Time-Domain Soil-Structure Interaction Analysis for Nuclear Facilities

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Abstract

The Nuclear Regulatory Commission (NRC) regulation 10 CFR Part 50 Appendix S requires consideration of soil-structure interaction (SSI) in nuclear power plant (NPP) analysis and design. Soil-structure interaction analysis for NPPs is routinely carried out using guidance provided in the ASCE Standard 4 titled "Seismic Analysis of Safety-Related Nuclear Structures and Commentary (ASCE-4, 1998)". This document, which is currently under revision, provides detailed guidance on linear seismic soil-structure-interaction (SSI) analysis of nuclear facilities using deterministic and probabilistic methods. However, a new appendix is being added to the revised version of ASCE Standard 4 (ASCE-4, 2015) to provide guidance for time-domain nonlinear SSI (NLSSI) analysis. Nonlinear SSI analysis will be needed to simulate the following behaviors: material nonlinearity (in soil and/or structure), rocking or sliding of the foundation, static and dynamic soil pressure effects on deeply embedded structures, local soil failure at the foundation-soil interface, nonlinear coupling of soil and pore fluid, nonlinear effects of gaps between the surrounding soil and the embedded structure and seismic isolation systems.

ASCE-4 2015 Appendix B provides general criteria to consider while performing NLSSI analysis, but lacks a methodology for performing the analysis. This paper provides a description of the NLSSI methodology developed for application at nuclear facilities, including NPPs. This methodology is described as series of steps to be followed to produce reasonable results. These steps require some numerical capabilities, such as nonlinear soil constitutive models, which are also described in the paper.

1. Introduction

In the 1960s, the dynamic response of nuclear power plant (NPP) structures was calculated using analytical calculations on fixed-base structures. Computer programs were developed in the late 1960s that could calculate seismic response of multi-degree of freedom dynamic analysis, and by the late 1970s numerical programs that consider soil-structure interaction (SSI) problems were developed. Although these programs were linear and functioned in the frequency domain, they were used with equivalent-linear soil properties to approximate the nonlinear response. The System for Analysis of Soil-Structure Interaction (SASSI) (Lysmer, Ostadan et al. 1999) is a frequency-domain code developed in this era, and is currently still widely used to calculate the dynamic response of nuclear facilities.

Current seismic analysis of NPPs, high-hazard nuclear waste facilities, and spent fuel storage systems is carried out using the guidance provided in American Society of Civil Engineers (ASCE) Standard 4 (ASCE 1998), and is generally performed using frequency-domain codes such as SASSI. These linear analysis codes, when used in accordance with ASCE Standard 4 and Standard 43 (ASCE 2005), should generally lead to conservative NPP designs for low to moderate amplitude seismic events. However, for larger seismic events, these codes are theoretically not accurate when nonlinear effects such as gapping and sliding, and local soil nonlinearities affect the in-structure response. Nonlinear analysis is required to calculate the in-structure responses for such events.

As the seismic hazard values at nuclear facility sites increase, and additional beyond design basis earthquakes (BDBE) are observed, nonlinear effects become increasingly important. With the increase in seismic hazard values, the input ground motions used to numerically evaluate nuclear facility responses become more intense, resulting in larger soil strains, and increasing the potential for gapping and sliding at the foundaton. Some of the recent earthquake recordings at nuclear power plants sites have exceeded design basis values. The peak ground acceleration values of these earthquakes, recorded at North Anna (Virginia Electric and Power Company), Fukushima Daichii and Daini (TEPCO 1), and Kaswazaki-Kariwa (TEPCO 2), are presented in Table 1.

Table 1: Peak recorded acceleration and the corresponding design values

	KK 2007	**Fukushima 2011**	**North Anna 2011**
Design Value (g)	0.20	0.26 (Original) 0.45 (Updated)	0.18
Recorded Value (g)	0.32	0.56	0.26

Nonlinear SSI analysis in the time-domain is needed to explicitly capture material nonlinearity in the soil, and geometric nonlinearity at the foundation (gapping and sliding) during design basis and beyond-design basis earthquakes. Since nonlinear time-domain analysis is not routinely performed in the nuclear industry, a methodology is needed for its widespread implementation. Appendix B in the forthcoming ASCE Standard 4 (ASCE forthcoming) provides guidance for implementing such a methodology.

Nuclear facility designs to resist seismic shaking are expected to be conservative, while seismic probabalistic risk assessments (SPRA) are expected provide results that are best estimate. Therefore, nonlinear soil-structure interaction (NLSSI) analysis could be used for providing an economic nuclear facility design at sites with moderate to high seismic hazard.

Additionally NLSSI could be used when performing SPRA calculations and for consideration of seismic isolation.

The primary objectives of this paper are to 1) present a methodology for performing NLSSI analysis in the time domain, 2) discuss guidance provided in Appendix B of the forthcoming ASCE Standard 4, 3) discuss the circumstances in which, nonlinear seismic soil structure interaction (SSI) analysis may be necessary for NPPs and high-hazard nuclear waste facilities, 4) provide a framework for acceptable numerical methodology that can be used for performing nonlinear time-domain analysis, and 5) discuss minimum acceptable analysis and peer review requirements.

2. Literature review

Nonlinear SSI analysis in the time domain can be performed using commercial programs such as ABAQUS (Dassault Systèmes 2005), ANSYS (ANSYS Inc. 2013) and LS-DYNA (LSTC 2013). Nonlinear SSI analysis has been previously been performed for structures such as canal locks, bridges and liquid natural gas tanks (Willford, Sturt et al. 2010), but is yet to be used for nuclear applications.

