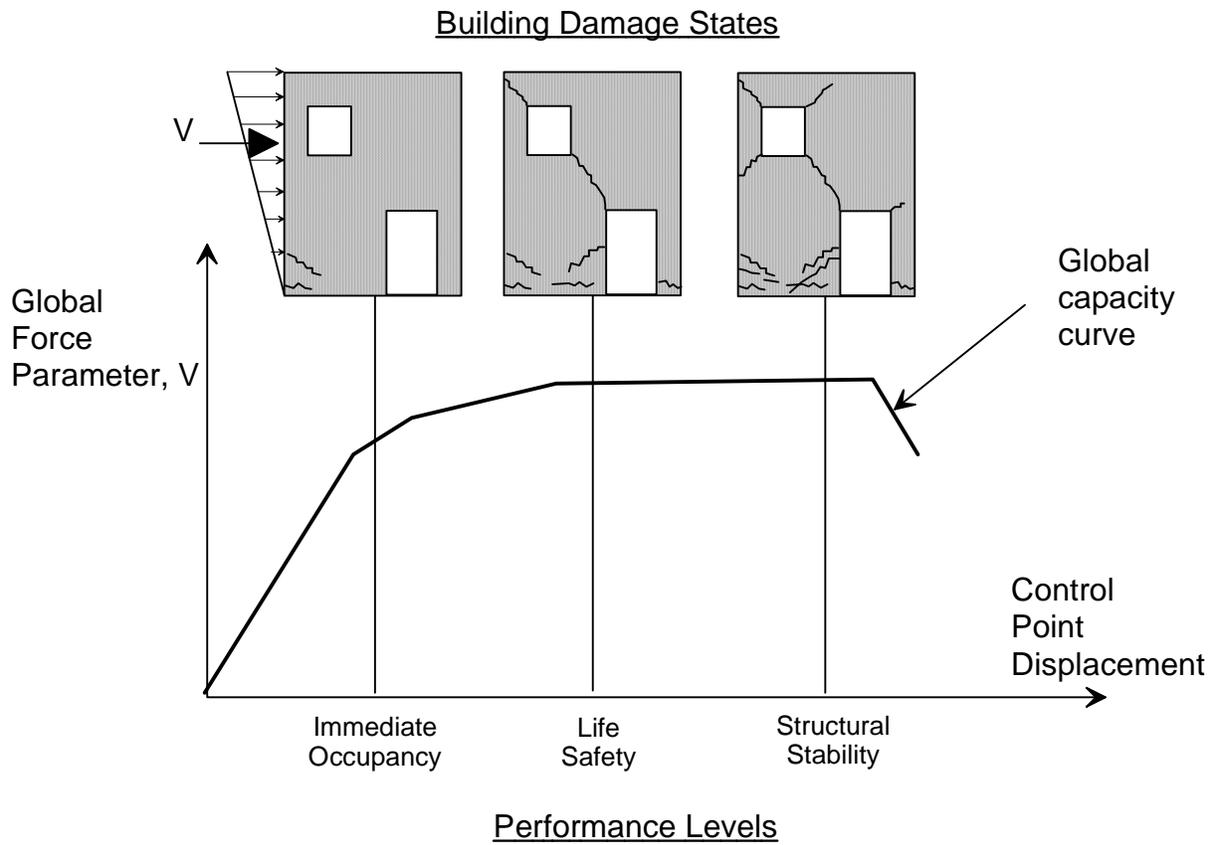


Figure 2. Capacity curve for nonlinear structure and associated damage states (after Comartin et al., 2000)

