

CONSIDERATIONS OF SOIL-STRUCTURE-INTERACTION FOR SEISMICALLY ISOLATED ADVANCED REACTORS

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ABSTRACT

Soil-structure-interaction (SSI) analysis is required for the design of conventionally founded large light water reactors in the United States (U.S.), unless the shear-wave velocity of the supporting rock exceeds 2,400 m/sec. This regulatory requirement is driven by the legacy assumption that reactor buildings are both very heavy and stiff: the attributes needed for significant SSI on soil sites. Many of the advanced and micro reactors proposed for possible deployment in the U.S. are one or two orders of magnitude lighter and smaller than gigawatt-scale large light water reactors (LLWRs). Seismic isolation, which can substantially reduce the impact of the seismic load case, adds flexibility at the base of advanced reactors and micro reactors. Taken together, SSI may be negligible for seismically isolated advanced and micro reactors and that is the subject of the study reported here. Analysis is performed for two seismically isolated advanced reactor buildings, a horizontally configured high temperature gas reactor and a fluoride salt-cooled high temperature reactor, for multiple isolation systems, soil domains, seismic inputs, and intensities of shaking. The preliminary analysis results indicate that SSI analysis is not needed for the design of seismically isolated advanced small modular and micro reactors unless the horizontal frequency of the supporting soil domain is very close to that of the isolation system.

INTRODUCTION

At the time of this writing, soil-structure-interaction (SSI) analysis is by-and-large required in the U.S. for the design of a nuclear power plant (NPP), unless it is founded on hard rock. The requirement for SSI analysis ensures that every new build NPP is by-and-large both site-specific and first-of-a-kind, challenging standardization. Many of the advanced and micro reactors, hereafter grouped together as *advanced* reactors, now being planned for deployment in the U.S. and abroad, are substantially smaller in plan, and at least an order of magnitude lighter, than the gigawatt-scale LLWRs in the U.S. operating fleet, for which SSI analysis has been performed. Seismic isolation, which is being considered as an integral design feature in some advanced reactor designs, can mitigate the impact of the seismic load case by adding 2D or 3D flexibility at the base of a reactor building. Taken together, SSI effects may be negligible for these reactors. Determining whether this hypothesis is correct is the focus of this paper.

Soil-structure-interaction analysis of nuclear facilities in the U.S. is governed by standards and regulatory guidance. Section 5 of ASCE/SEI Standard 4-16 (ASCE, 2017) presents requirements for SSI analysis of safety-related nuclear structures, developed for conventionally designed (i.e., non-isolated) nuclear facilities, including LLWRs. The standard permits fixed-based analysis (i.e., no soil domain) of a nuclear facility if *one* of the following three conditions are met: 1) the dominant frequency of the fixed-based nuclear facility is less than one half of the value calculated assuming the facility to be rigid and supported on springs representing the supporting soil domain¹ (also included in ASCE 4-

¹ The stiffness of the soil springs is calculated per tables and a figure presented in the Standard, developed by Whitman and Richart (1967). The half-space assumed in the development of the spring constants is assigned a single value of shear modulus, assumed to be low strain, and Poisson's ratio. This representation of the foundation medium is simple, as required for an analytical solution, but is not realized in practice.

98), 2) the facility is supported on rock with a shear wave velocity greater than 1,067 m/s and it is demonstrated that SSI effects are negligible, and 3) the facility is supported on rock with a shear velocity of greater than 2,400 m/s. Condition 1 will rarely be met for conventionally founded NPPs because the reactor building is stiff, with first mode frequencies in the range of 5 Hz to 15 Hz. Condition 2 obligates SSI analysis to demonstrate it is unnecessary. Condition 3 would only apply to reactors constructed on very hard rock. Section 3.7.2 of NUREG-0800, *Standard Review Plan* (USNRC, 2013) adopts a similar approach to condition 3, writing, “For structures founded on materials having a shear wave velocity of 8,000 feet per second [2,400 m/sec] or higher, under the entire surface of the foundation, a fixed base assumption is acceptable [SSI analysis is not required]”. Although these conditions and requirements are likely inappropriate for many advanced and micro reactors, they provided a starting point for the investigation introduced below.

To identify whether SSI effects are significant for base-isolated nuclear structures, response-history analysis is performed for three cases shown in Figure 1 for a sample seismically isolated advanced reactor building. In Case 1, the isolated reactor building is placed atop a soil domain and the coupled soil-structure system is modelled explicitly, per traditional practice. Seismic inputs are applied at the base of the soil domain. Case 2 involves 1D site-response analysis to capture surface free-field motion for the same inputs applied to the Case 1 model. In Case 3, the isolated reactor building is analyzed using the surface free-field motions generated from Case 2. If the responses of the reactor building and its equipment for Case 1 (depicted as **R1** in Figure 1) are similar or less than those for Case 3 (**R2**), SSI can be assumed to be inconsequential or beneficial, with no need for such analysis. Herein, *similar* is assumed to be (+20%, -0%), noting that greatly simplifying assumptions must be made for SSI analysis, including a) characterization of ground motion at depth, b) a horizontally layered soil stratigraphy, and c) 1D vertically propagating shear waves.