Nonlinear SSI analysis for nuclear applications has received increased attention in the last decade, and most of the studies involve benchmarking of the time-domain codes against the well-established frequency-domain codes. Only few such studies have been performed so far, mainly due to the contemporariness of the time-domain, nonlinear methods. In one study Xu, Miller et al. (2006) compared the predictions made using SASSI and LS-DYNA (LSTC 2013) from the SSI analyses of deeply embedded nuclear structures. They found that the results calculated using SASSI and LS-DYNA differed considerably for both linear and nonlinear analyses. While the differences are expected for nonlinear analyses, they stipulated that the results from linear analyses performed in the two codes diverged due to the differences in the corresponding damping formulations. However similar studies performed by Anderson, Elkhoraibi et al. (2013) and Coronado, Anderson et al. (2013) showed that the linear SSI analyses of deeply embedded nuclear structures using time-domain and frequency-domain codes resulted in very similar structural responses. In these studies, Anderson, Elkhoraibi et al. (2013) compared the results from SAP2000 (Computers and Structures Inc. 2011) to those from SASSI2010 (Ostadan and Deng 2011), while Coronado, Anderson et al. (2013) compared the results from the extended subtraction method in SASSI2010 to those calculated using the commercial finite-element code ANSYS (ANSYS Inc. 2013). Spears and

Coleman (2014), in a comprehensive study, developed a methodology for nonlinear SSI analysis in the time domain, compared the SSI responses calculated using SASSI and time-domain codes, and identified some issues regarding the usage of these codes. A similar study was performed by Bolisetti and Whittaker (2015) and Bolisetti, Whittaker et al. (2015), which involved benchmarking time-domain SSI analysis in LS-DYNA against SASSI2000, performing detailed comparisons between the SASSI2000 and LS-DYNA results for highly nonlinear cases and identifying potential practical issues in performing SSI analysis using these programs.

While the studies cited above use commercial codes for nonlinear time-domain SSI analysis, the US Nuclear Regulatory Commission has funded the development of a dedicated code for nonlinear SSI analysis. This code, currently being developed and maintained by Jeremic, Kammerer et al. (2011), performs a fully nonlinear SSI analysis through nonlinear constitutive material models, contact elements and utilizes the domain reduction method (Bielak, Loukakis et al. 2003) for ground motion input.

3. A methodology for time-domain SSI analysis

Idaho National Laboratory (INL) is currently performing research and development to develop a nonlinear soil-structure interaction (NLSSI) methodology for application to nuclear facilities (Spears and Coleman 2014). This methodology is a series of steps that an analyst or reviewer can follow to perform a fully nonlinear SSI analysis in the time domain including 1) nonlinear site response, 2) nonlinear hysteretic soil behavior at the foundation vicinity, 3) geometric nonlinearities at the foundation including gapping and sliding and 4) nonlinear behavior of the structure such as concrete cracking. While all these nonlinear effects can be accounted for in a single analysis using the NLSSI methodology, it is important that the analyst and peer reviewer determine the effects that are most significant in each problem and consider those effects only. A broad view of the process of performing NLSSI analysis is illustrated in Figure 1.

The NLSSI methodology was developed through continuous benchmarking involving comparison of the SSI results from the commercial time-domain code LS-DYNA, and the frequency-domain code, SASSI. A generic NPP structure placed on a representative soil site was used for these benchmarking analyses. The finite element model of this structure is presented in Figure 2.