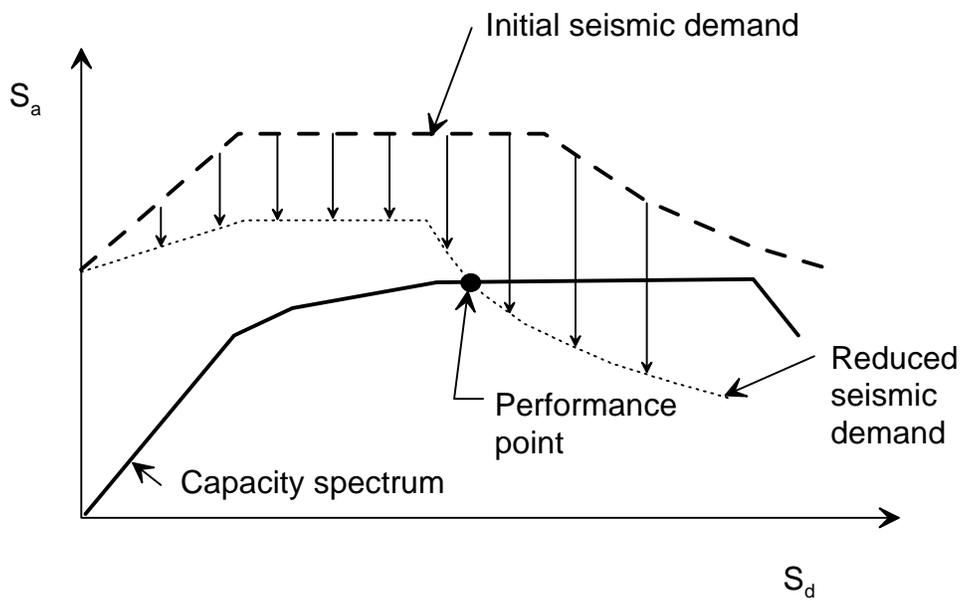


Figure 3. Use of capacity and demand spectra to define performance point (after Comartin et al., 2000)

The effects of SFSI on the above process are two-fold. First, SFSI affects demand spectra through the effective system damping (inertial interaction) and spectral shape (kinematic interaction). These SFSI effects are neglected in current design documents (e.g., ATC-40 and FEMA 273). Second, capacity assessment is controlled by nonlinear component models used to assemble the nonlinear system model used in the pushover analysis. Accordingly, nonlinear models for foundation-soil response are needed for such analyses. Our emphasis in this section is SFSI effects on capacity assessment.

Models for SFSI Effects and Calibration Data Needs

Nonlinear component models for soil-foundation interaction usually take the form of nonlinear springs that are attached at one end to a structural foundation element and at the other end to “free-field” soil (i.e., soil sufficiently far from the foundation so as to not be affected by foundation displacements). The characteristics of these springs differ depending on the type of foundation (e.g., shallow vs. deep) and the direction of foundation displacement.

The importance of including appropriate soil springs in the evaluation of the system capacity curve has been demonstrated by Comartin et al. (1994), who performed both fixed- and flexible-base pushover analyses of the building illustrated in Figure 4. Analyses with the fixed-base model indicated that the critical portion of the structure requiring retrofit was the shear wall, whereas the flexible-base model allowed rocking of this wall that transferred the critical zone to moment connections in the frame. This example illustrates that failure to properly account for SFSI effects can lead to a misdiagnosis of the critical failure mechanism in structures.

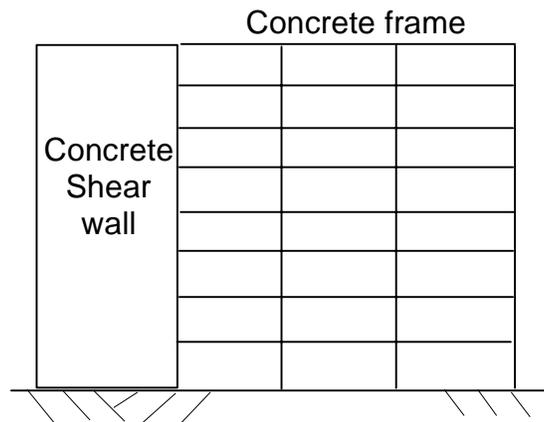


Figure 4. Schematic illustration of building type considered in performance analysis by Comartin et al. (1994)

A number of nonlinear spring models are available for deep foundations such as driven piles or drilled shafts. For lateral response analyses, the springs located along the shaft are referred to as p - y curves, with p representing the resultant soil reaction force per unit length and y representing the lateral pile displacement. For vertical response analyses, the springs are referred to as t - z curves along the shaft and q - z curves at the toe, with t representing the vertical reaction force per unit length, q representing vertical stress at the toe, and z representing the vertical displacement. Existing models for p - y , t - z , and q - z curves are presented elsewhere [Reese (1984), Reese et al. (1986) for p - y ; Kraft et al. (1981) for t - z ; O'Neill and Reese (1999) for q - z]. The key points to be made here are two-fold: (1) the curves consist of a nonlinear portion leading up to a maximum capacity, and (2) the curves are either based directly on back-analysis of field measurements, or calibration of analytical models from field performance data.

Like deep foundations, nonlinear springs for shallow foundations are characterized by an initial, strain softening segment, and an ultimate capacity. Current practice (ATC-40, FEMA-273) is to use a bi-linear representation of the spring, with the ultimate resistance established from bearing capacity analysis and the initial stiffness taken as the real part of a foundation impedance function for a rigid circular or rectangular foundation shape on a simple soil profile. While a fairly substantial number of laboratory and field tests have been performed to support the development of bearing capacity analysis procedures (synthesized in Vesic, 1973, 1975), characteristics of nonlinear springs prior to shear failure (bearing capacity) are not well constrained from experimental data. Issues associated with the development of such models are similar to those affecting impedance functions (i.e., effects of soil layering and foundation flexibility, shape, and embedment), but also include the effects of soil nonlinearity at large strain. The development of models that appropriately account for these various effects is an important research need that is well suited to field testing. Field testing is desirable for the obvious reason that boundary conditions are properly incorporated, but is also convenient because sensors can be readily installed along shallow foundation elements within existing structures. Field testing may be less practical for deep foundations, unless sensors are installed within the shafts during construction.