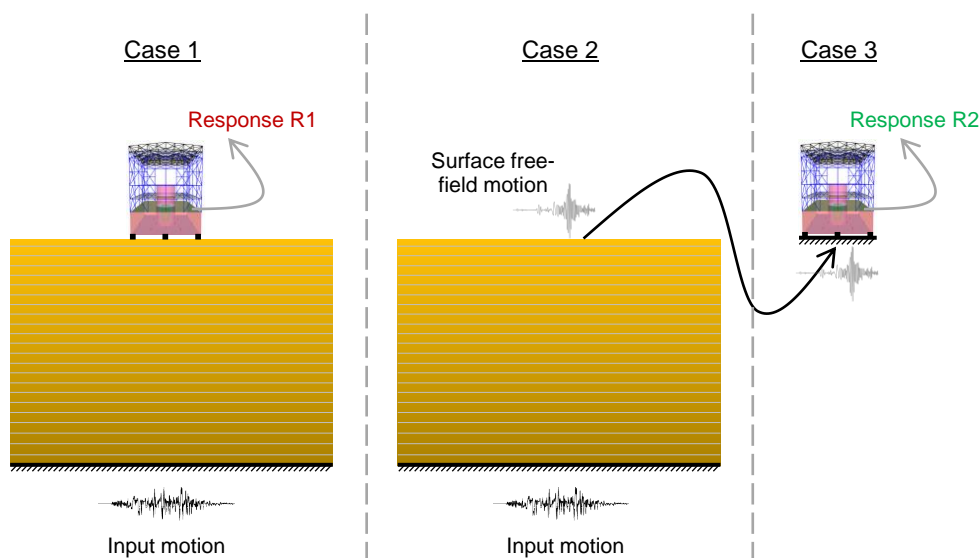


Figure 1. Response-history analysis cases 1, 2, and 3

Analysis is being performed for combinations of a) soil columns (10) that cover a range of sites across the U.S., with 7 forming part of the [Design Certification Documents](#) (DCD) that enabled the KEPCO APR1400 to be certified for use by the USNRC under the 10CFR Part 52 licensing framework, b) fundamentally different advanced reactor designs (3), c) 2D isolation systems (8), d) seismic inputs (3), and e) multiple (2) levels of ground shaking at depth, addressing moderate and severe seismic hazard. Results from a subset of the analysis cases are presented here: three soil columns (of 10), two reactor buildings (of 3), two isolation systems (of 8), one seismic input (of 3), and two levels of ground shaking (of 2). Results for all other combinations will be presented in Lal *et al.* (forthcoming). The following sections describe the two reactor buildings, two seismic isolation systems, one seismic input, and three soil columns addressed in this paper, followed by results of the preliminary analysis, closing remarks, and a draft recommendation.

REACTOR BUILDINGS AND ISOLATION SYSTEMS

[Boston Atomics](#) (BA) is developing a horizontal-compact high temperature gas reactor (HC-HTGR). The numerical model of an early version of the BA HC-HTGR (Stewart *et al.*, 2021), developed in the commercial finite element code SAP2000 (CSI, 2023), is presented in Figure 2a. The reactor building is a 10 m tall, reinforced concrete (RC) structure with plan dimensions of 14 m by 75 m. The building includes six major compartments: entry (red in Figure 2a), reactor maintenance (green), control operations (yellow), primary system (grey), fuel storage (purple), and refueling (orange). The entry, control operations, and fuel storage compartments are multi-story; all others are single story. The building has a 1.2 m thick perimeter RC wall adjacent to the primary system compartment, sized for radiation shielding; the remaining perimeter walls are 0.75 m thick. The floor slabs and partition walls between the compartments are 0.45 m thick. The basemat is 1.2 m thick. The primary system is aligned horizontally and integrates the reactor vessel (RV), steam generator (SG), and the heat circulator (HC). The total weight of the HC-HTGR building and its equipment is 11,900 tons. Modal analysis was performed to identify the dominant frequencies of the fixed-based reactor building and the primary system: 32 Hz and 8 Hz for the reactor building and 2 Hz and 1.6 Hz for the primary system in the H1 and H2 directions, respectively (see Figure 2a for the coordinate system). In the vertical direction, the mass of the building and the primary system is distributed across several modes with a frequency between 15 Hz to 40 Hz. Detailed information on the building can be found in Parsi *et al.* (2024).

Yu *et al.* (2023) developed a numerical model of an early version of the [Kairos Power](#) fluoride salt-cooled high temperature reactor in SAP2000. The numerical model of the FHR building in SAP2000 is presented in Figure 2b. The plan dimensions are 25 m by 25 m and the height from the basemat to the peak of the roof is 33 m. The building includes four floors: an RC basemat, a suspended RC floor (green in Figure 2b), and two composite floors (yellow). At the center of the building is a cylindrical RC shield structure (pink in Figure 2b) with a diameter of 8.8 m, which extends from the basemat to an elevation of 18 m. The basemat and the suspended floor are 1.2 m thick. The composite floors consist of a 76 mm concrete slab atop a 76 mm deep metal deck, modeled as a concrete slab with an equivalent thickness of 114 mm. Above the suspended RC floor, the building is non-safety class, and is framed in structural steel designed to commercial building standards. The part of the building below the suspended RC floor is assumed to be safety-related and houses one reactor vessel, one reactor vessel auxiliary cooling system (RVACS), and four primary heat exchangers. The total weight of the FHR building, including its equipment, is 8,920 tons. The dominant frequencies of the fixed-based building and its equipment in the two horizontal (vertical) directions are: 10 Hz (24 Hz) for the reactor building, 13 Hz (37 Hz) for the reactor vessel, and 7 Hz (27 Hz) for RVACS. More information on the HC-HTGR building can be found in Yu *et al.* (2023).