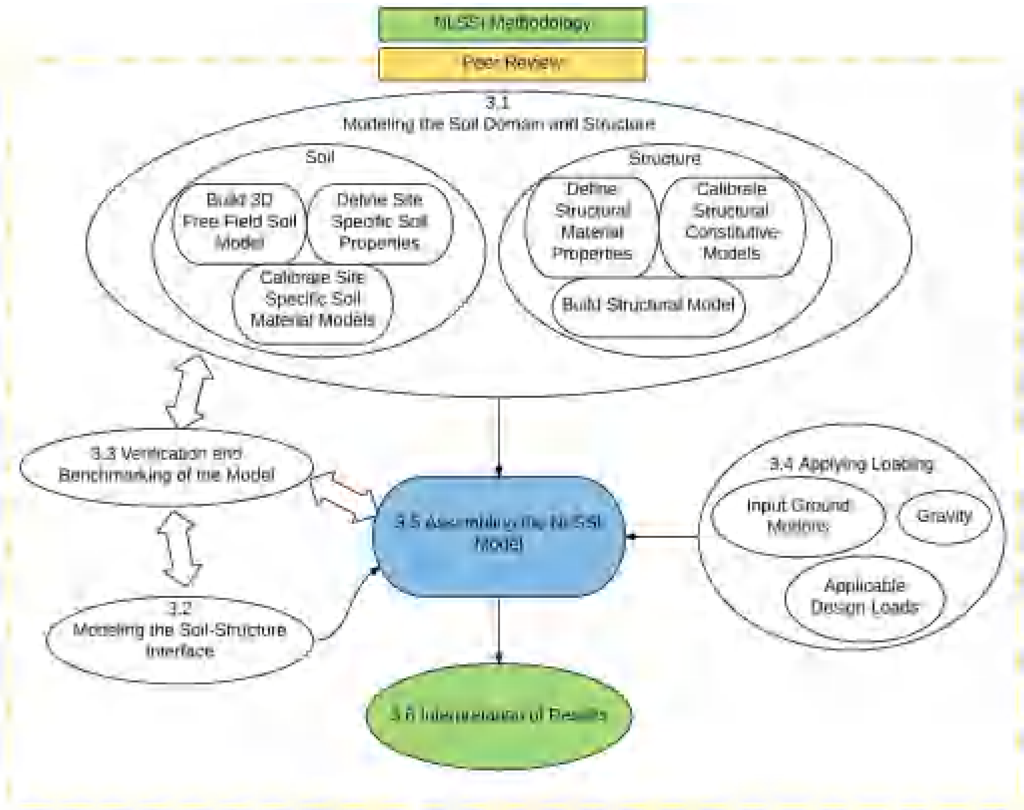

Figure 1: The time-domain NLSSI methodology

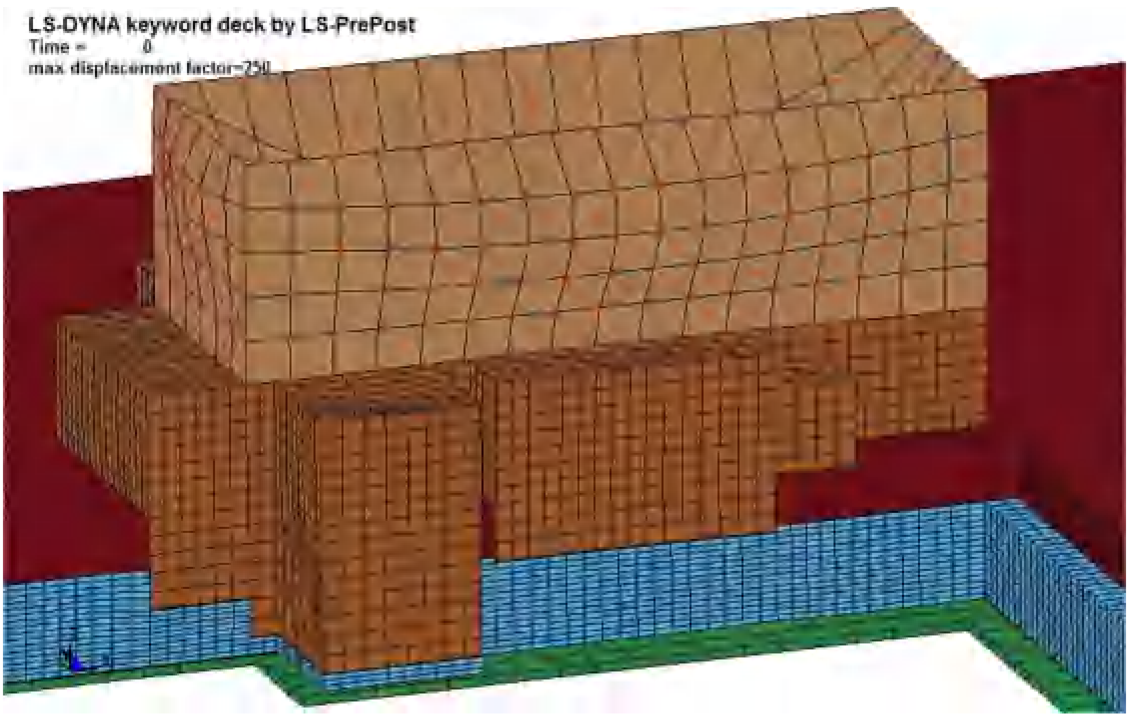

Figure 2: Finite-element NLSSI model of a generic nuclear power plant on a representative soil site used in development of the NLSSI methodology

As illustrated in Figure 1, building an NLSSI model requires 1) modeling the soil domain and the structure, 2) modeling the interface between the soil domain and the structure and 3) applying seismic loading, while verifying the performance of each of these models independently. The procedures and necessary precautions for building these models are explained in the subsequent sections, the numbers of which, are also indicated in Figure 1. The NLSSI methodology can be implemented in practice through two methods: 1) the direct method, and 2) the domain reduction method. These methods differ primarily in the procedures of ground motion input: the direct method is capable of simulating only vertically propagating shear or compressive wave input, and the domain reduction method is capable of simulating a three-dimensional wave-field input. Since the NLSSI methodology, as presented in this paper, focuses on vertically propagating input, the modeling procedures decsribed in the subsequent sections correspond to the direct method of SSI analysis.

3.1 Modeling the soil domain and structure

3.1.1 Modeling the soil domain

The soil domain in an NLSSI analysis serves the following purposes: 1) simulating the site effects on input ground motion, and 2) dissipating the scattered waves before reaching the lateral boundaries. The soil domain therefore has to be large enough to dissipate the scattered waves from the structure, and should be constrained at the lateral boundaries in such a way that the response at these boundaries is almost identical to the free-field response. The ground motion input is applied at the base of the soil domain either as a force input (when an outcrop motion is available) or an acceleration input (when a within profile motion is available) as explained in Section 3.4. An illustration of the NLSSI model in the direct method is provided in Figure 3.

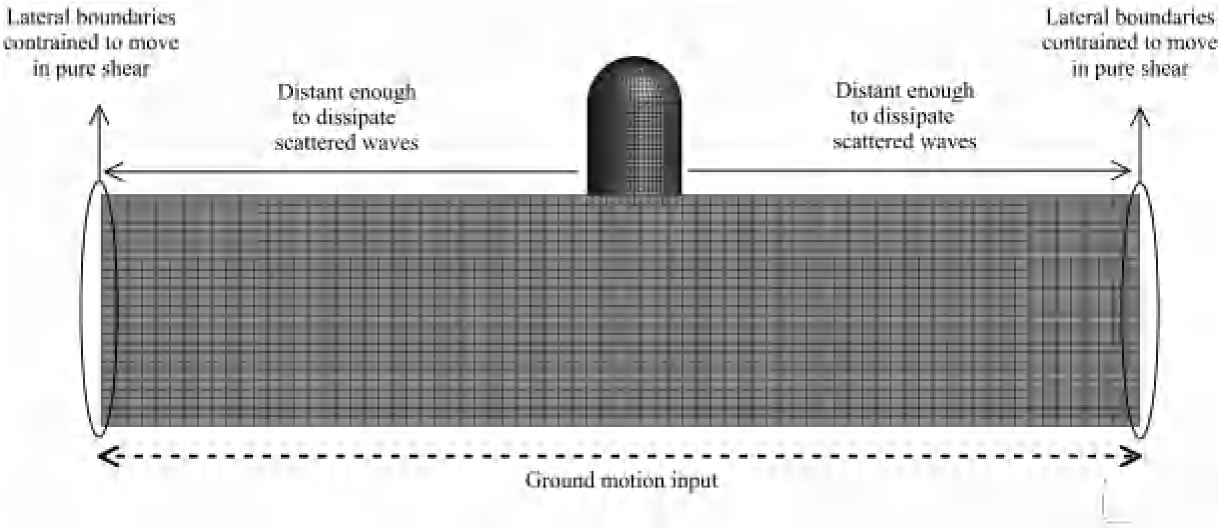

Figure 3: Illustration of the soil domain in direct method of NLSSI analysis (Bolisetti and Whittaker 2015)

