Recent Advances in Seismic Soil-Structure Interaction Analysis of Nuclear Power Plants

Mansour Tabatabaie, Ph.D., P.E.
SC Solutions, Inc.
801 Ygnacio Valley Rd., Ste. 200
Walnut Creek, CA 94596 U.S.A.
Tel: 408-617-4530
Email: mansour@scsolutions.com

ABSTRACT

Three-dimensional seismic soil-structure interaction (SSI) analysis of nuclear power plants (NPPs) is often performed in the frequency domain using programs such as SASSI. This enables the analyst to properly a) address the effects of wave propagation in an unbounded soil media, b) incorporate strain-compatible soil shear moduli and damping properties, and c) specify input motion in the free field using the de-convolution method and/or spatially variable incoherent ground motions. However, the size of the SSI system that can be analyzed by SASSI has been limited to coarse finite element models of the structures. Furthermore, because the frequency-domain procedure is limited to linear systems, SASSI is not directly applicable to structures exhibiting nonlinearities at the soil/structure interface (such as base sliding and/or uplift and sidewall/back soil separation) or within the structure (such as component isolation). While these problems require using time-domain methods, the available software is limited in its ability to model dynamic SSI effects.

This paper presents three new modeling methodologies implemented in SASSI to solve large-scale SSI problems and address foundation and in-structure nonlinearities. They include large core solution (LCS), component mode synthesis (CMS), and distributed parameter foundation (DPF) models.

The application of the LCS model in SASSI to the SSI analysis of the US EPR™ nuclear island is also discussed in this paper, the results of which are used to address several SSI analysis and design issues, such as stick versus detailed structural modeling, basemat flexibility, floor, wall and roof slab flexibility, backfill soil and cracked concrete modeling, foundation mesh refinement, dynamic soil pressures, and effects of structure-soil-structure interaction (SSSI) and incoherent ground motions.

INTRODUCTION

Three-dimensional seismic soil-structure interaction (SSI) analysis of nuclear power plants (NPPs) in the United States is often performed in the frequency domain using SASSI [1, 2]. This enables the analyst to properly a) address the effects of wave propagation in an unbounded soil media, b) incorporate strain-compatible soil shear moduli and damping properties, and c) specify input motion in the free field using the de-convolution method and/or spatially variable ground motions. For large-scale structural systems with several hundred thousand degrees-of-freedom (DOFs) and large foundation impedance matrices associated with deeply embedded structures, the conventional sub-structuring solution methodology employed in SASSI often results in a coefficient matrix that is too large to solve with currently available computer resources. Furthermore, for structures that exhibit nonlinearities at the soil/structure interface (such as potential base sliding and/or uplift and sidewall/back soil separation) as well as within the structure (such as component isolation), the frequency-domain methodology is not directly applicable as it is limited to linear systems. To address these problems, the analytical method in SASSI has been advanced to include three new models: the large core solution (LCS), the component mode synthesis (CMS), and the distributed parameter foundation (DPF) model. These new models, now incorporated in ^{MTR}/SASSI [3], are briefly discussed in this paper, followed by a description of the SSI analysis of US EPR^{TM} nuclear island (NI) using ^{MTR}/SASSI, the results of which are used to address several key aspects of the SSI analysis of NPPs.

LARGE CORE SOLUTION (LCS) MODEL

Seismic response analysis of NPPs in the United States is often required for frequencies up to 33 Hz [4]. In addition, NPPs founded on hard rock in the Eastern United States are now required to be analyzed to frequencies up to 50 Hz [5]. Because the foundation soil media for typical NPPs and the sidewall backfill for NPPs founded on hard rock generally have relatively low shear wave velocities (V_s = 200 – 500 m/s), the above frequency passing requirements often result in large-scale finite element soil and structural models that are too big to handle using the conventional SASSI modeling methodologies.

Utilizing recent advances in computer software and hardware technology, the SASSI code has been modernized to incorporate a large core solution (LCS) model. This feature now makes it possible to efficiently analyze large-scale, deeply-embedded nuclear plants. SSI models with up to 400,000 degrees-of-freedom (DOFs) have been analyzed in SASSI using LCS model. In addition, because the structural nodes and elements can be numbered arbitrarily, the SASSI model can serve as a duplicate copy of the corresponding detailed FE model of the structure used for structural design. This greatly facilitates model development, translation, calibration, and maintenance. The application of the LCS model to seismic SSI analysis of a detailed NI model is discussed later in this paper.

COMPONENT MODE SYNTHESIS (CMS) MODEL

For large-scale structural models, the component mode synthesis (CMS) model is another alternative method of SSI analysis in SASSI. The primary function of this method is to represent a large, complex structural system as an assemblage of various components represented by their modal properties. These modal properties include fixed-interface natural vibration modes, rigid-body modes, and interface constrained modes, which fully describe the displacement behavior of a component. CMS involves three basic steps: 1) division of the structure into components, 2) definition of sets of component modes, and 3) coupling of the component modes to form a reduced order system model. Detailed formulation of the CMS procedure to solve a large-scale finite element SSI model in SASSI is presented in Ref. [6]. Figure 1 shows a flow diagram of CMS-based SSI analysis by SASSI.

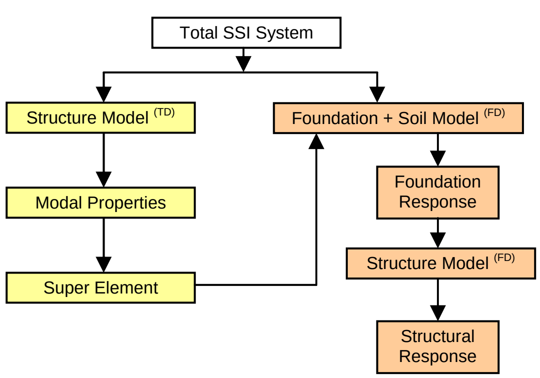

Figure 1: Flow Diagram of CMS-Based SSI Analysis in ^{MTR}/SASSI

To validate the CMS model, the dynamic response of a simple-lumped parameter model with mass eccentricity supported on a square mat foundation on uniform halfspace and subjected to vertical and horizontal excitation is obtained using the CMS procedure in MTR/SASSI and the results are compared against those of baseline solution also calculated by SASSI. This model is shown in Figure 2. Further details of the model are presented in Ref. [6].

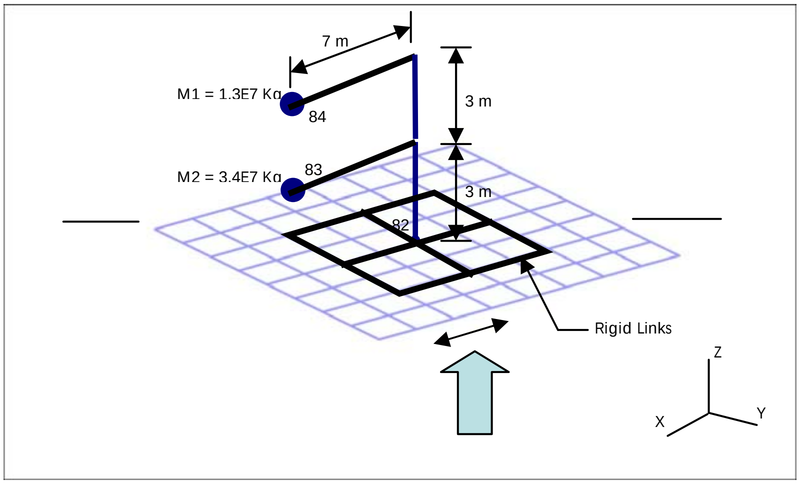

Figure 2: Example SSI Model

The total SSI system was first analyzed in the frequency domain using SASSI to obtain the baseline solution. The same model was then analyzed using the CMS procedure implemented in SASSI, as shown in Figure 3.

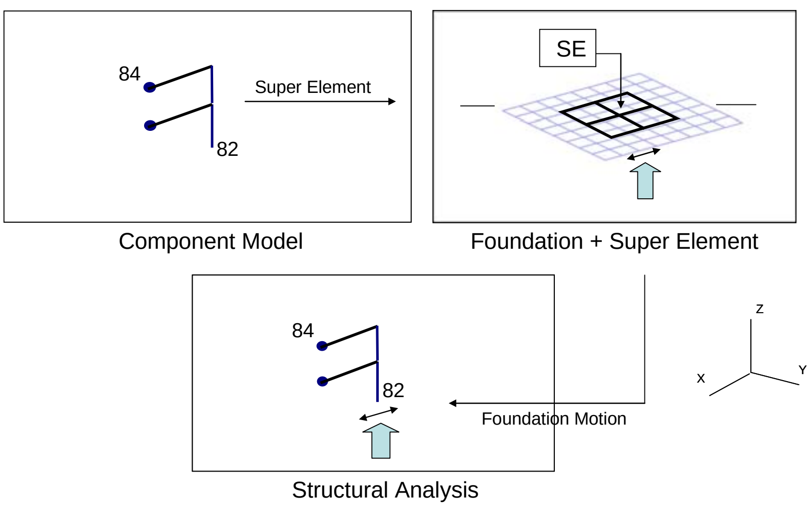

Figure 3: CMS-Based SSI Analysis Steps

Two cases corresponding to rigid and flexible basemats were analyzed. The computed x and y transfer functions at Node 84 (upper mass node), due to the x-input obtained with the CMS procedure, are compared with those of the baseline solution in Figure 4. Similar comparisons for the x and z transfer functions due to the z-input are shown in Figure 5. Both figures show excellent agreement between the two results. This methodology is currently being developed further.