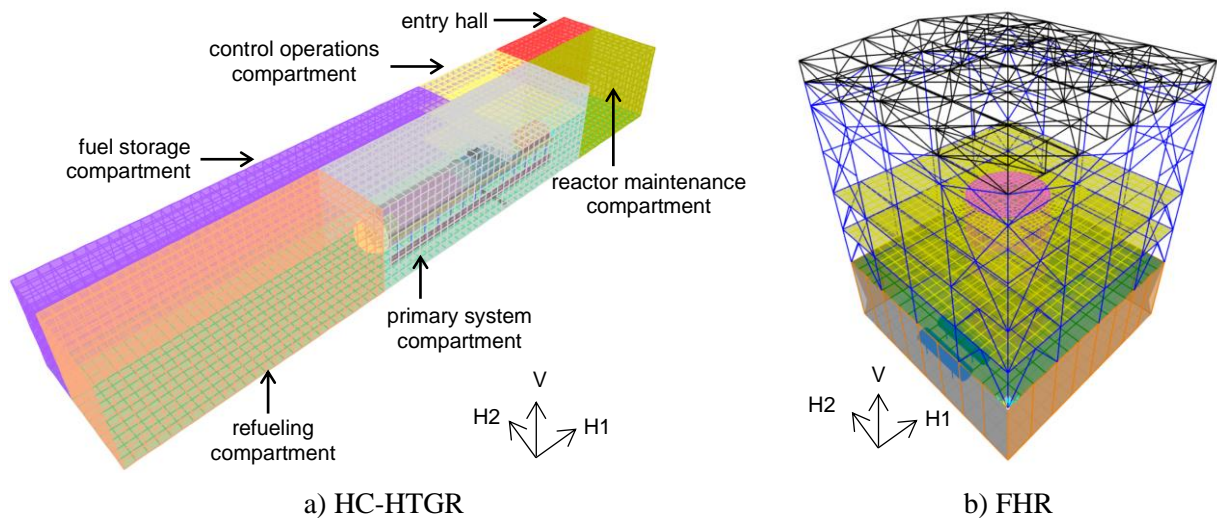


Figure 2. Numerical models of reactor buildings

Analysis results are presented below for two bilinear 2D horizontal isolation systems, realized using single Friction Pendulum (SFP) bearings. Their horizontal force-displacement behavior is a function of the sliding period (which depends on the geometry of the SFP bearings) and a coefficient of sliding friction. The two isolation systems herein are 1) IS1 with a sliding period of 2 seconds and coefficient of friction of 12%, and 2) IS2 with a 3-second sliding period and 9% friction. The HC-HTGR and the FHR buildings are supported on 48 and 17 isolators, respectively. The SFP bearings are modeled using two joint link elements in SAP2000 per the procedures outlined in Yu *et al.* (2023).

SEISMIC INPUTS

Results are presented in this paper for one two-component (horizontal directions, H1 and H2) earthquake motion extracted from the [PEER NGA-West 2 database](#), Record Sequence Number 164: a broadband motion. Acceleration response spectra of the as recorded, horizontal components are presented in Figure 3. The earthquake motions were amplitude scaled to a peak geomean horizontal acceleration of 0.2 g (GM1A) and 0.6 g (GM1B), representing moderate and high intensity shaking, respectively.

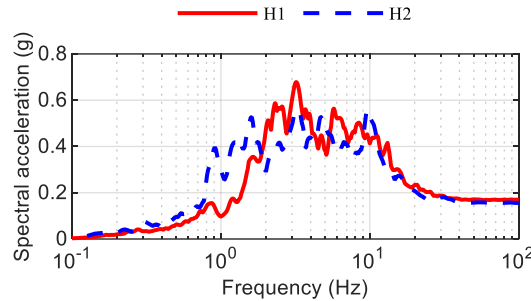


Figure 3. Acceleration response spectra, as-recorded horizontal components, 5% damping

SOIL DOMAINS

Three soil profiles are considered here, each 30 m deep, with the following characteristics: A) a constant low-strain shear wave velocity (V_s) of 300 m/sec and density (ρ) = 2,050 kg/m³, B) a constant low-strain V_s of 1,000 m/sec and ρ = 2,400 kg/m³, and C) a linearly varying low-strain V_s (ρ), from 300 m/sec (2,050 kg/m³) at the top of the soil column to 1,000 m/sec (2,400 kg/m³) at the bottom of the column. The lower bound value of 300 m/s for V_s is based on the NRC definition of minimum shear wave velocity for competent material supporting the foundation of an NPP per Section 3.7.2 of NUREG-0800. SSI effects are expected to be most significant for the soil column with the lowest shear wave velocity.

Herein, equivalent linear analysis is used to model the soil domains. Shear modulus and damping vary as a function of the shear strain developed during earthquake shaking. For the three soil columns herein, assumed to be sand, best-estimate shear modulus reduction and damping curves from Seed and Idriss (1970) are utilized. These are presented in Figure 4. A two-step process is used to capture the nonlinear behavior of soil in SAP2000. In the first step, an equivalent linear site-response analysis is performed in DEEPSOIL (Hashash *et al.*, 2016) to obtain strain-compatible soil layer properties, that is, shear modulus and damping values consistent with the strain demand in each layer. In the second step, a) the strain-compatible soil properties are used to generate linear soil models in SAP2000, and b) the models of the isolated reactor buildings are added to the soil domain to complete the soil-structure system for SSI analysis. The soil layers are modeled using eight-noded elastic solid elements. The thickness of the soil layers is selected to pass 30 Hz motion through the soil column, calculated per ASCE/SEI 4-16, as $V_{s0} / (10 \times f_{\max})$, where V_{s0} is the low-strain, shear wave velocity of a soil layer and f_{\max} is the maximum wave passage frequency. This provides a maximum soil layer thickness of 1 m for soil columns A and C, and 3.3 m for soil column B. Herein, a 1 m layer thickness is used for the three soil columns, resulting in 30 layers each. The vertical boundaries of the soil domains are

constrained to displace equally. Since DEEPSOIL performs 1D site-response analysis, strain-compatible shear modulus and damping are computed separately for each layer in the H1 and H2 directions, and then averaged for input to SAP2000. The seismic input is applied at the base of the soil profile, which is a reflecting boundary.