The first step in building the soil domain model is to gather the site-specific soil material properties. These properties, which include shear wave velocity profile, unit weight profile, Poisson's ratio and the dynamic properties such as backbone curves and damping curves. These properties are typically measured from laboratory measurements or taken from existing databases of similar soils. The soil properties are then used to calibrate the nonlinear soil constitutive model of the numerical code. The parameters of the calibrated model can be verified by performing 'shear test' on a single solid element as described in Section 3.3.

The second step is to build the finite element mesh of the soil domain. The required mesh density of the soil domain depends on the soil properties, element formulation (constant strain, fully integrated, etc.), integration technique (implicit or explicit) and the desired maximum frequency of analysis. The soil domain mesh should be dense enough to adequately transmit the seismic motions up to the cut-off frequency and capture the nonlinear soil behavior, especially at the foundation vicinity, but should also be optimized to reduce the computation time. For a soil domain mesh built with constant-strain elements, the desired frequencies of analysis can be captured if the the longest dimension of each element (Δh), is given by

$$\Delta h \leq \frac{v_s}{10 * f_{\max}} \tag{1}$$

where $f_{\max}$ is the cutoff frequency, and ν_s is the small-strain shear wave velocity (Jeremić, Jie et al. 2009). Note that the maximum element size depends on the shear wave velocity and should therefore be calculated for each soil layer. Equation 1 is based on the assumption that

10 elements per shortest wavelength (which corresponds to the maximum frequency) allows adequate propagation of all the frequencies of the analysis. Figure 4 presents a comparison of the displacement snapshots of two soil columns with the same material properties but different mesh densities. The soil columns are subjected to a simple wavelet. The figure shows a clearer propagation of the wavelet when 10 elements per wavelength are used and noisy propogation when five elements per wavelength are used.

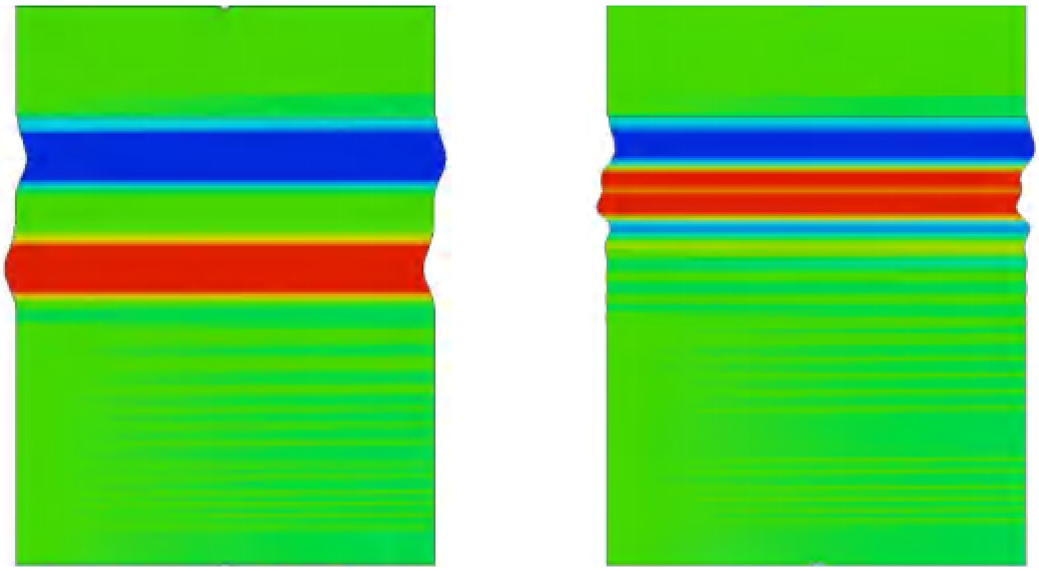

Figure 4: Comparison of displacement snapshots in a soil column when 10 elements per wavelength are used (left) and five elements per wavelength are used (right)

After determining the mesh density according to the considerations described, the plan dimensions of the soil domain are chosen. The lateral dimensions depend on the intensity of scattered waves, which depends on the mass of the structure and intensity of shaking, and the inherent damping in the soil domain. Since nonlinear soil material exhibits hysteretic damping, a nonlinear soil domain dissipates the scattered waves more efficiently than an elastic soil domain, thereby leading to a reduction in the domain size when nonlinear soil models are used. The lateral dimensions of the soil domain are chosen on a trial-and-error basis, by verifying that 1) the surface response of the soil domain at the farthest point from the structure is almost identical to the free-field response calculated from a separate one-dimensional site-response analysis, and 2) the structural response does not change with a further increase in the lateral dimensions. The depth of the soil domain is determined by the depth at which the input ground motion is available, or the depth to which nonlinear site response is to be captured. The soil domain should also be deep enough such that the vertical dynamic stiffness of the foundation is captured. Typically a soil domain depth of one to two times the largest plan dimension of the foundation below the foundation itself can produce reasonable results. As a final consideration, it should be noted that the soil domain should be large enough such that the mass of the soil domain is considerably greater than the mass of the structure. This ensures that the shaking of the structure does not affect the dynamic response of the whole soil domain.