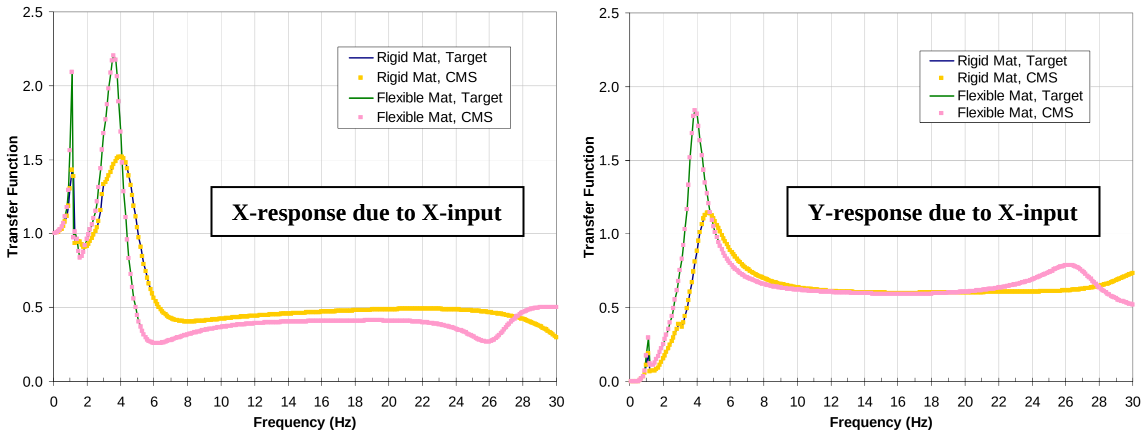

Figure 4: Comparison of Transfer Functions at Node 84 due to X-Input

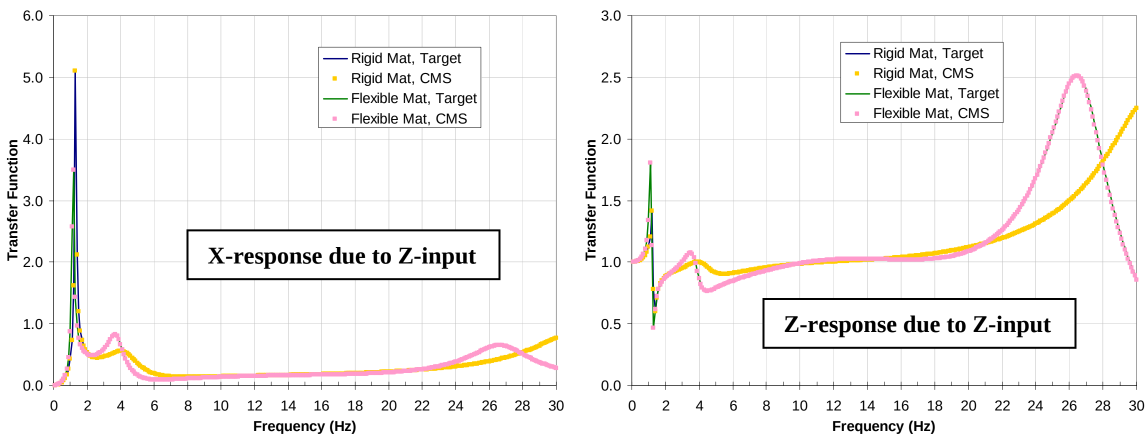

Figure 5: Comparison of Transfer Functions at Node 84 due to Z-Input

DISTRIBUTED PARAMETER FOUNDATION (DPF) MODEL

Because the frequency-domain procedure is limited to linear systems, SASSI is not directly applicable to structures exhibiting nonlinearities at the base (such as base sliding and/or uplift) or within the structure (such as component isolation). The author has introduced a hybrid frequency/time domain procedure called the distributed parameter foundation (DPF) model that allows the structure to be partitioned from the total SSI system and analyzed in the time domain while the foundation media is modeled using the frequency-domain procedures [7]. The DPF method involves four steps: 1) calculating the foundation dynamic impedance at each foundation interaction node from soil reaction forces and interaction displacements in the frequency domain using SASSI, 2) developing equivalent simple-damped oscillators with constant parameters (spring, mass and dashpot) representing the frequency-dependent dynamic impedance functions obtained in Step 1, 3) calculating the foundation scattering motions at the same interaction nodes from SASSI, and 4) implementing the results as boundary conditions in the time-domain dynamic response analysis of the structure. To model base sliding and/or uplift, inelastic springs are provided between the linear foundation KMC and the structure at each interaction node [8].

An example of the application of the DPF method to seismic SSI analysis of NPPs is shown in Figure 6. The model represents a typical PWR containment structure consisting of a primary concrete containment, a secondary steel containment, and concrete internal structures housing the reactor. The cylindrical reactor cavity is deeply embedded below the ground surface.

This model is an idealization of a typical pressurized water reactor and does not represent an actual or existing structure. For simplification, the steel containment and parts of the internal structure such as the steam generator and pressurizer compartments are not included. The basemat and exterior walls below grade are assumed rigid; the floor slabs are simplified and included mainly for their effect on the structural response. Further model details and the SSI analysis are presented in Ref. [9].

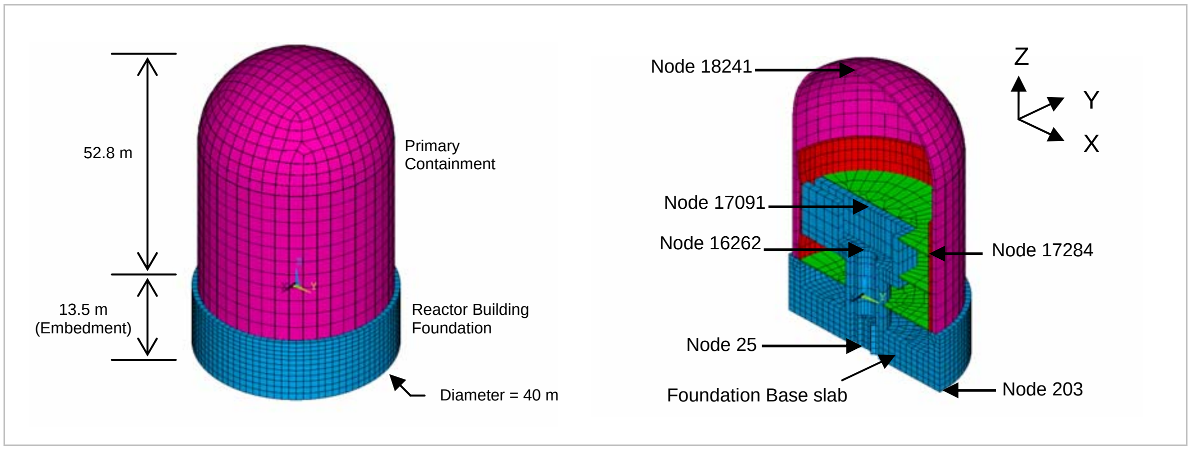

Figure 6: SASSI/ANSYS Finite Element Structural Models

Figure 7 shows a comparison of the typical SSI results in terms of the computed response spectra at several key locations in the reactor building due to x-input obtained from the frequency- and time-domain solutions using SASSI and ANSYS [10], respectively.

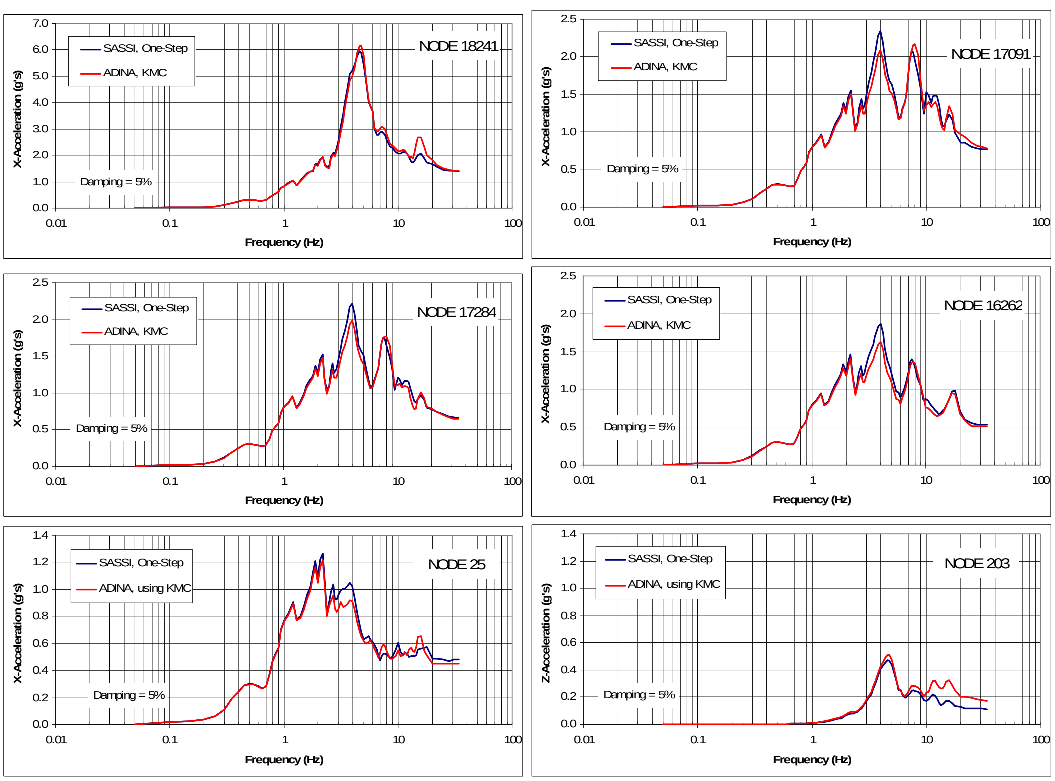

Figure 7: Comparison of SSI In-Structure Response Spectra due to Input Motion in X-Dir

SSI ANALYSIS OF US EPR™ NUCLEAR ISLAND (NI)

The following is description of the application of the LCS model in MTR/SASSI to a large-scale NPP model. The layout of the US EPR™ plant is shown in Figure 8. The plant consists primarily of a nuclear island (NI) and several other significant structures outside of and in close proximity to the NI. The NI structures consist of the Reactor Building Containment (RBC), Reactor Building Shield (RBS), Reactor Building Internal Structures (RBI), Fuel Building (FB), Safeguard Building 1 (SB1), Safeguard Building 2/3 (SB2/3), and Safeguard Building 4 (SB4) - all of which share a common foundation basemat. The NI is embedded approximately 11.6 meters below ground surface.

Figure 8: Layout of EPR™ Plant

The plant is analyzed for 8 generic soil profiles as part of the standard design certification. The generic soil profiles used for the SSI analysis are shown in Figure 9. The input motions consist of three-component, spectra–compatible EUR Soft, EUR Medium and EUR Hard motions specified as free-field outcrop motions at the base of the NI basemat. The response spectra of the input motions are shown in Figure 10.