For SSI analysis, the soil is modeled as a finite domain. The plan dimensions of the soil domain are chosen such that waves radiating away from the reactor building are dissipated before reaching its lateral boundaries to avoid reflection. Herein, the plan dimensions are determined by trial-and-error such that the acceleration response at the boundaries of the soil domain in SAP2000, that is, the surface free-field response, is equal to that from site-response analysis in DEEPSOIL per step one above. The plan dimensions of the soil domains for the HC-HTGR and the FHR building were determined to be 5 and 6 times the largest plan dimension of the reactor buildings, respectively.

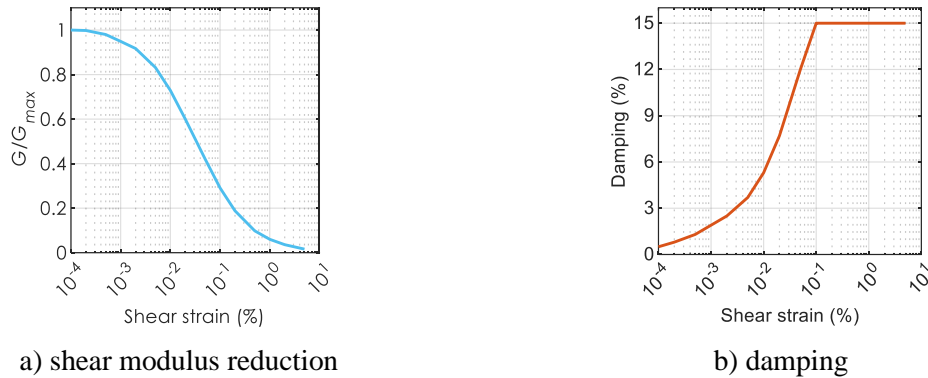


Figure 4. Variation of shear modulus and damping as a function of shear strain (adapted from Seed and Idriss (1970))

Modal analysis is performed in SAP2000 to determine the site periods (i.e., period of oscillation of the soil columns in the horizontal direction) for the three soil columns for very low excitation (i.e., G_{\max}) and at excitations GM1A and GM1B (i.e., shear modulus lower than G_{\max}). Table 1 presents results.

Table 1. Modal analysis results for the three soil columns: low strain and higher strain

Soil column	Seismic input	Site-period (frequency) based on low-strain shear modulus	Site-period (frequency) based on degraded shear modulus
A	GM1A	0.40 sec (2.5 Hz)	0.83 sec (1.2 Hz) ¹
	GM1B		1.25 sec (0.8 Hz) ¹
B	GM1A	0.12 sec (8.3 Hz)	0.14 sec (7.1 Hz)
	GM1B		0.19 sec (5.3 Hz)
C	GM1A	0.16 sec (6.3 Hz)	0.21 sec (4.8 Hz)
	GM1B		0.30 sec (3.3 Hz)

1. The substantial increase in site period is associated with soil softening and very large shear strains, for which equivalent linear analysis alone may not be sufficient. However, the use of equivalent linear analysis for this study, across all soil profiles, does not compromise either the outcomes or the recommendation.

RESULTS AND DISCUSSION

Acceleration histories from response-history analysis, utilized to derive acceleration response spectra, are extracted at the following locations: 1) near the boundaries of the soil domain on the ground surface,

that is, free-field motions, 2) on the basemat (immediately above the isolation system) of the two reactor buildings, 3) on the roof of the HC-HTGR building, and 4) atop the cylindrical reactor shield structure (RSS) in the FHR building. Monitoring locations 2, 3 and 4 represent potential points of attachment of safety-related equipment.

Figures 5 and 6 present acceleration response spectra at the surface free-field (top of the soil domain) and the rock horizon (base of the soil domain) for profile A and seismic inputs GM1A and GM1B, respectively. Figure 7 presents the corresponding results for soil profile C and GM1B. Geomean horizontal peak accelerations for all analysis cases herein are presented in Table 2. The geomean horizontal peak acceleration of 0.2 g for GM1A at the rock horizon is increased to 0.27 g at the surface free-field for profile A: an expected outcome for a stiff soil site subjected to low-to-moderate intensity earthquake motion. Deamplification of high-frequency content is evident in Figure 6: the 0.6 g geomean horizontal peak acceleration for GM1B at the base of the soil domain is reduced to 0.39 g at the surface free field. For soil profile C and GM1B (Figure 7), the geomean horizontal peak acceleration of 0.6 g at the base of the column is amplified to 1.8 g at the surface free field: an extreme ground motion.