The lateral boundaries of the soil domain should ideally 1) simulate the stress equilibrium at the lateral boundaries to account for the rest of the soil domain that is not included in the finite domain model and 2) dissipate the scattered waves that arrive at the boundary. The latter can be achieved by using absorbing boundary models at the lateral boundaries that absorb the incoming waves and minimize reflections. Absorbing boundary models have been developed and implemented in commercial finite element programs, but are limited to linear elastic materials. No absorbing boundaries have yet been developed for nonlinear materials, to the knowledge of the authors. Therefore a reasonable approach to model the lateral boundaries is to build a large soil domain such that the scattered waves are dissipated before reaching the boundary, and constrain the boundary nodes to move in pure shear (thus simulating a free-field stress equilibrium). The pure shear constraint can be achieved by constraining the lateral boundary nodes at each elevation to move together horizontally and vertically, assuming that the input ground motion consists of vertically propagating waves.

3.1.2 Modeling the superstructure

Superstructure models for traditional SSI analysis comprise of simple, lumped-mass stick models that capture the mass, stiffness and natural frequencies of the actual structure. These idealized structures are computationally efficient and may provide a reasonable estimate of the linear response. However, since most of the computation time in NLSSI analysis is a result of the large soil domain, a more comprehensive structural model can be used for these analyses with a minor increase in computational requirements. Additionally, these models can be used with the widely available nonlinear constitutive models to simulate nonlinear structural behavior during beyond-design basis earthquakes.

Similar to the soil domain modeling, the first step in structural modeling is to calibrate the numerical constitutive model with existing material property data. This data can be gathered from laboratory testing and/or existing databases. The structural model is then assembled and verified as described in Section 3.3. When using an explicit solver, the critical time-step of the NLSSI model is likely controlled by a structural element. Therefore to avoid large computation times, the analyst must be wary of this time step, and avoid using small element sizes in the structural model. In order to have an optimal computation time, it is recommended that the structural model is meshed such that the critical time step of the structural model is similar to the critical time step of the soil domain model.

3.2 Modeling the soil-structure interface

The soil-structure interface is a key component of NLSSI model for the simulation of geometric nonlinearities at the foundation such as gapping and sliding. The soil-structure interface can be modeled using 1) contact elements that are available in most commercial finite-element codes, 2) nonlinear springs and dashpots connected between the soil and foundation nodes that have zero strength in tension and simulate the necessary frictional behavior in shear, and 3) thin layers of nonlinear hysteretic soil that has zero strength in tension and necessary peak shear strength to simulate friction.

The choice of contact interface depends on the numerical code, and the expected significance of gapping and sliding in the structural response. If gapping and sliding effects are expected to be only moderate, they can be simulated by using a thin, soft soil layer between the foundation and the soil domain, with the properties calibrated to model a certain friction behavior. In cases where significant gapping and sliding are expected and their effect on components such as connecting pipes, etc., are to be evaluated, the soil-structure interface can be easily modeled using contact models. However these models are known to result in numerical noise, or in some cases, spurious ‘spikes’ in the acceleration response histories. In such cases, the soil-structure interface can be modeled using nonlinear springs and dashpots between the foundation and soil nodes. The properties of these spring and dashpot elements can be adjusted to the desired uplift and sliding behavior. Additionally it is recommended that the modeled soil-structure interface is independently verified using simple analyses as described in Section 3.3.

3.3 Verification of the model

Model verification and benchmarking are crucial aspects of the NLSSI methodology in order to avoid modeling errors. Verification is the process of determining if the numerical model solves the required mathematical problem. Benchmarking is the process of comparing the results calculated from the numerical model being used with other verified solutions or numerical codes. The NLSSI methodology includes verification and/or benchmarking of all the model components including the soil domain model, structural model, soil-structure interface and the fully assembled NLSSI model. Through the verification process, it is important to understand the physical behavior that is being simulated at each step as identified in Figure 1. The verification and benchmarking processes for the NLSSI steps are summarized in Table 2 and are performed as follows:

Table 2: Recommended approaches for verification or benchmarking of NLSSI modeling steps

Modeling description section	**NLSSI modeling step**	**Parameters to be verified or benchmarked**	**Verification and/or benchmarking approach**
3.1.1	Calibration of site-specific soil material models	Constitutive model including energy dissipation (damping)	Single element models
3.1.1	Building 3D soil model	Material model; boundary conditions	Comparison with equivalent linear site-response codes for low-amplitude inputs
3.1.2	Calibration of structural constitutive models	Constitutive models including energy dissipation (damping)	Single element models
3.1.2	Building structural model	Fixed-base frequencies; structural damping	Modal analysis and comparison of fixed-base responses with other verified codes
3.2	Modeling soil-structure interface	Co-efficient of friction; lateral force required for uplift	Analysis of simple models or comparison with closed form solutions
3.5	Assembling NLSSI model	Absorbing boundary conditions; size of soil domain	NLSSI analyses with increasing soil domain sizes
3.5	Assembling NLSSI model	Final benchmarking	Comparison with equivalent-linear SSI codes for low-amplitude inputs