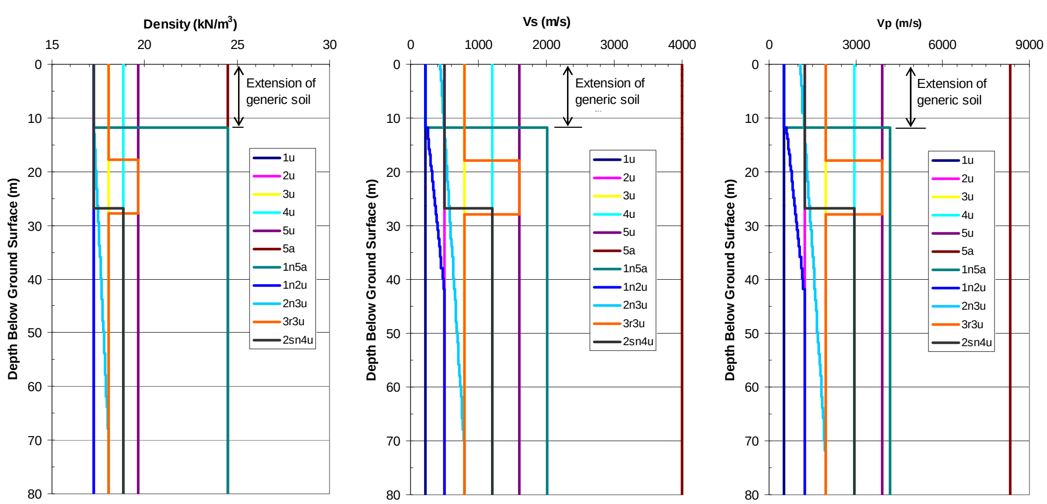

Figure 9: Generic Soil Profiles and Properties

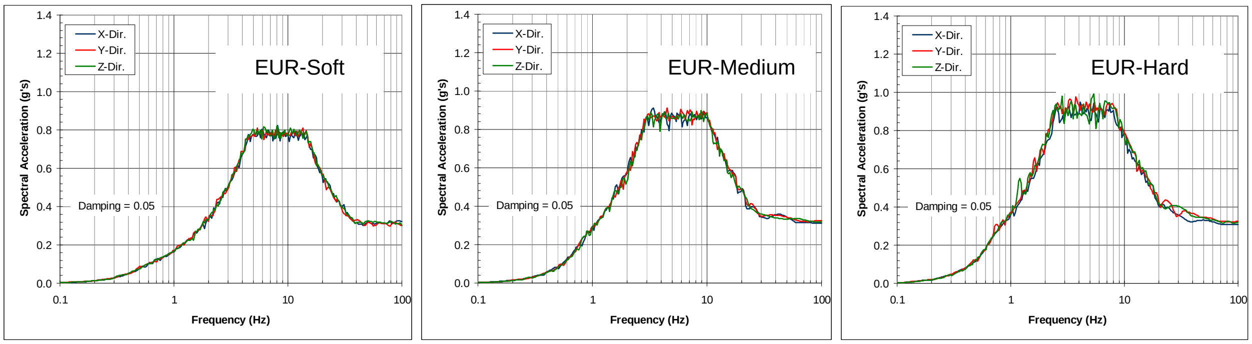

Figure 10: Acceleration Response Spectra of Reference Outcrop Motions

Two structural models are used for the SSI analysis: a stick model and a detailed finite element model, as shown in Figure 11 and Figure 12, respectively.

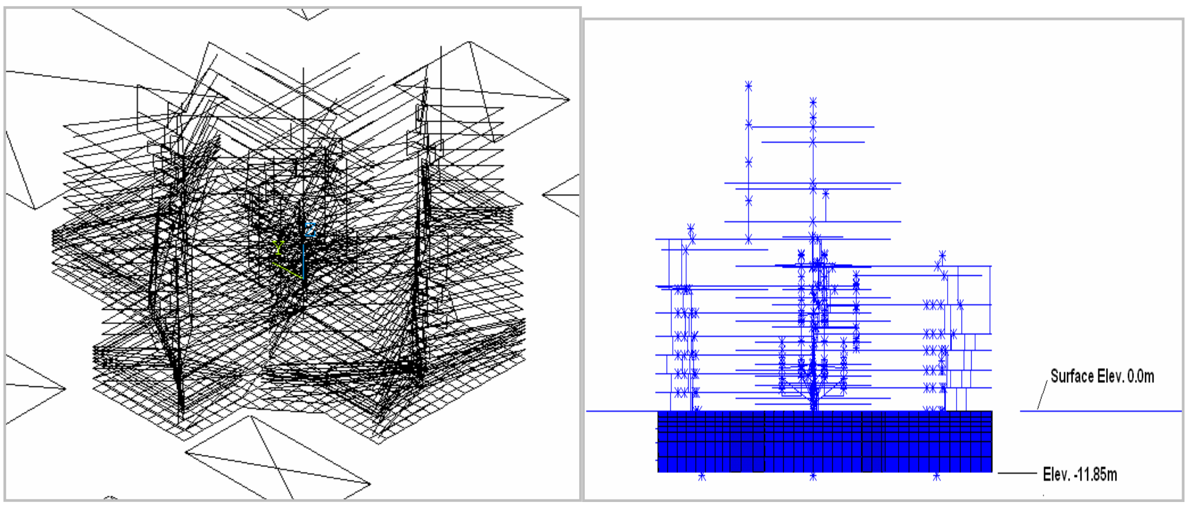

Figure 11: SASSI Stick Model of NI Structures

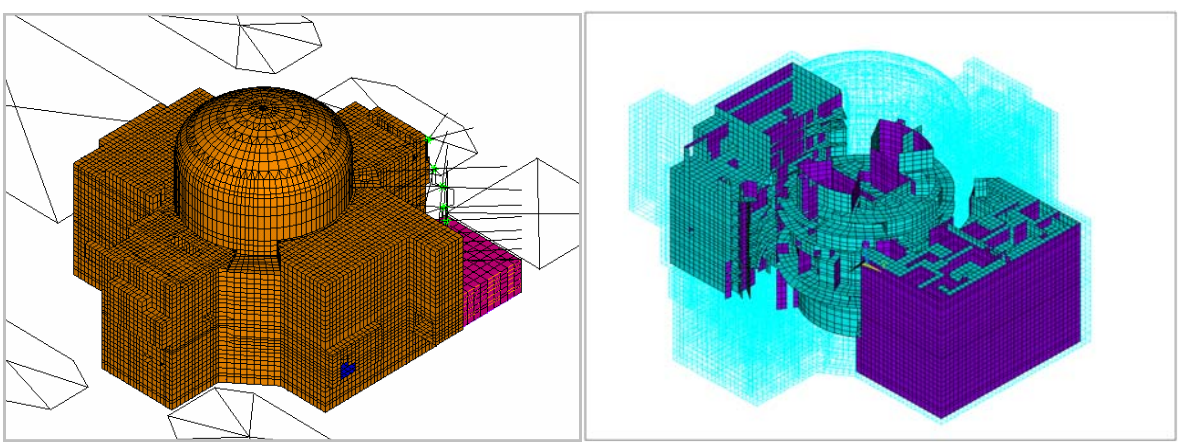

Figure 12: SASSI Detailed FE Model of NI Structures

The stick model consists of multiple interconnected sticks representing the walls and columns between the principal floor elevations of the structures. To model embedment effects, horizontal rigid beams are added along the excavation face at soil layer interfaces where the NI walls bear against soil (these beams share common nodes with the soil interaction nodes). The beams are then connected to the FB shield stick, the SB2/3 shield stick, and the SB1 and SB4 sticks with rigid links to provide lateral support from side soils and to transfer forces from the side soils to the sticks. The detailed FE model incorporates all the major details of the NI structures. The FE model consists mainly of shell elements representing the concrete floors, walls, and basemat - all of which are modeled as flexible members. The NSSS, major equipment supports, and polar crane are modeled by beam elements. The fixed-base modes of the stick model have been aligned against the global modes of the detailed FE model.

The FE model of NI foundation is shown in Figure 13. This model is the same for both the stick and detailed FE models, except that in the stick model the basemat is rigid. All the basement walls are connected to side soils, with the exception of the walls adjacent to the Nuclear Auxiliary Building (NAB) and Access Building (AB).

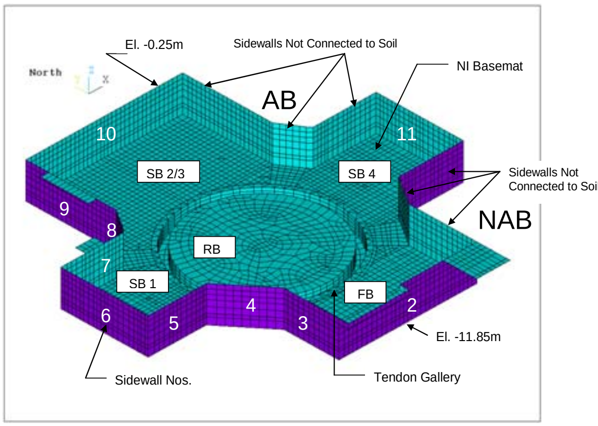

Figure 13: NI Foundation Model

Using a combination of the soil profiles and input motions, a total of 12 SSI analysis cases of the surface-supported NI model and 13 SSI analysis cases of the embedded NI model were evaluated. Each analysis case consisted of three separate SASSI runs with three components of the input motion applied separately in the x-, y- and z-directions. The results of the three analyses (i.e. responses due to x-input, y-input and z-input) in terms of acceleration time history responses at any output node were then algebraically summed, and the results were used to calculate the maximum accelerations and acceleration response spectra. The results of the SSI analyses of the US EPR^{TM} NI structures are used to address several aspects of the SSI modeling and analysis of NPP structures.

Stick Versus Detailed FE Model

The SASSI program was used to calculate the SSI response of the US EPR^{TM} NI using both stick and detailed FE models [11]. The analysis was performed for Soil Case 2sn4u and EUR Medium motion. The maximum average element size in the soil model is about 2.3 meters, which corresponds to a passing frequency of 500/5/2.3 = 44 Hz. Because the input motion does not contain significant energy beyond 40 Hz, the frequency cutoff for the SSI model was set at 50 Hz. The analysis was performed for 42 and 66 computed frequencies of the stick and detailed finite element models, respectively, with the intermediate frequency response values of the transfer functions obtained by interpolation. The computed transfer functions at key structural locations were plotted and visually examined to ensure that adequate frequency responses were computed for later interpolation of transfer functions.

Typical results of the SSI analysis of the detailed FE model of NI in terms of the computed maximum accelerations in the x-, y- and z-directions are shown in Figure 14, Figure 15 and Figure 16.

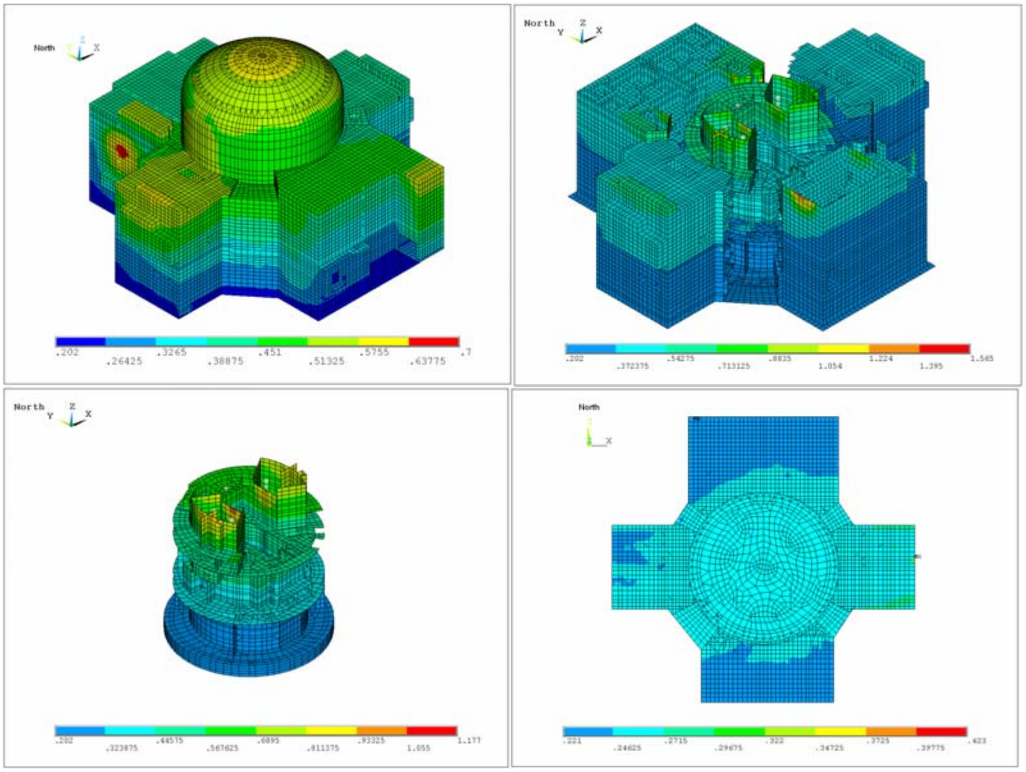

Figure 14: Maximum Acceleration Contours in X-Dir.