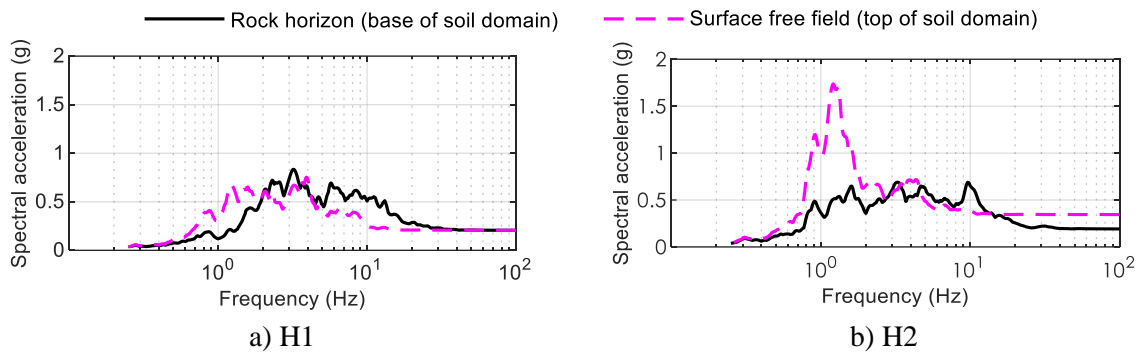


Figure 5. Acceleration response spectra, 5% damping, soil profile A, GM1A

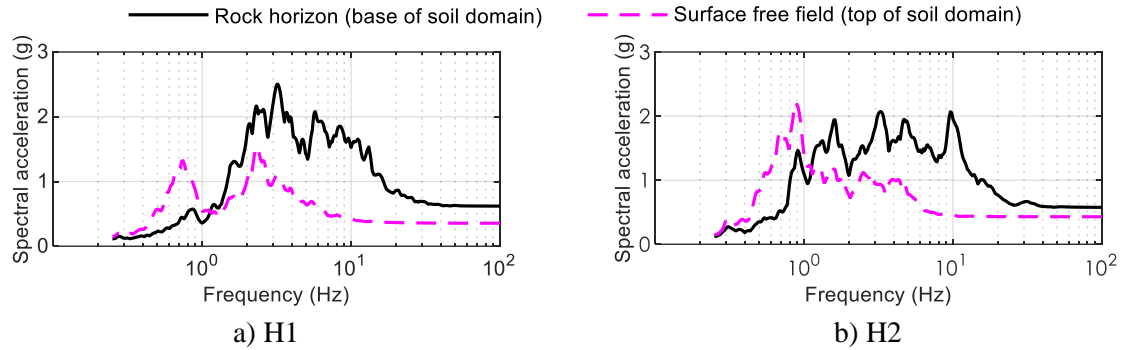


Figure 6. Acceleration response spectra, 5% damping, soil profile A, GM1B

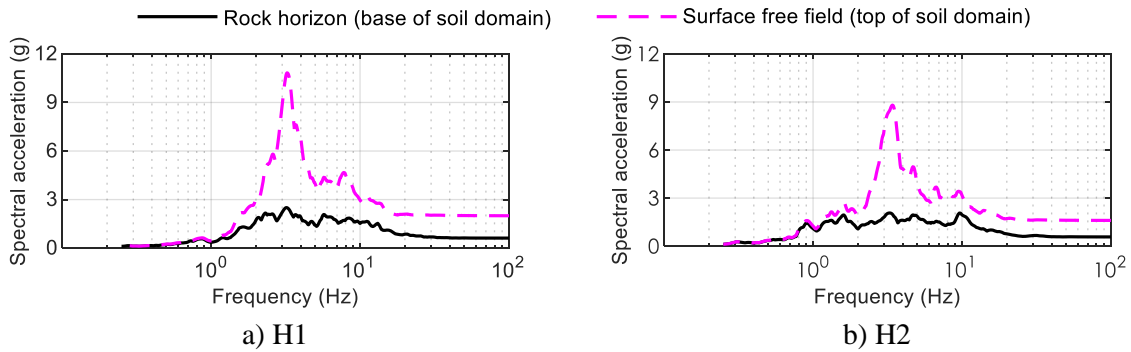


Figure 7. Acceleration response spectra, 5% damping, soil profile C, GM1B

Figures 8 through 15 present acceleration response spectra in the two horizontal directions, H1 and H2, at points of attachment of equipment in the two reactor buildings. Results are presented for frequencies between 1 and 50 Hz, likely capturing seismic input to all safety-class equipment.

The spectra in Figures 8, 10, 11, 12, 14, and 15, for case 1 (SSI analysis) and case 3 (analysis with surface-free-field inputs) are essentially identical, making clear that SSI analysis is not needed to compute in-structure spectra for design of safety-related equipment attached to the basemat for these combinations of soil domains, isolation systems, and ground motion inputs. Similar outcomes are observed at other points of attachment of equipment in these two buildings. For all these combinations of soil profiles, ground motion, and isolation systems, the ratios of the fundamental periods of the sliding isolation system to the degraded soil column is greater than 2: the basis of a rule-of-thumb used for 30+ years in the U.S. for the selection of isolation-system period, albeit not with a focus on equipment (or non-structural components). The Case 1 spectral ordinates in Figures 9 and 13, both involving the flexible soil profile A, the 2-second bilinear isolation system IS1, and the strong ground motion GM1B, are greater than those for Case 3, by up to 40%, for frequencies between 2 Hz and 50 Hz. For this combination of soil profile and ground motion input, the ratio of the fundamental periods of the isolation system (= 2 seconds) to the degraded soil column (= 1.25 seconds, see Table 1) is 1.6. The closeness of these two periods is the reason the Case 1 spectral demands exceed the Case 3 demands. Importantly, the use of a longer-period isolation system, the 3-second IS2, for the same combination of soil profile A and ground motion GM1B, does not trigger soil-structure-interaction: see Figures 10 and 14, wherein the Case 1 and Case 3 spectral demands on the basemat are identical for H1 and H2.