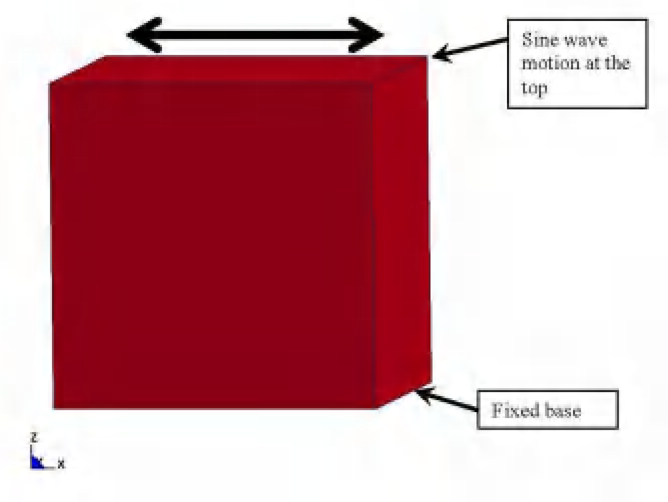

Figure 5: Sample single element model for verification of the soil model parameters

- Calibration of site-specific soil constitutive model: After determining the site-specific elastic and dynamic properties, the corresponding paramters of the nonlinear hysteretic soil constitutive model need to be calculated. Calibration of the constitutive model can be accomplished through analyses of a single element, typically an eight-noded brick (which is generally used to build the 3D soil domain). A cyclic load is applied to the top as shown in Figure 5. Using the results from this analysis, it can be demonstrated that the stress-strain curve and the hysteretic energy dissipated is similar to that recorded during laboratory testing.
- Verification of soil domain model: After building the soil domain it has to be verified for the material properties and boundary conditions. This verification can be performed by comparing the response of the free-field soil domain model (without the structure) with equivalent linear site-response analysis codes at low levels of ground motion.
- Calibration of structural constitutive model: The Nuclear Regulatory Commission (NRC) requires that the structural response of NPPs be elastic during the occurrence of a design basis ground motion. However during beyond-design basis motions the structural response maybe in-elastic. Similar to the calibration and verification of soil constitutive models, the structural constitutive models need to be calibrated to the material properties. The calibrated models can then be verified through simple analyses on single elements.
- Benchmarking of fixed-base structural model: After assembling the structural model it has to be benchmarked for the fixed-base natural frequencies, modal mass participation ratios and damping. Benchmarking can be performed through a modal analysis of the fixed-base model that results in natural frequencies and modal mass participation ratios,

which are compared to those calculated from a verified numerical code. Finally a response-history analysis can be performed and the results can be benchmarked against those calculated from an established code in order to verify structural damping.

- Modeling the soil-structure interface: The soil-structure interface should be verified for the desired sliding and gapping behavior before performing assembling the NLSSI model. The sliding behavior can be verified by applying a slowly increasing horizontal load on the foundation and comparing th sliding response to the desired response, for instance, a bilinear force-displacement response for a Coulomb friction model. Gapping behavior can be verified by applying a slowly increaseing vertical load at the edge of the foundation and calculating the force at which uplift occurs. This result can be verified with simple hand calculations to calculate the uplift force.
- Assembling the NLSSI model: After assembling soil, structure and interface models to build the NLSSI model, it is necessary to verify and benchmark the performance of absorbing boundary conditions, and adequacy of the soil domain size. As described in Section 3.1.1, verification of the soil domain size is performed through a trial-and-error basis until the structural response remains unchanged with an increase in the soil domain size. After the verification of the individual components of the NLSSI model, a final benchmarking analysis is strongly recommended. This benchmarking should be performed by comparing the results of the NLSSI analysis at low levels of ground motion (that result in almost linear soil, structure and interface behavior) to those calculated using an established SSI analysis code (e.g., SASSI).

3.4 Applying loading

Input ground motions corresponding to vertically propagating wavefields are be applied at the base of the soil domain in the direct method of NLSSI analysis. When a within motion is available, or when the soil or rock below the soil domain is assumed to be rigid, the input ground motion can be applied as an acceleration, velocity or displacement input. When an outcrop motion is available, or the elasticity of the soil or rock below the soil domain needs to be captured, the input ground motion is applied as a force history, and an absorbing boundary condition is provided at the base. Since an outcrop motion is essentially the incoming wave, which travels upwards and reflects back into the soil halfspace, the absorbing boundary condition is provided to absorb the reflections and avoid ‘trapping’ the energy in the soil domain. The process of specifying an outcrop input is illustrated in Figure

6. The three components of ground motion can be applied simultaneously to the NLSSI model with the same approach.

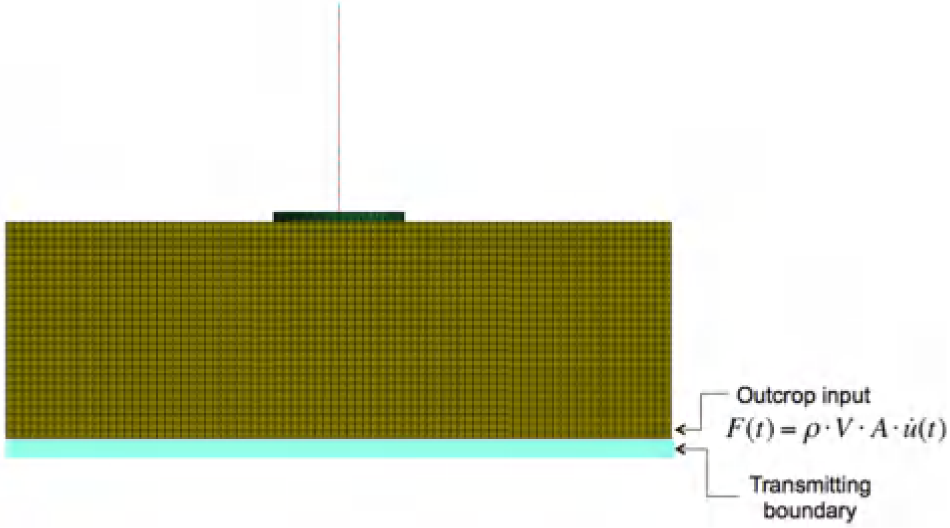

Figure 6: Specification of outcrop input in the direct method of NLSSI

The NLSSI methodology also requires the application of gravity and dead loads prior to the application of the seismic input. These vertical loads can be applied using a static or pseudo-static analysis to create the in-situ stresses in the soil and the soil-structure interface. These stresses are necessary to accurately model the soil and the interface behavior.