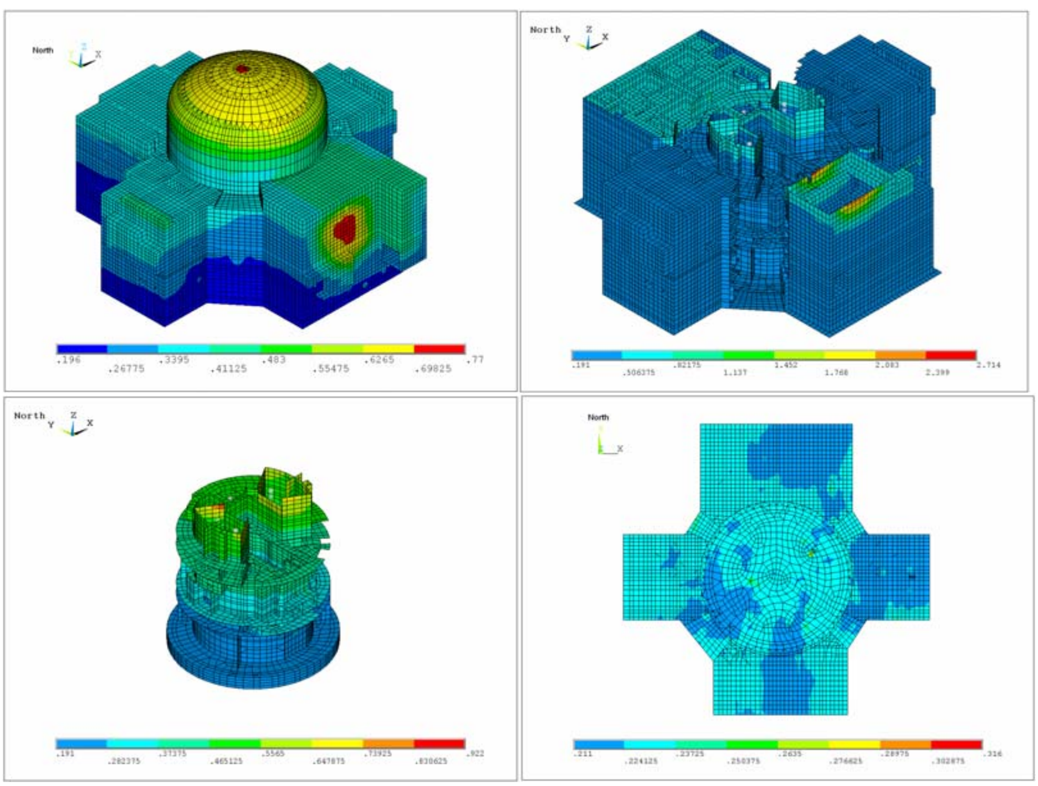

Figure 15: Maximum Acceleration Contours in Y-Dir.

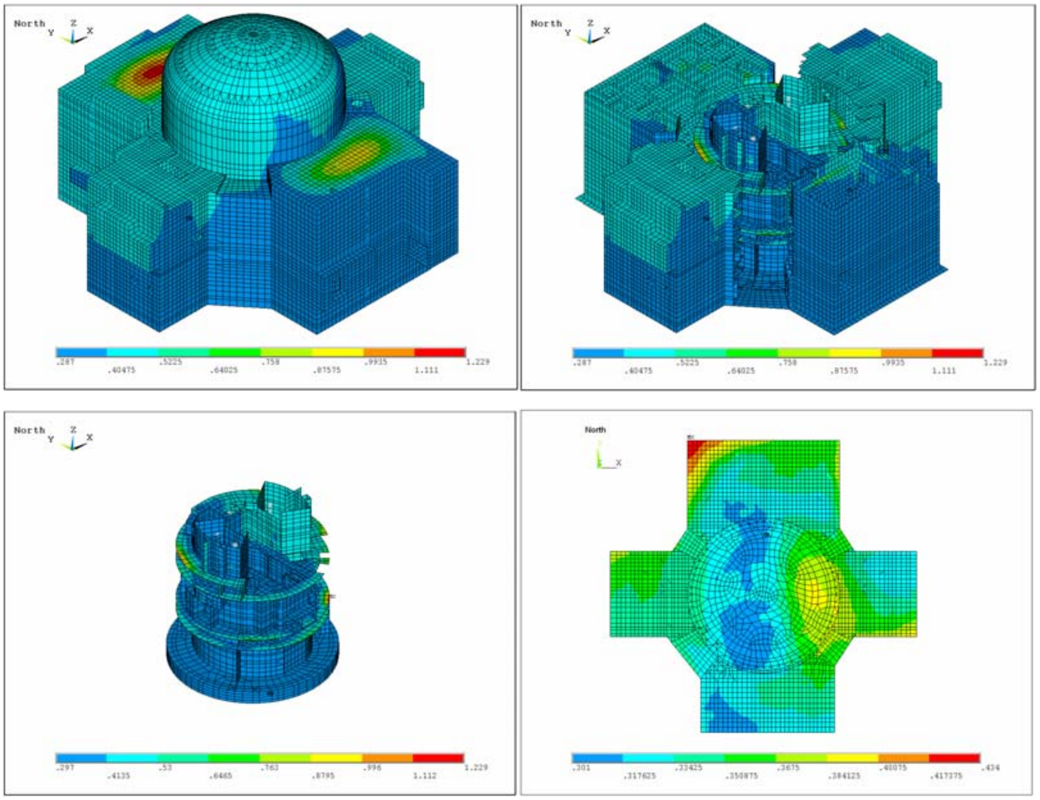

Figure 16: Maximum Acceleration Contours in Z-Dir.

The maximum accelerations at several key locations in the major floor elevations of the NI structures are calculated from the stick and detailed FE model analyses and listed in Table 1 and Table 2, respectively. Although the results could not be compared at exactly the same locations, in general they indicate similar results from the stick and detailed FE models. The typical acceleration response spectra calculated at the top of the reactor containment building from the stick and detailed FE models are compared in Figure 17. A typical global response in terms of the interstory forces and moments is shown in Figure 18.

Table 1: Maximum Accelerations of Stick Model (g's)

Location	Elev. (m)	X	Y	Z
Center of NI Basemat	-11.85	0.277	0.210	0.318
Reactor Building IS	+5.15	0.347	0.258	0.341
Reactor Building IS	19.50	0.421	0.391	0.366
Safeguard Building 1	29.30	0.564	0.502	0.501
Safeguard Building 2/3	12.00	0.411	0.409	0.446
Safeguard Building 4	29.30	0.580	0.621	0.556
Fuel Building	3.70	0.350	0.294	0.357
Reactor Containment Bldg	58.00	0.738	0.620	0.893
Reactor Shield Bldg	61.40	0.843	0.854	0.578

Table 2: Maximum Accelerations of Detailed FE Model (g's)

Location	Elev. (m)	X	Y	Z
Center of NI Basemat	-11.85	0.262	0.224	0.319
Reactor Building IS	+5.15	0.379	0.320	0.374
Reactor Building IS	+19.50	0.513	0.419	0.388
Safeguard Building 1	+21.00	0.478	0.356	0.399
Safeguard Building 2/3	+16.30	0.400	0.433	0.413
Safeguard Building 4	+21.00	0.340	0.335	0.394
Fuel Building	+4.20	0.300	0.364	0.298
Reactor Containment Bldg	58.00	0.869	0.734	0.516
Reactor Shield Bldg	61.40	0.598	0.679	0.490

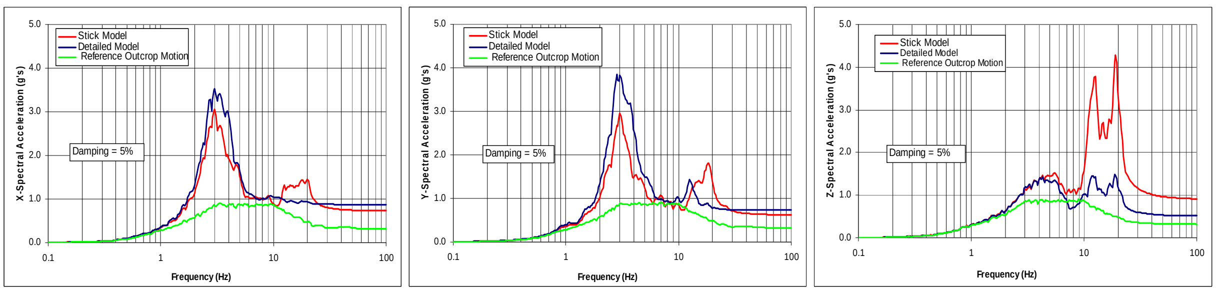

Figure 17: Comparison of Response Spectra at Top of Reactor Containment Bldg (Elev. 58.0 m)

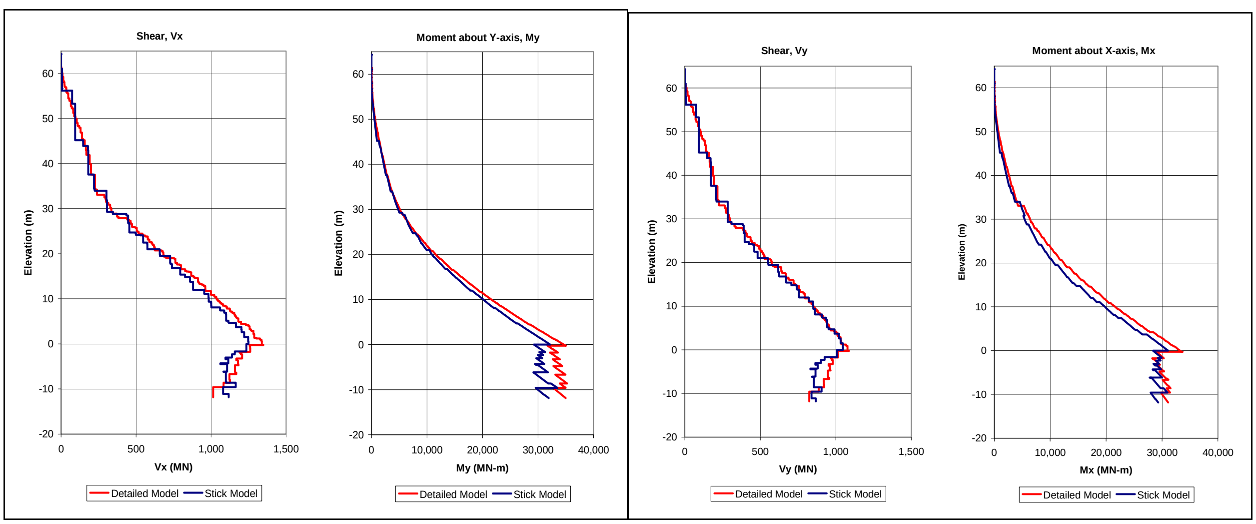

Figure 18: Maximum Absolute Total Interstory Forces and Moments

Based on the results of this study, the following conclusions can be drawn:

- Stick models are capable of determining global seismic responses, but they can lead to excessively conservative results in the vertical direction due to the limited number of modes that can be modeled.
- The detailed FE models capture local responses, thus eliminating the need for modeling single DOF oscillators.
- Effects of the basemat flexibilities can be considered in the detailed FE models.
- Meshing can be made sufficiently small in detailed FE models to capture the response due to high frequency input motions.