Table 2 presents geomean horizontal peak accelerations for the three soil profiles, multiple monitoring locations, and moderate and intense ground motion inputs, for Case 1 (SSI analysis) and Case 3 (analysis with surface-free-field inputs). Aside from soil profile A, isolation system IS1, and ground motion GM1B (cells shaded light red), the isolation systems substantially reduce the peak accelerations on the basemat of each reactor building, by between 38% (soil profile A, isolation system IS1, and ground motion GM1A) and 90% (soil profile C, isolation system IS2, and ground motion GM1B). The differences in the tabulated values for Case 1 and Case 3 may appear to contradict the data presented in Figures 8 through 15, but do not, because the peak accelerations along the axes H1 and H2 that are used to compute the geomean are a) measured at different times steps in a response-history analysis, and b) correspond to a frequency of 200 Hz.

Table 2. Geomean horizontal peak accelerations (g), three soil columns, two ground motion pairs, multiple monitoring locations

Soil profile	Location	GM1A				GM1B			
		IS1		IS2		IS1		IS2	
		Case 1	Case 3	Case 1	Case 3	Case 1	Case 3	Case 1	Case 3
A	Free field/input ¹	0.27		0.27		0.39		0.39	
	HTGR basemat	0.17	0.17	0.12	0.12	0.62	0.46	0.22	0.19
	HTGR roof	0.18	0.21	0.12	0.16	0.63	0.47	0.22	0.20
	FHR basemat	0.16	0.16	0.12	0.12	0.59	0.45	0.22	0.20
	FHR top of RSS	0.26	0.36	0.20	0.25	0.61	0.50	0.26	0.26
B	Free field	0.63		0.63		1.5 g		1.5 g	
	HTGR basemat	0.16	0.19	0.12	0.14	0.25	0.26	0.17	0.17
	HTGR roof	0.17	0.34	0.13	0.26	0.25	0.37	0.17	0.28
	FHR basemat	0.21	0.20	0.16	0.15	0.26	0.25	0.19	0.18
	FHR top of RSS	0.53	0.55	0.43	0.43	0.52	0.54	0.42	0.42
C	Free field	0.75		0.75		1.8		1.8	
	HTGR basemat	0.16	0.19	0.12	0.14	0.27	0.28	0.17	0.18
	HTGR roof	0.17	0.31	0.12	0.24	0.27	0.35	0.17	0.27
	FHR basemat	0.22	0.22	0.17	0.16	0.26	0.27	0.18	0.18
	FHR top of RSS	0.56	0.53	0.42	0.38	0.46	0.54	0.34	0.39

1. Represents the surface free-field acceleration for Case 1 and the input acceleration for Case 3 (see Figure 1)

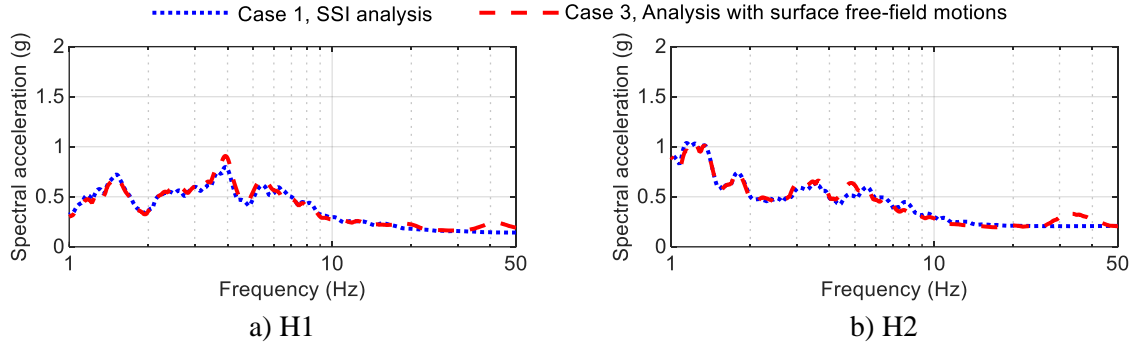


Figure 8. Acceleration response spectra on the basemat of the HC-HTGR building, 5% damping, soil profile A, isolation system IS1, ground motion GM1A

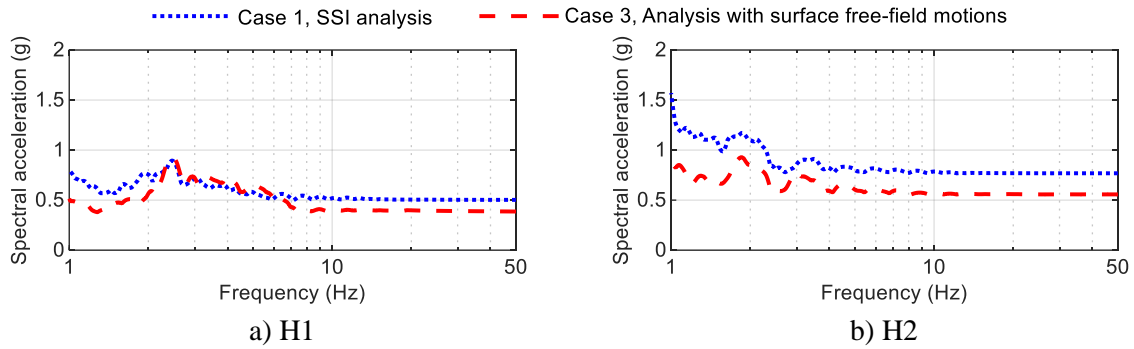


Figure 9. Acceleration response spectra on the basemat of the HC-HTGR building, 5% damping, soil profile A, isolation system IS1, ground motion GM1B