An alternative method of providing ground motion input for NLSSI is the domain reduction method developed by Bielak, Loukakis et al. (2003). This is a two-step approach to input a complex three-dimensional wave field for the seismic analysis of a soil-structure system. The first step typically involves a large-scale source-to-site type simulation that results in a set of free-field motions (or equivalent forces) at the 'domain reduction boundary', which around the area of interest of nonlinear SSI. These motions (or equivalent forces) are then input to the NLSSI model (which is slightly different from the NLSSI model for the direct method) to calculate the structural response. The NLSSI methodology is yet to be extended for three-dimensional input, and is therefore yet to include the domain reduction method.

3.5 Assembling the NLSSI model

After building and verifying the individual components of the NLSSI model, namely, the soil domain, structure and the soil-structure interface, these components are assembled to build the final NLSSI model. The loads are then applied and benchmarking analyses are performed as described in Section 3.3. The verified and benchmarked NLSSI model can then be used for nonlinear SSI analyses.

3.6 Interpretation of results

Similar to SSI analysis in the frequency domain, some considerations need to be made while interpreting the results from a nonlinear SSI analysis. Specifically, these considerations correspond to the results in the higher frequencies, where in some cases the response can be a result of spurious oscillations that are a result of the numerical model, for instance, contact elements. For all NLSSI analyses, the results are considered credible only until a cutoff frequency. This frequency is the smaller of 1) the maximum frequency that can propagate through the soil domain mesh according to Equation 1, and 2) the Nyquist frequency of the ground motion.

4. Validation of the NLSSI methodology

Developing confidence in accurate numerical predictions of the seismic response of nuclear facilities relies heavily on verification and validation (V&V) process. Verification and validation procedures are the primary means of assessing accuracy in modeling and computational simulations. Verification provides evidence that the mathematical models are solved correctly and involves comparing the numerical results with known closed-form solutions. Validation provides evidence that the correct model is solved, and typically involves comparisons with experimental data (Oberkampf and Trucano 2008).

There are three nonlinear behaviors in a nonlinear SSI problem that should be independently validated: soil nonlinearity, structural nonlinearity, and contact interface nonlinearities (sliding and/or separation). Validation can be achieved by comparing results of the analytical model with experimental data. Some references for validating soil, concrete, and contact models and some experimental results are provided in (Maekawa, et al. 2008).

In the early 1980's the Electric Power Research Institute (EPRI) constructed two scaled nuclear containment structures in Lotung, Taiwan. The site and structures were instrumented to obtain soil site response and structural response during multiple earthquake events. The site was instrumented with a three arm surface array and two downhole arrays of accelerometers (Glaser and Leeds 1996). The Lotung array recorded several large amplitude seismic ground motions that were used to validate frequency-domain codes. However, the complexity of the soil site and three-dimensional nature of the wavefield drove the researchers working on validating the equivalent linera codes to make several assumptions to achieve a closer match with recorded results.

Future testing for validating NLSSI will need to consider larger soil strains and higher frequencies than have typically been gathered to date. The tests will need to consider both material and geometric nonlinearities, starting with simple, one-dimensional site-response tests and then moving to more complex tests. After the individual nonlinear behaviors are validated, large-scale tests can be performed for the validation of complete SSI systems.

5. Implementation in Appendix B of ASCE Standard 4

Nonlinearities in the soil and structure become more pronounced with increasing intensity of earthquake shaking. When approximating nonlinear behavior using numerical tools it is important to reasonable the physical behavior.

Appendix B of the forthcoming ASCE Standard 4 provides non-mandatory guidance for performing nonlinear, three dimensional SSI analysis in the time domain. This guidance can be used to explicitly consider nonlinearities in the materials and/or geometry such as loss of contact between soil and structure, and inelastic action in soil and structure. The guidance notes that nonlinear response of a nuclear facility should include the following: material nonlinearity, hysteretic behavior (in soil and/or structure), significant uplift or sliding of the foundation, static and dynamic soil pressure effects on deeply embedded structures, local soil failure at the foundation-soil interface, nonlinear coupling of soil and pore fluid, nonlinear effects involving gapping between the structure and surrounding soil at the soil-structure interfaces, and nonlinear behavior of seismic isolators when the structure is base isolated. NLSSI can be used to provide element forces and deformations for superstructure component checking and in-structure response spectra, or foundation input motions, which is the first step in a multistep analysis.

Appendix B of ASCE 4 does not alter other guidance on the use of three soil columns [best estimate (BE), lower bound (LB), upper bound (UB)] for SSI analysis or peak smoothing and broadening of in-structure response spectra. Appendix B provides information on the development of finite element meshes for analysis, ground motion input, nonlinear constitutive models, analysis results and interpretation, and verification and validation.

6. Summary and conclusions

This paper presents the NLSSI methodology for application to nuclear facilities for both design and beyond-design basis ground motions. The current methodology assumes vertically propagating shear waves and implements the effects of nonlinear soil behavior and geometric nonlinearities, such as gapping and sliding.

Seismic design of nuclear facilities is expected to be conservative, whereas seismic risk calculations are expected provide best estimate results so that the possibility of masking other concerns is minimized. Seismic risk calculations focus on beyond-design basis earthquake ground motions that are larger in amplitude than design basis earthquake. Therefore consideration of nonlinear effects is likely more important in these risk calculations. However, NLSSI should be considered also for the design of nuclear facilities located at sites with high seismic hazard.