Effects of Basemat Flexibility

The results of the stick versus detailed FE model discussed above indicate the importance of the basemat flexibility in calculating the spectral response of NI structures at higher frequencies (see Figure 17). In addition, the basemat flexibility will most likely affect the local uplift behavior of large basemats.

Effects of Foundation Embedment and Side Soil De-Bonding

To assess the effects of the foundation embedment and de-bonding (or separation) of the side soils from the exterior walls below grade, sensitivity studies were performed using the stick model of US EPR^{TM} NI. The analysis was performed for Soil Case 2sn4u and EUR Medium motion, in which four NI foundation cases were analyzed: fixed-base, surface-supported, embedded with fully bonded side soil, and embedded with partially bonded side soil. For the case of surface-supported NI, the 11.85 meter-thick side soil layer extending from the ground surface to the bottom of the NI basemat was ignored. For the case of partially bonded side soil, the lateral soil confinement in the upper 1.5 meters below ground surface was removed. This was achieved by introducing double nodes at the wall/soil interface, with one node attached to the wall and the other to the soil but with no connectivity (force transfer) between the two nodes. Further details may be found in Ref. [12].

Table 3 summarizes the typical reductions in the computed maximum accelerations at key structure locations due to embedment effects. Figure 19 shows the typical results of response spectra calculated in the reactor building internal structures (RBIS) at Elev. 13.8 meters.

Table 3: Comparison of Maximum Accelerations for Embedded vs. Surface-Supported NI

Location		% Reduction in Maximum Accelerations due to Embedment					
		Fully Bonded Side Soil			Partially De-Bonded Side Soil		
Bldg	Elev. (m)	X-Dir	Y-Dir	Z-Dir	X-Dir	Y-Dir	Z-Dir
Center of NI basemat	-11.85	10.2	39.1	21.2	12.2	39.6	21.3
Reactor Building IS	1.50	12.0	40.9	19.2	12.6	40.9	19.0
Reactor Building IS	5.15	10.2	36.2	18.7	11.4	37.5	19.0
Reactor Building IS	9.40	11.5	30.4	18.3	12.7	32.2	18.5
Reactor Building IS	13.80	14.7	23.1	18.2	15.7	24.9	18.3
Reactor Building IS	19.50	18.0	15.5	18.5	19.0	17.5	18.5
MS Valve Room	22.50	19.8	24.5	17.8	19.9	23.9	18.2
MS Valve Room	21.00	17.9	28.6	25.7	18.4	30.5	25.5
Safeguard Building 1	29.30	20.8	29.3	25.4	21.4	31.6	25.2
Safeguard Building 2/3	12.00	18.8	27.7	23.1	20.2	28.9	23.2
Safeguard Building 4	29.30	22.7	12.9	0.0	22.4	14.5	0.10
Fuel Building	3.70	26.8	27.9	25.9	26.3	29.4	25.6

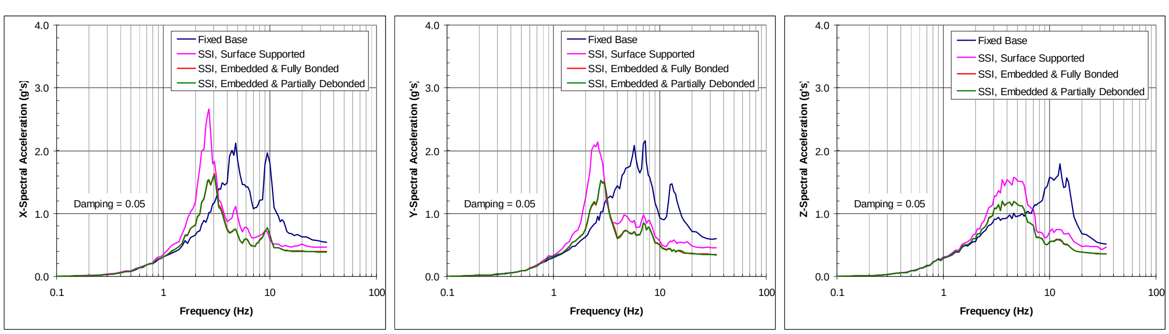

Figure 19: Comparison of In-Structure Response Spectra for RBIS at Elev. 13.8 m

Based on a comprehensive review of the results, the following conclusions can be drawn:

- These results, in general, indicate significant reduction in global maximum accelerations as well as spectral acceleration response at structural frequencies occurring above 3 Hz, with increase in the spectral acceleration response as a result of rigid-body rocking due to SSI at frequencies below 3 Hz. This is consistent with the behavior of stiff and massive NPP structures analyzed on flexible foundation soil media as compared to infinitely rigid base rock (fixed-base).
- Comparison of the surface-supported versus embedded SSI results indicates 10-30%, 10-40% and 15-25% reductions in maximum acceleration responses in the x-, y- and z-directions, respectively, due to embedment effects. Similarly, significant reduction in the overall shape and peak of the spectral acceleration response is also observed due to embedment effects. For example, the peak spectral acceleration response is reduced by 30-40%, 25-30% and 20-30% for the responses in the x-, y- and z-directions due to consideration of foundation embedment.
- Comparison of the embedded EPR results assuming fully bonded versus 1.5-meter de-bonded side soil indicates no significant differences in the results due to partial de-bonding effects. Therefore, the gains achieved by considering the embedment effects remain unchanged due to partial de-bonding of NI walls from side soils near ground surface.
- Analysis of maximum dynamic soil pressures occurring along the centerline of the FB shield wall, SB1 wall, SB2/3 shield wall and SB4 shield wall indicates a possible concentration of high soil strains adjacent to NI walls within depths of 6 meters below ground surface. These strains, in general, can cause some reduction in lateral soil stiffness and redistribution of dynamic earth pressures due to secondary effects. Because it is difficult to capture these secondary effects in a linear analysis, the effect of a possible concentration of high soil strains near the ground surface may be accounted for by allowing the de-bonding of side soil from basement walls.

EFFECTS OF FOUNDATION MESH REFINEMENT

To assess the effects of foundation mesh refinement, two sets of SSI analyses of the US EPR™ NI structures were performed using the stick model. The first set considered the surface-supported EPR™ model with coarse and refined foundation mesh (Figure 20); the second set consisted of the embedded EPR™ model with coarse and refined foundation mesh (Figure 21). The analysis was performed for Soil Case 1n2u and EUR Soft motion. Based on the minimum shear wave velocities and average excavated soil element size, the passing frequencies for the coarse and refined foundation models are approximately 10.4 Hz and 18.4 Hz, respectively.

Comparisons of maximum accelerations calculated at several key locations in the NI structures for the coarse and refined foundation mesh are shown in Table 4 for the surface-supported model and in Table 5 for the embedded NI model. Comparisons of typical response spectra calculated in the Fuel Building for the surface-supported and embedded NI models are shown in Figure 22 and Figure 23, respectively.

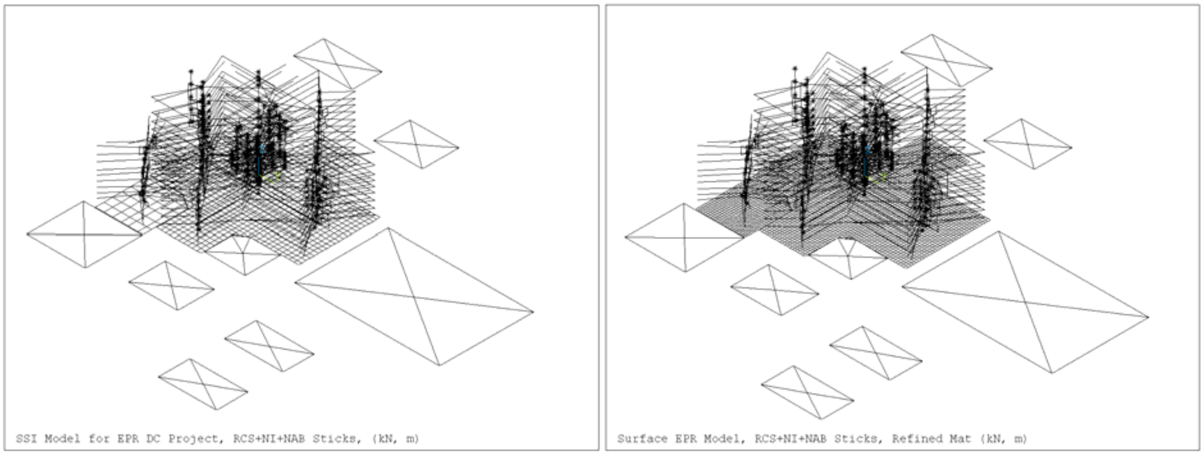

Figure 20: Surface-Supported Stick NI Model with Coarse and Refined Foundation Mesh

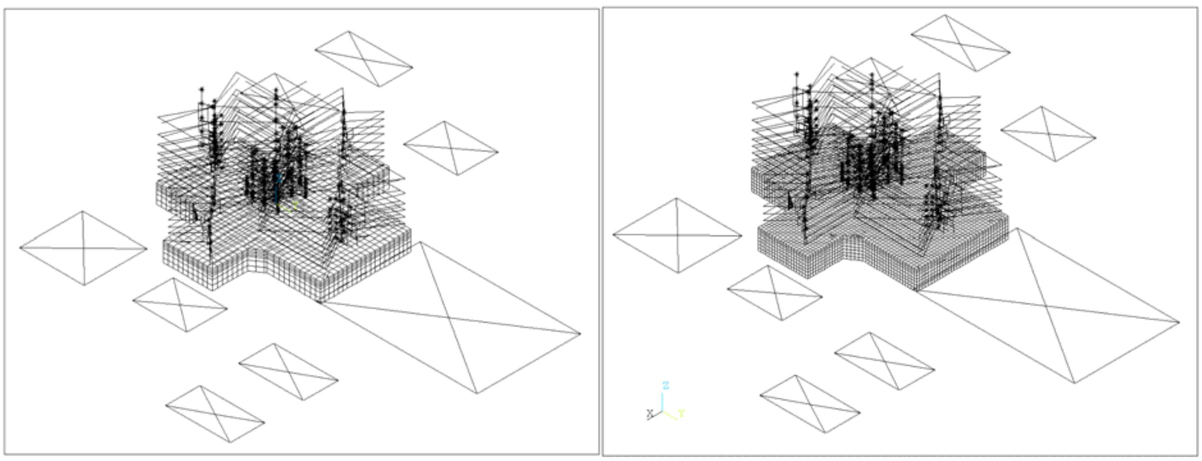

Figure 21: Embedded NI Stick Model with Coarse and Refined Foundation Mesh

Table 4: Comparison of Maximum Accelerations, Coarse vs. Refined Surface-Supported NI