Field Testing Equipment to Meet SFSI Research Needs

A National Earthquake Engineering Simulation (NEES) equipment site is being developed at UCLA that will enable field testing and monitoring to meet the aforementioned research needs. The project is specifically targeted at field vibration testing and earthquake aftershock monitoring of structural and foundation systems for building and bridge structures.

Project Overview

A mobile field laboratory is under development to enable state-of-the-art seismic performance characterization of full-scale structural and foundation systems. The main components of this mobile field laboratory are schematically shown in Figure 5 and include: (1) shakers, (2) sensors for installation on structural components or within the ground, and (3) equipment for rapid data acquisition, processing, and distribution.

A total of four vibration sources are being developed to enable a wide range of loading conditions. Three eccentric mass vibrators are being acquired that can be used independently or synchronized to provide sinusoidal unidirectional, bi-directional, or multi-source excitation. Two of the shakers are identical and have an operational frequency range of 0 to 25 Hz, and when synchronized can produce a maximum force of about 900 kN. The third shaker has a frequency range from 0.1 to 4.2 Hz and a maximum force of 180 kN. In addition, a linear inertial shaker will enable dynamic loading capabilities across a very wide frequency range - with an approximate maximum force of 45 kN.

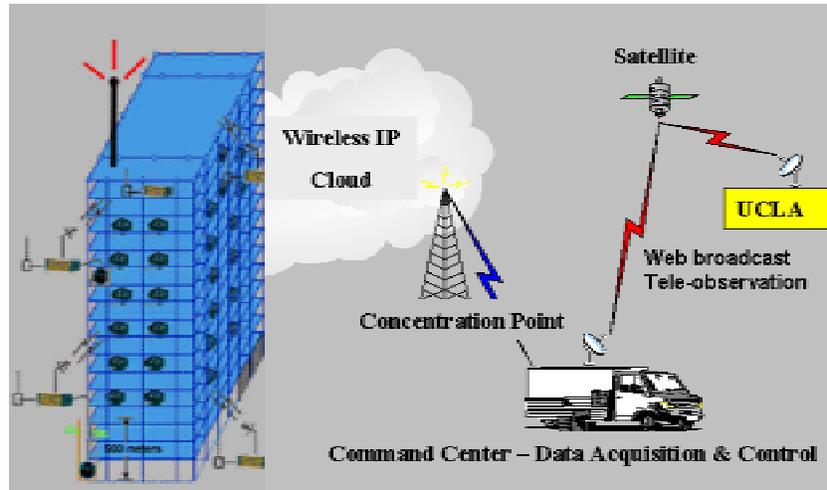


Figure 5. Schematic overview of the UCLA mobile field laboratory

The accelerometers being acquired for deployment in structures have a frequency range of 0 to 200 Hz with a 4.0 g acceleration limit and wide dynamic range (155 dB, which allows for 10^{-8} g resolution). Additional sensors include accelerometers to monitor soil vibrations (under development), and displacement transducers (both linear variable differential transducers and fiber optic sensors). These sensors will be connected to a digitizer and wireless router, which transmits the data wireless to a central data recording system within a mobile trailer parked on-site. This recording system will include a UNIX workstation configured with the Antelope™ software program, which is widely used in seismology.

Twenty custom soil vibration monitoring probes are being fabricated to enable simultaneous subsurface measurements. These probes will be inserted and later retrieved from soil around a foundation by a Hogentogler cone penetration truck, which has been purchased as a part of the NEES project. Data from these sensors will be particularly useful for measuring the energy radiating away from vibrating foundations. These probes will be attached to the conventional cone tip via a non-tension connection, such that upon cone withdrawal, the probe will be left in place. Once in place, the probes will be capable of measuring small amplitude ground vibrations from surface testing or motions from earthquake aftershocks. Each probe will be outfitted with a triaxial accelerometer that has a frequency range of 0 to 150 Hz, 3.0 g acceleration limit and 45

μg resolution. The probes will interface with the central data acquisition system and communicate data to the trailers using wireless technology.

The mobile trailer will house a satellite uplink in addition to the central data acquisition system, to enable rapid dissemination of data to another UNIX workstation located at UCLA. The satellite uplink will transmit data at 1.28 Mbits/s as well as substantial video signal transmission if desired. Equipment installed at UCLA will receive the satellite signal, and allow the data to be accessed over the local area network. Lastly, off-site personal computers networked to the UNIX workstation located at UCLA will be able to teleobserve the field experiments in real time.

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