7. Ongoing research and development

Spears and Coleman (2014) developed, verified and benchmarked the methodology presented in this paper by comparing the NLSSI results to those calculated using SASSI for multiple levels of ground motion. They considered a representative site in the western United States for this study and are currently extending the methodology for a site and seismic hazard representative of the central and eastern United States. Additional research at Idaho Laboratory includes the investigation of an efficient method of modeling the soil-structure interface to avoid the high-frequency noise. This method will be employed in an advanced seismic probabilistic risk assessment study, that will examine the effect of gapping and sliding on a generic nuclear power plant structure in terms of the risk of core damage.

Future work includes the streamlining of the NLSSI process in commerical finite-element codes by 1) identifying the various practical issues while using these codes and working with their developers to resolve these issues, 2) automating the model building process and make the usage of NLSSI methodology much easier for industry applications. Additionally, the NLSSI methodology will need to be extended to enable the examination of 1) inclined seismic waves coupled with gapping and sliding of foundations atop soil, 2) inclined seismic waves coupled with gapping and sliding of deeply embedded structures, 3) soil dilatancy, 4) soil liquefaction, 5) surface waves, 6) buoyancy, 7) concrete cracking and 8) seismic isolation. Additional research and development is needed to produce verified and validated numerical tools that reasonably approximate these behaviors.

Acknowledgments

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References

Anderson, L. M., T. Elkhoraibi and F. Ostadan (2013). Validation of SASSI2010 Solution Methods through Independent Verification Using SAP2000 for Deeply Embedded Structures with Large Footprints. Transactions: 22nd International Conference in Structural Mechanics in Reactor Technology - SMiRT 22. San Francisco, California.

ANSYS Inc. (2013). ANSYS 13.0. Canonsburg, Pennsylvania, ANSYS Inc.

ASCE (1998). Seismic Analysis of Safety-Related Nuclear Structures and Commentary. ASCE 4-98. Reston, Virginia, American Society of Civil Engineers.

ASCE (2005). Seismic Design Criteria for Structures, Systems and Components for Nuclear Facilities. ASCE 43-05. Reston, Virginia, American Society of Civil Engineers.

ASCE (forthcoming). Seismic Analysis of Safety-Related Nuclear Structures and Commentary. ASCE 4-**. Reston, Virginia, American Society of Civil Engineers.

Bielak, J., K. Loukakis, Y. Hisada and C. Yoshimura (2003). "Domain Reduction Method for Three-Dimensional Earthquake Modeling in Localized Regions, Part-I: Theory." Bulletin of the Seismological Society of America **93**(2): 817-824.

Bolisetti, C. and A. S. Whittaker (2015). Site Response, Soil-Structure Interaction and Structure-Soil-Structure Interaction for Performance Assessment of Buildings and Nuclear Structures. University at Buffalo, The State University of New York, Buffalo, NY, Multidisciplinary Center for Earthquake Engineering Research (MCEER).

Bolisetti, C., A. S. Whittaker and J. L. Coleman (2015). "Frequency- and Time-Domain Methods in Soil-Structure Interaction Analysis." Nuclear Engineering and Design **Under Review**.

Computers and Structures Inc. (2011). SAP2000 - Structural Analysis Program, Version 11.0.0. Berkeley, California, Computers and Structures, Inc.

Coronado, C., L. M. Anderson and F. Ostadan (2013). Verification of SASSI Extended Subtraction Method for Computing the Seismic Response of Deeply Embedded Structures. 22nd International Conference in Structural Mechanics in Reactor Technology - SMiRT 22. San Francisco, California.

Dassault Systèmes (2005). ABAQUS - Finite Element Analysis Software. Providence, Rhode Island, Dassault Systèmes.

Glaser, S. D., and Leeds, A. L. (1996). “Estimation of System Damping at the Lotung Site by Application of System Identification,” Division of Engineering Colorado School of Mines, NIST GCR 96-70.

Jeremić, B., G. Jie, M. Preisig and N. Tafazzoli (2009). "Time Domain Simulation of Soil-Foundation-Structure Interaction in Non-Uniform Soils." Earthquake Engineering & Structural Dynamics **38**(5): 699-718.

Jeremic, B., A. Kammerer, N. Tafazzoli and B. Kamrani (2011). The NRC ESSI Simulator. 21st International Conference in Structural Mechanics and Reactor Technology: SMiRT 21. New Delhi, India.

LSTC (2013). LS-DYNA Keyword User's Manual - Version R 7.0. Livermore, California, Livermore Software Technology Corporation. **1**.

Lysmer, J., F. Ostadan and C. Chin (1999). SASSI2000 - A System for Analysis of Soil-Structure Interaction. Berkeley, California, University of California.

Maekawa, K., et al. (2008). Practitioners' guide to finite element modelling of reinforced concrete structures. International Federation for Structural Concrete (fib). **45**.

Oberkampf, W. L. and T. G. Trucano (2008). "Verification and Validation Benchmarks." Nuclear Engineering and Design **238**(3): 716-743.

Ostadan, F. and N. Deng (2011). SASSI2010 - A System for Analysis of Soil-Structure Interaction, Version 1.1. San Francisco, California, Geotechnical and Hydraulic Engineering Services, Bechtel National Inc.

Spears, R. and J. Coleman (2014). Nonlinear Time Domain Soil-Structure Interaction Methodology Development. Idaho Falls, Idaho, Idaho National Laboratory.

Willford, M., R. Sturt, Y. Huang, I. Almufti and X. Duan (2010). Recent Advances in Nonlinear Soil-Structure Interaction Analysis using LS-DYNA. Proceedings of the NEA-SSI Workshop. Ottawa, Canada.

Xu, J., C. Miller, C. Costantino and C. Hofmayer (2006). Assessment of Seismic Analysis Methodologies for Deeply Embedded Nuclear Power Plant Structures, U.S. Nuclear Regulatory Commission, Washington, District of Columbia