Location		Maximum Accelerations (g's)						Ratio (2)/(1)		
		Coarse Model (1)			Refined Model (2)					
Bldg.	Elev. (m)	X	Y	Z	X	Y	Z	X	Y	Z
Center of NI Basemat	-11.85	0.22	0.24	0.28	0.22	0.24	0.28	0.99	0.98	0.99
Reactor Building IS	1.50	0.23	0.23	0.28	0.22	0.23	0.28	0.99	1.00	0.99
Reactor Building IS	5.15	0.24	0.23	0.29	0.23	0.23	0.28	0.98	1.00	0.98
Reactor Building IS	9.40	0.25	0.25	0.29	0.23	0.25	0.29	0.95	1.01	0.98
Reactor Building IS	13.80	0.25	0.27	0.29	0.24	0.28	0.29	0.97	1.01	0.98
Reactor Building IS	19.50	0.26	0.30	0.30	0.26	0.30	0.29	1.00	1.00	0.98
MS Valve Room	22.50	0.29	0.29	0.30	0.29	0.29	0.31	1.00	0.98	1.01
MS Valve Room	21.00	0.29	0.32	0.39	0.29	0.32	0.40	0.99	0.99	1.02
Safeguard Building 1	29.30	0.32	0.36	0.40	0.32	0.36	0.40	1.01	1.00	1.02
Safeguard Building 2/3	12.00	0.28	0.31	0.37	0.30	0.31	0.37	1.09	1.00	1.01
Safeguard Building 4	29.30	0.32	0.35	0.40	0.33	0.34	0.41	1.01	0.95	1.01
Fuel Building	3.70	0.23	0.26	0.44	0.24	0.26	0.44	1.04	1.00	1.00

Table 5: Comparison of Maximum Accelerations, Coarse vs. Refined Embedded NI

Location		Maximum Accelerations (g's)						Ratio (2)/(1)		
		Coarse Model (1)			Refined Model (2)					
Bldg.	Elev. (m)	X	Y	Z	X	Y	Z	X	Y	Z
Center of NI Basemat	-11.85	0.17	0.18	0.24	0.16	0.18	0.24	0.96	0.96	1.02
Reactor Building IS	1.50	0.16	0.18	0.25	0.16	0.17	0.24	0.97	0.95	0.98
Reactor Building IS	5.15	0.16	0.18	0.25	0.16	0.18	0.25	0.99	0.97	0.99
Reactor Building IS	9.40	0.17	0.19	0.25	0.17	0.19	0.25	1.00	1.00	1.01
Reactor Building IS	13.80	0.18	0.21	0.26	0.17	0.21	0.26	0.97	1.01	1.02
Reactor Building IS	19.50	0.18	0.24	0.26	0.18	0.24	0.27	1.00	0.99	1.03
MS Valve Room	22.50	0.21	0.22	0.26	0.21	0.22	0.26	0.96	1.00	0.99
MS Valve Room	21.00	0.21	0.24	0.33	0.20	0.24	0.33	0.97	0.98	1.01
Safeguard Building 1	29.30	0.23	0.29	0.34	0.22	0.26	0.33	0.98	0.92	1.00
Safeguard Building 2/3	12.00	0.20	0.24	0.30	0.19	0.24	0.31	0.96	1.00	1.00
Safeguard Building 4	29.30	0.23	0.30	0.32	0.23	0.28	0.31	1.01	0.94	0.97
Fuel Building	3.70	0.19	0.20	0.37	0.18	0.20	0.35	0.96	0.98	0.95

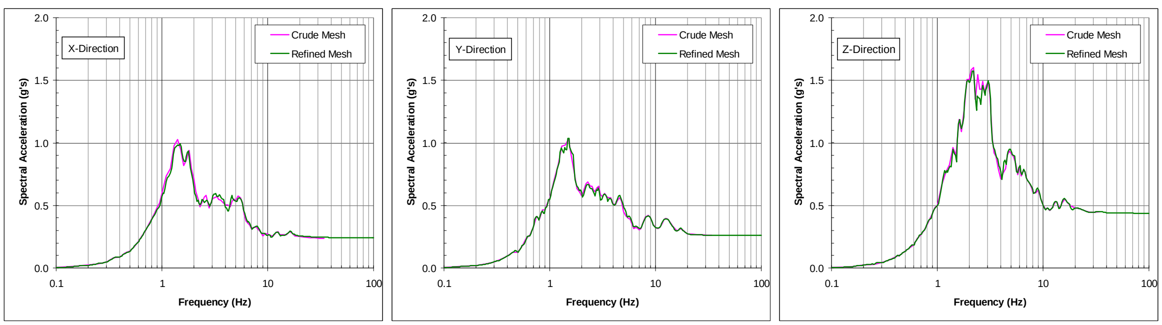

Figure 22: Comparison of Response Spectra of Fuel Bldg, Coarse vs. Refined Surface-Supported NI

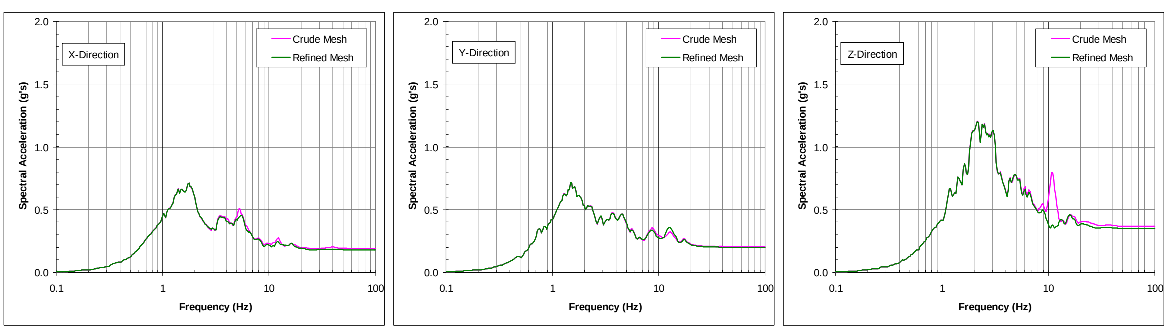

Figure 23: Comparison of Response Spectra of Fuel Bldg, Coarse vs. Refined Embedded NI

Comparisons of the analysis results for the surface-supported NI model with coarse and refined foundation mesh show no significant differences in the computed spectral responses for frequencies up to 25 Hz in the x-, y- and z-directions. In addition, the difference in the computed maximum acceleration response is found to be less than 5% (see Table 4). For the soft soil profiles with minimum V_s = 250 m/s and Soft EUR motion, therefore, the use of the coarse foundation mesh for the surface-supported EPR™ SASSI analyses is appropriate.

Similarly, close agreement between the results of the computed spectral acceleration responses in the structure are observed for the embedded EPR™ model with coarse and refined foundation mesh, except for a few locations in the NI where significant increase in the spectral acceleration response is observed at frequencies above 10 Hz. This is consistent with the passing frequency calculated for the coarse foundation model of 10.4 Hz. The maximum difference in the computed ZPA, however, is less than 5% at all key locations in the structure (see Table 5).

Effects of Floor, Wall and Roof Slab Flexibility

To assess the floor, wall, and roof slab flexibility, the SSI response of the EPR™ NI structures was evaluated using stick and detailed FE models. The results and findings of this study in relation to the effects of floor, wall, and roof slab flexibility were discussed previously.

Effects of Soil Stiffness

To assess the effects of foundation and side soil stiffness, dynamic SSI responses of the US EPR™ NI structures were evaluated for the 13 soil analysis cases discussed previously. The summary of the results for 6 uniform soil cases (1u through 5u and 5a) and 3 input motions EUR (Soft, Medium and Stiff) are presented below. Figure 24 shows comparisons of the acceleration response spectra calculated in the Fuel Building at Elev. 3.7 meters for several combinations of soil stiffness properties and input motions.

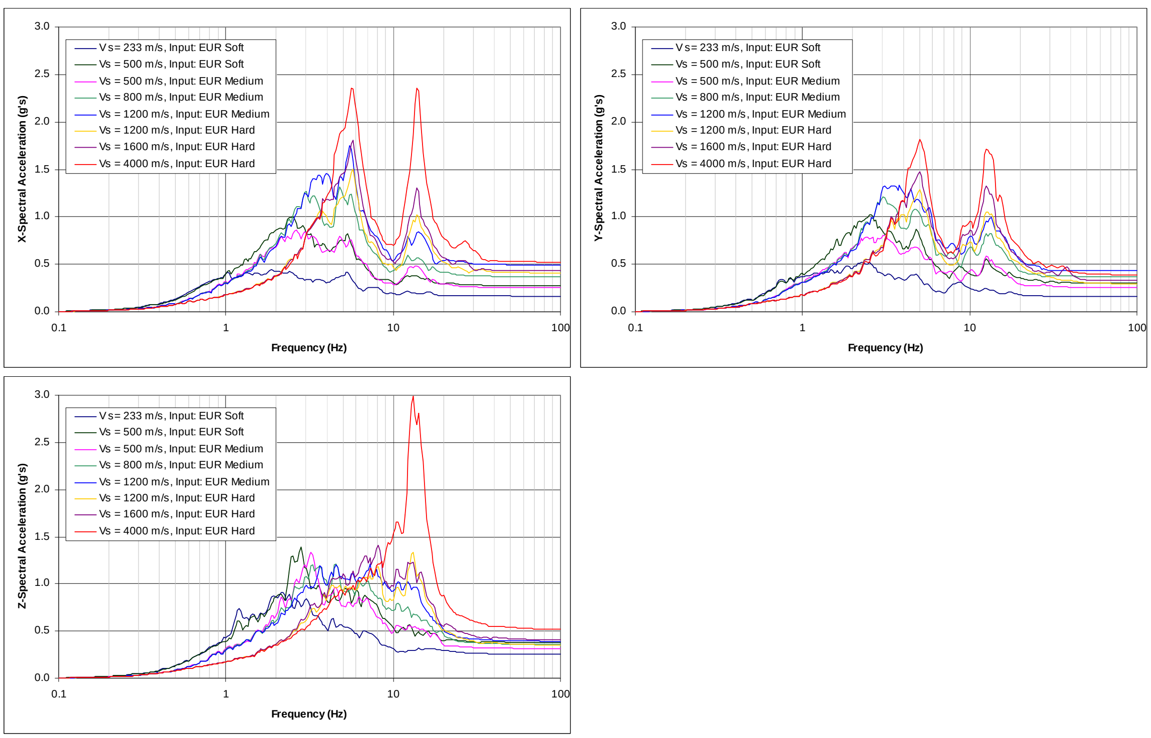

Figure 24: Comparison of Response Spectra at Top of Reactor Containment Bldg (Elev. 58.0 m)

Based on the above results, the following conclusions can be drawn:

- Reducing the foundation and side soil stiffness causes the spectral peaks corresponding to the natural frequencies of the structure to shift slightly to lower frequencies and drop significantly in amplitude. On the other hand, softening the foundation and side soils introduces a rigid-body rocking component in the response that increases the horizontal amplitude of the spectral response at low frequencies.