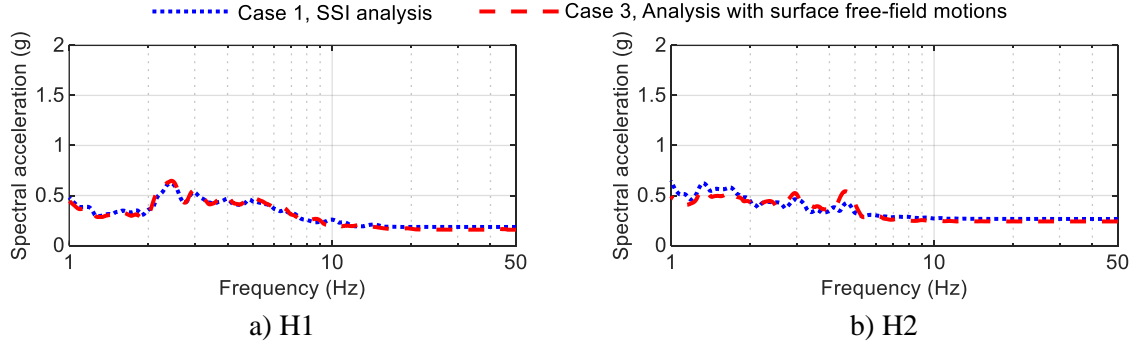


Figure 10. Acceleration response spectra on the basemat of the HC-HTGR building, 5% damping, soil profile A, isolation system IS2, ground motion GM1B

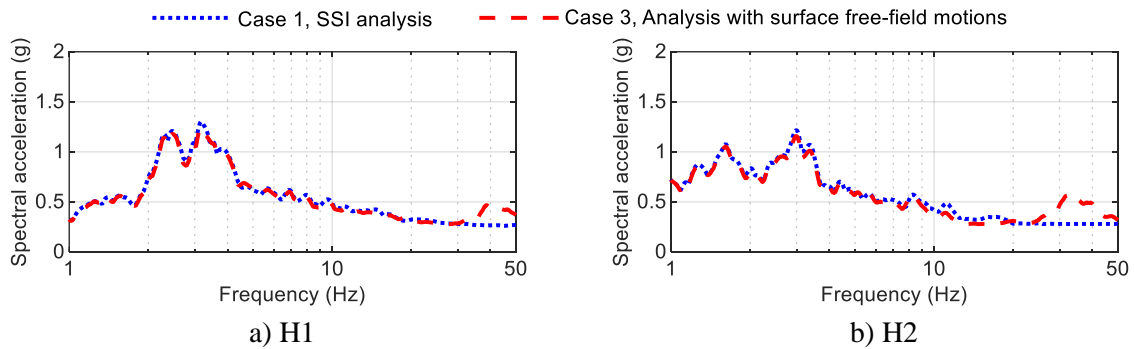


Figure 11. Acceleration response spectra on the basemat of the HC-HTGR building, 5% damping, soil profile C, isolation system IS1, ground motion GM1B

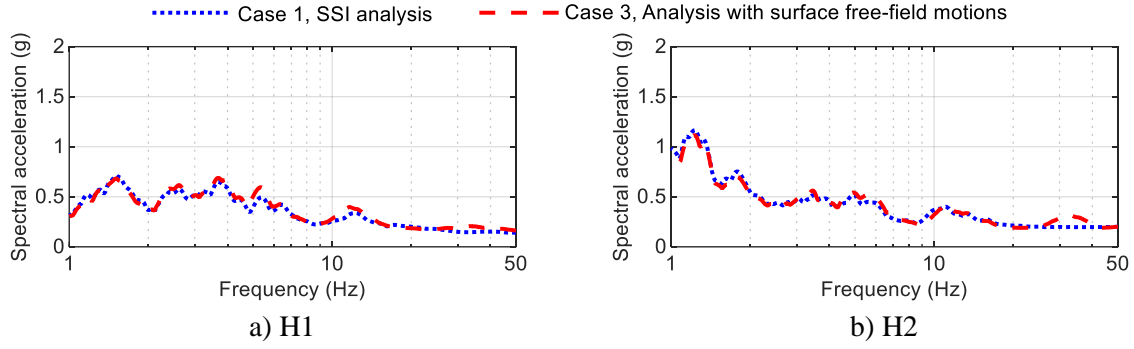


Figure 12. Acceleration response spectra on the basemat of the FHR building, 5% damping, soil profile A, isolation system IS1, ground motion GM1A