- The spectral responses at higher frequencies (about 10-12 Hz) are generally controlled by the foundation and side soils which have stiffer properties. Because NPPs are large, stiff, and massive structures founded in relatively softer foundation soil media, this aspect of the SSI response is significant.

Effects of Concrete Cracking

The effects of concrete cracking are often modeled by reducing the flexural stiffness of the beam and shell elements by 50%. In stick models, this can result in excessively conservative results and, in some cases, incorrect results because the stiffness reduction is taken across the board in the structural members. Significantly improved results can be obtained by applying this procedure to a detailed FE model of the NI structures modeled using shell elements. To assess the effect of concrete cracking, the seismic response of the EPR^{TM} NI structures with and without concrete cracking were calculated for all generic soil cases using the ^{MTR}/SASSI program. Figure 25 shows typical results for acceleration response spectra obtained in Safeguard Building 4 (SB4) at Elev. +21 meters for Soil Case 1n5a and EUR Hard motion. This soil analysis case consists of NI founded on hard rock (V_s = 4000 m/s) at a depth of 11.85 meters below ground surface and surrounded by backfill material (V_s = 233 m/s). Table 6 shows the ratio of cracked to uncracked maximum acceleration response at several key locations in the structure.

Based on the above results, the following conclusions can be drawn:

- Concrete cracking can increase the maximum acceleration response in the structure.
- Concrete cracking shifts the spectral peaks to lower frequencies and increases the out-of-plane spectral amplitudes of floor, wall and roof slabs.

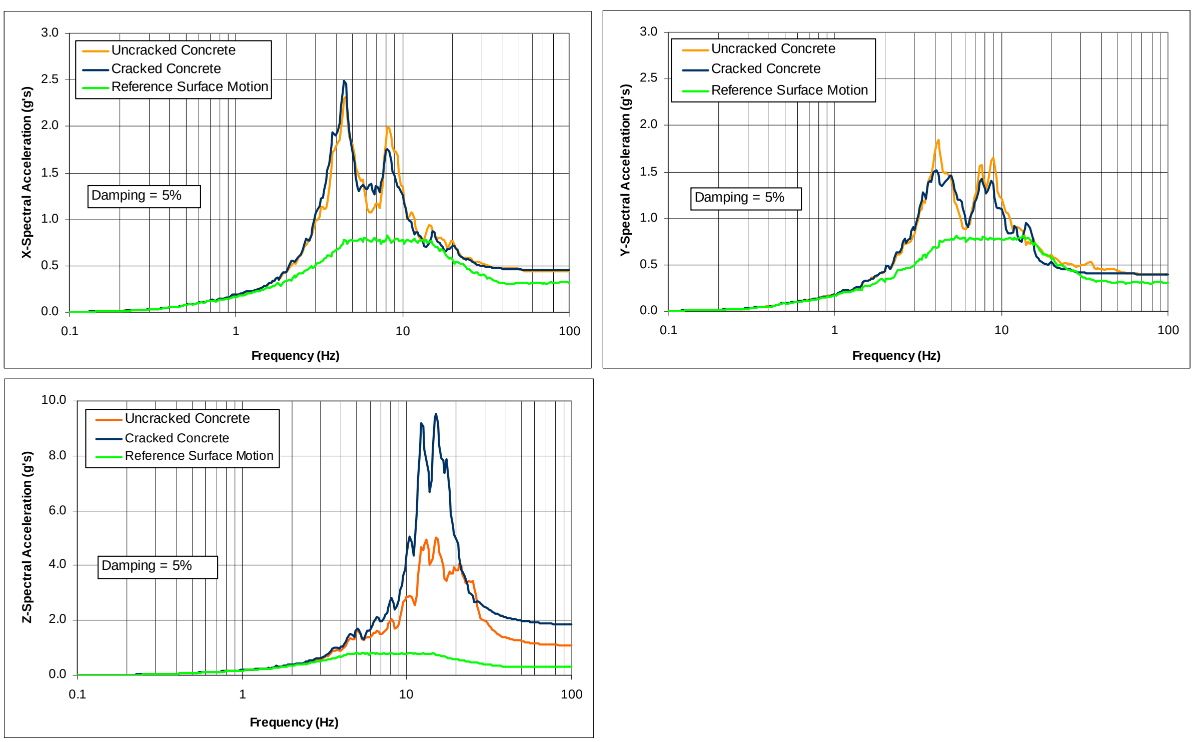

Figure 25: Comparison of Response Spectra in Safeguard Bldg. 4 (Elev. 21.0 m)

Table 6: Comparison of Maximum Acceleration Response, Cracked vs. Uncracked Concrete EPR™

Location	Elev. (m)	Ratio of Cracked to Uncracked		
		X	Y	Z
Center of NI Basemat	-11.85	0.994	0.985	0.934
Reactor Building IS	+5.15	0.900	1.156	0.975
Reactor Building IS	+19.50	1.000	1.109	1.108
Safeguard Building 1	+21.00	0.865	0.933	0.924
Safeguard Building 2/3	+16.30	0.958	0.996	0.978
Safeguard Building 4	+21.00	1.023	1.016	1.706
Fuel Building	+4.20	0.945	0.927	1.018
Reactor Containment Bldg	58.00	1.049	1.129	1.009

An improved method for modeling concrete cracking in SASSI all-shell FE models is being developed. It is an iterative procedure whereby the shear and flexural stiffness of the shell elements are updated based on the calculated strain levels at each solution cycle until convergence is obtained. This procedure is briefly described below.

Step 1: Generate deformation envelopes from the time history responses.

Step 2: Assess the maximum curvature in order to determine whether there is cracking in the concrete plate/shell elements due to bending and to calculate the bending stiffness reduction factor, as shown in Figure 26.

Step 3: Assess the maximum shear strain in order to determine whether there is cracking in the concrete plate/shell element due to shear stress and to calculate the shear stiffness reduction factor, as shown in Figure 27.

Step 4: Update the bending and shear stiffness of all concrete plate/shell elements to be used in the next iteration.

Step 5 Repeat Steps 1 through 4 until convergence is achieved.

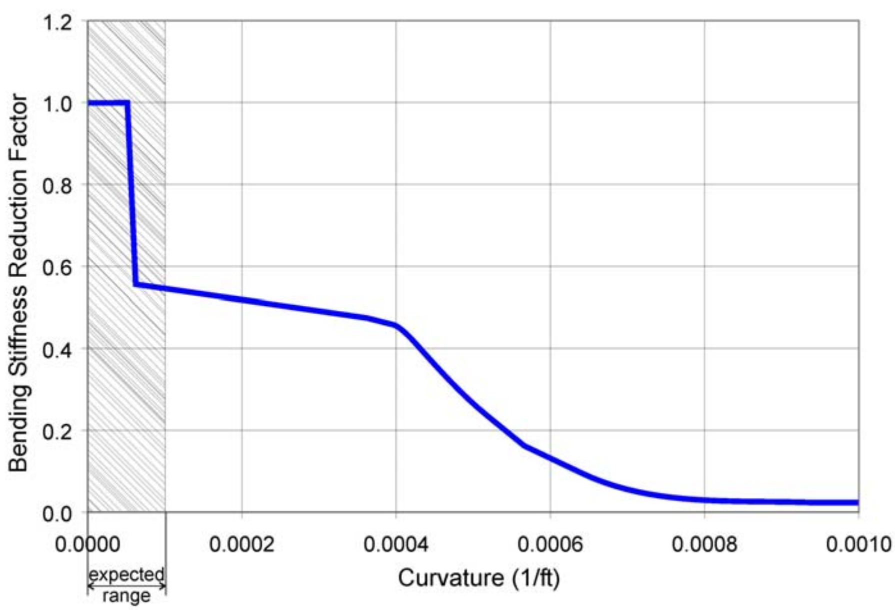

Figure 26: Bending Stiffness Reduction Factor

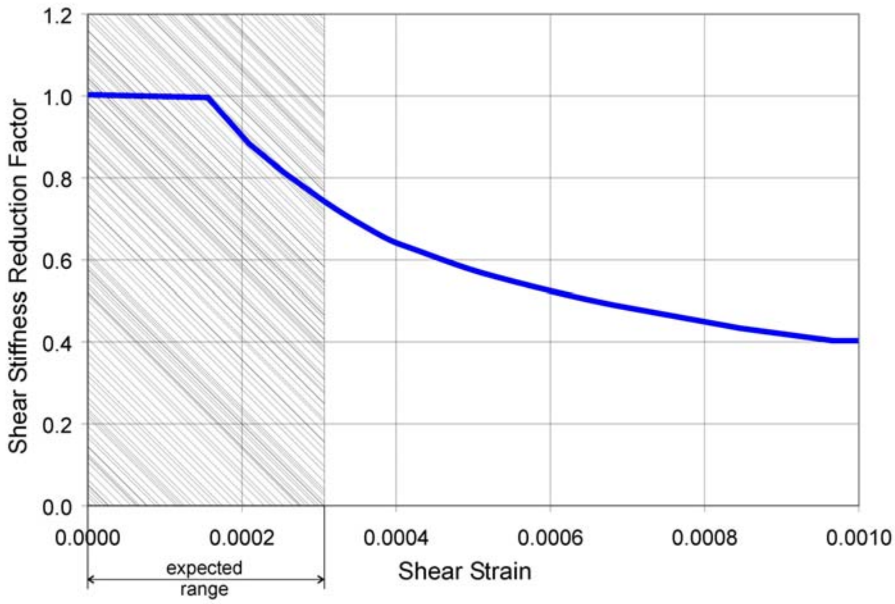

Figure 27: Shear Stiffness Reduction Factor

Effects of Structure-Soil-Structure Interaction (SSSI)

In general, the response of light structures, tunnels and conduits are affected by the nearby heavy buildings due to structure-soil-structure interaction (SSSI) effects. The SSSI effects generally increase lateral dynamic soil pressures on the basement walls of adjacent structures. Recent studies of the dynamic soil pressures on below grade walls using SASSI indicate that normal wall pressures can be significantly affected due to the inertial mass of the structures as well as SSSI effects during seismic shaking. In these studies it is assume that the soil strains near the wall are caused primarily by soil nonlinearities in the free-field. The additional soil strains caused by the soil nonlinearities due to the inertia of the structures are ignored.

To assess the accuracy of ^{MTR}/SASSI in predicting the dynamic soil pressures on below grade walls, the incremental dynamic soil pressures on the US EPR^{TM} exterior walls below grade, calculated using $^{MTR/}$SASSI, are compared with those obtained by ADINA [13]. The SSI analysis was performed for soil case 1n5a and EUR Hard motion. In this study, the foundation basemat and walls were considered without the buildings. Four cases corresponding to rigid-bonded, rigid-smooth, flexible-bonded, and flexible-smooth walls were analyzed; the results are shown in Figure 28 and Figure 29 for the rigid-bonded and rigid-smooth wall cases and in Figure 30 and Figure 31 for the flexible-bonded and flexible-rigid wall cases. The results for the rigid wall are also compared with Wood's solution [14] in Figure 28 and Figure 29. There is good agreement between the results of all three methods.