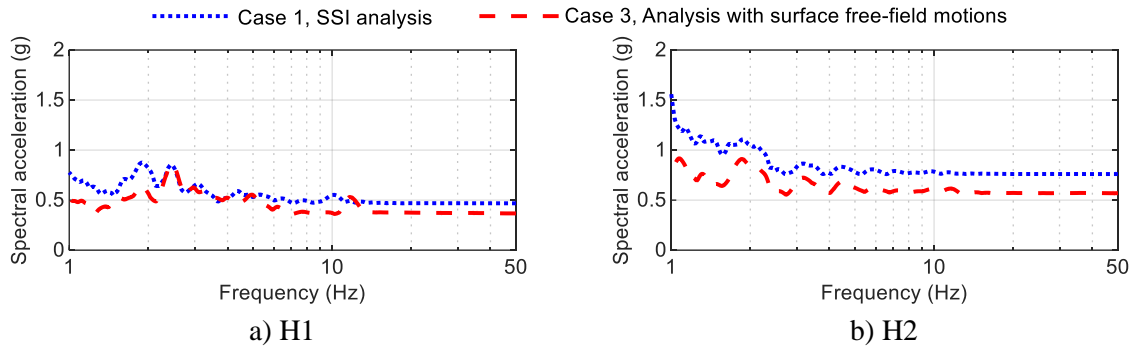


Figure 13. Acceleration response spectra on the basemat of the FHR building, 5% damping, soil profile A, isolation system IS1, ground motion GM1B

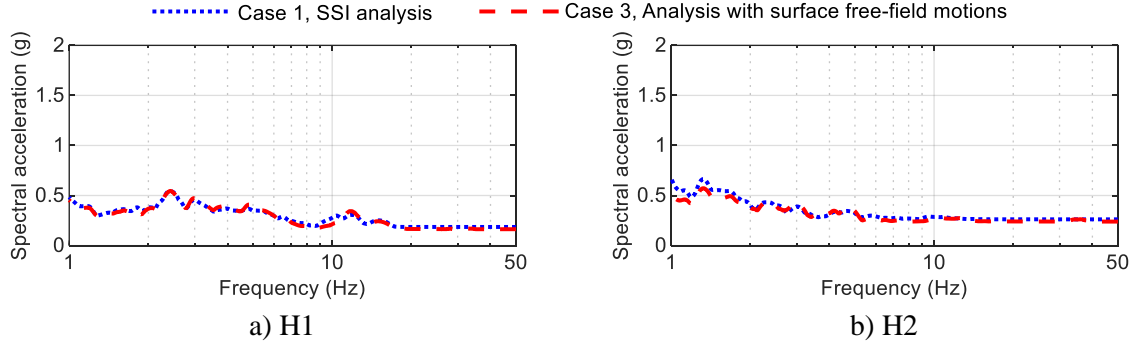


Figure 14. Acceleration response spectra on the basemat of the FHR building, 5% damping, soil profile A, isolation system IS2, ground motion GM1B

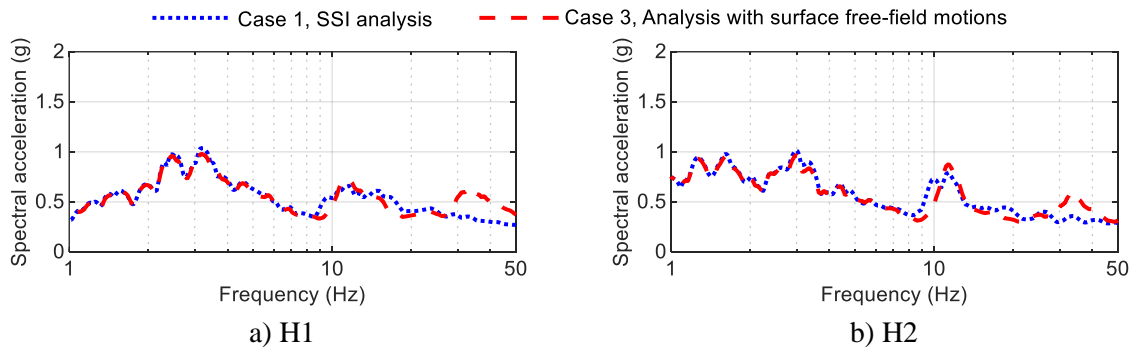


Figure 15. Acceleration response spectra on the basemat of the FHR building, 5% damping, soil profile C, isolation system IS1, ground motion GM1B

CLOSING REMARKS

Soil-structure-interaction analysis is routinely required for new build NPPs: a legacy requirement based on the assumption that LLWRs are both stiff and heavy. Advanced small modular reactors are generally an order of magnitude or more smaller than these LLWRs, and if they are base isolated, will be both flexible and light. Rules written for SSI analysis of stiff, heavy LLWRs likely do not apply to isolated advanced reactors, and that is the focus of the preliminary study reported in this paper. The key conclusion of this study is that the ratio of the fundamental periods of the isolation system to the degraded soil column will determine whether SSI analysis is required. A ratio of 1.75 is an *interim* recommendation based on results presented here. On-going analysis of seven realistic soil columns from the APR1400 DCD, three fundamentally different advanced reactors, eight 2D isolation systems, three ground motion pairs, and two levels of ground shaking support a lower value, approaching 1.25.

ACKNOWLEDGEMENTS

The information, data, or work presented herein was funded by the U.S. Department of Energy under its Advanced Reactor Concept (ARC-20) project, Award Number DE-NE0009049. The views and opinions of the authors expressed herein do not necessarily state or reflect those of the United States Government or any agency thereof, the University at Buffalo, Massachusetts Institute of Technology, Applied Geodynamics, and Simpson, Gumpertz & Heger. The authors thank Professor Youssef Hashash of the University of Illinois at Urbana-Champaign for his support of our use of DEEPSOIL.

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