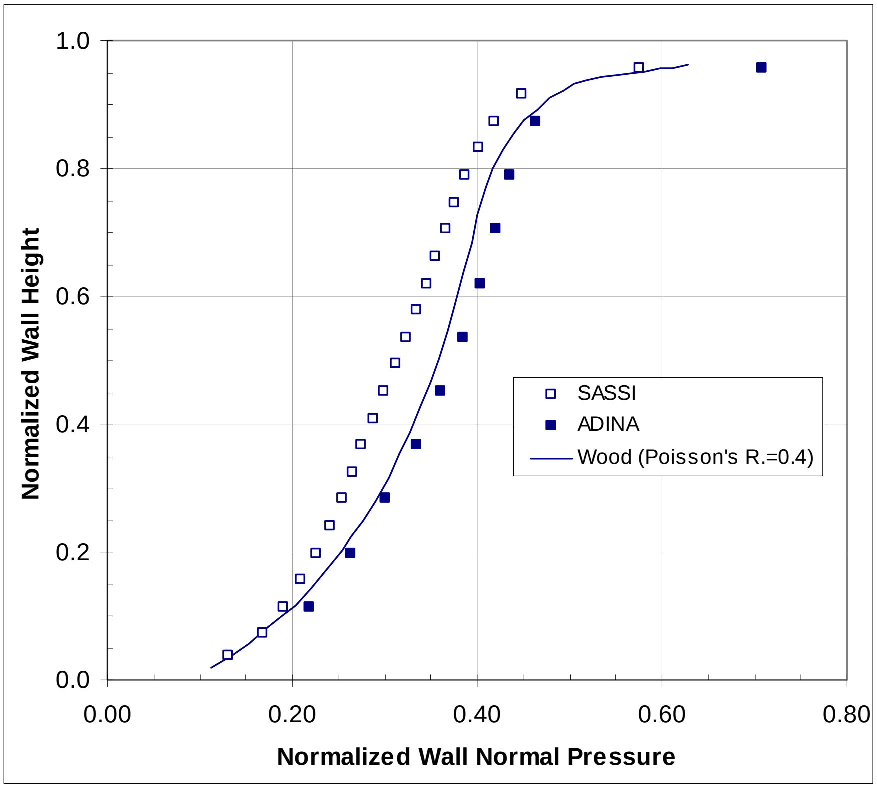

Figure 28: Incremental Dynamic Soil Pressures on Rigid-Bonded Wall

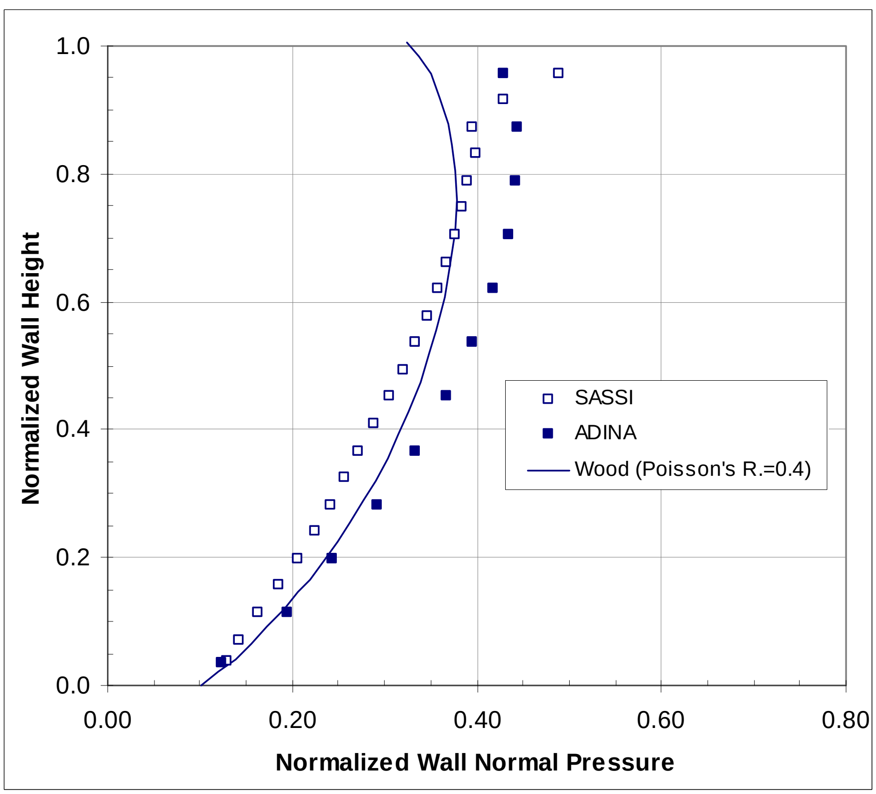

Figure 29: Incremental Dynamic Soil Pressures on Rigid-Smooth Wall

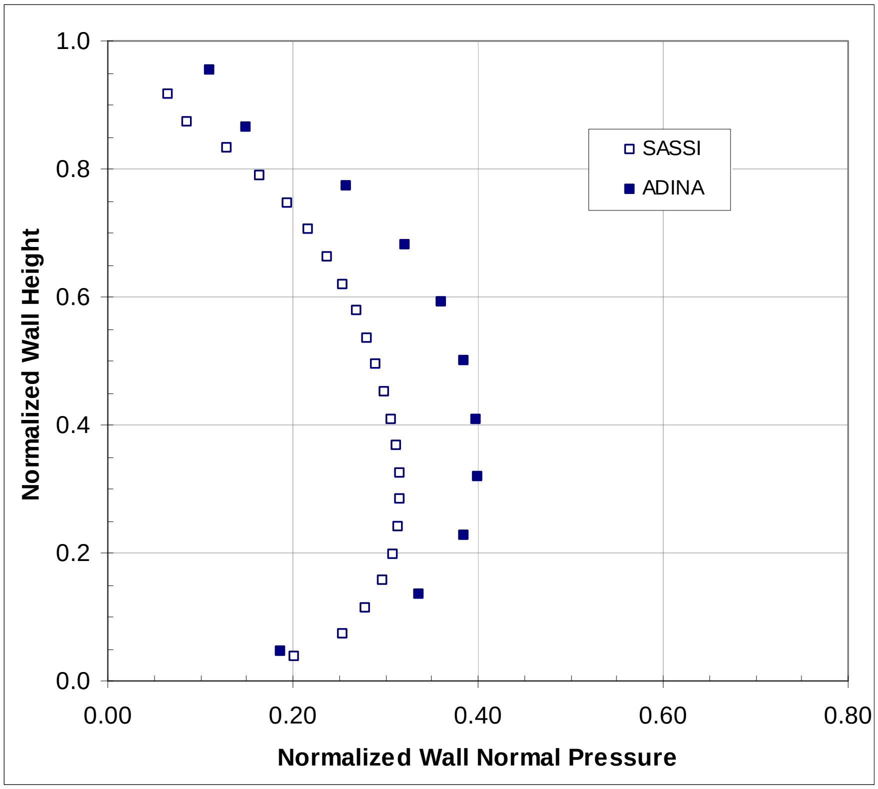

Figure 30: Incremental Dynamic Soil Pressures on Flexible-Bonded Wall

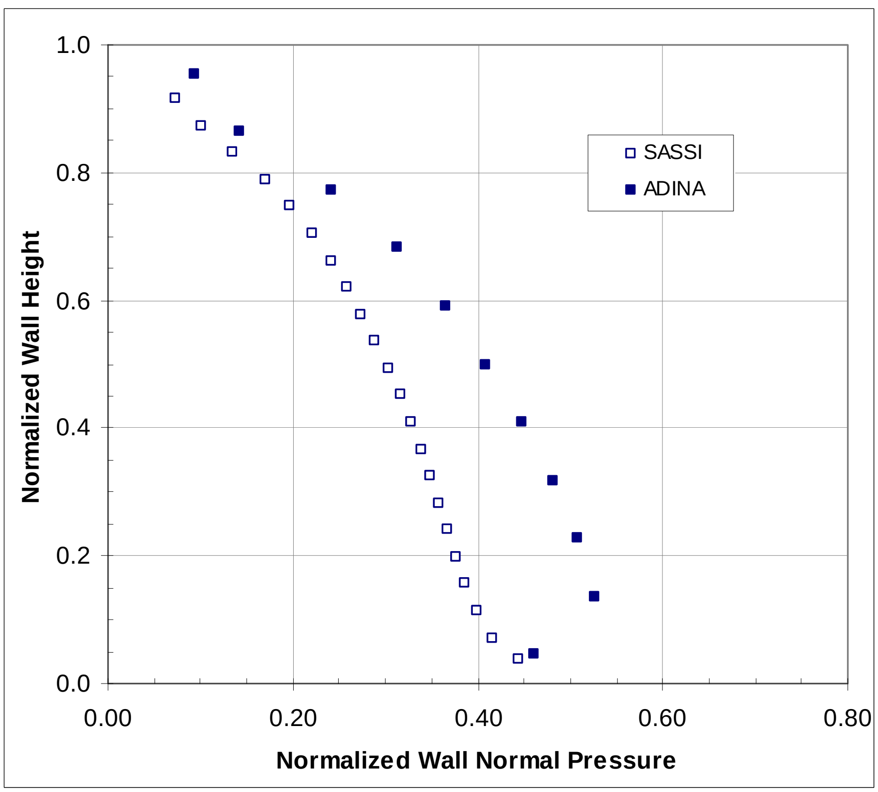

Figure 31: Incremental Dynamic Soil Pressures on Flexible-Smooth Wall

SSI ANALYSIS INCORPORATING SPATIAL INCOHERENCE OF GROUND MOTIONS

The coherence model describes the relationship between ground motions at different locations as a function of the separation distance and the frequency of the ground motions. It is derived from statistical analysis of recorded ground motions at dense array sites, as characterized below:

$$\gamma ij\ (\omega) = Sij\ (\omega) / SQRT\ [Sii(\omega)\ .\ Sjj(\omega)]$$

$$0 \leq |\ \gamma ij\ (\omega)\ | \leq 1$$

Where γij is the coherence function, Sij is the spectral density function, and i and j are two recording stations. Figure 32 shows typical coherence functions in the horizontal and vertical directions. Incoherent ground motions may be characterized as follows:

$$\{UI_g(\omega)\} = [\varphi(\omega)]\ [\lambda(\omega)]\ \{\eta_\theta(\omega)\}\ U_o(\omega)$$

Where:

$\{UI_g(\omega)\}$ = Incoherent ground motion vector

$[\lambda(\omega)]$ = Eigenvalues of coherence matrix

$[\varphi(\omega)]$ = Eigenvectors of coherence matrix

$\{\eta_\theta(\omega)\}$ = Random phase angle (uniformly distributed from 0 to 2π)

$U_o(\omega)$ = Control motion

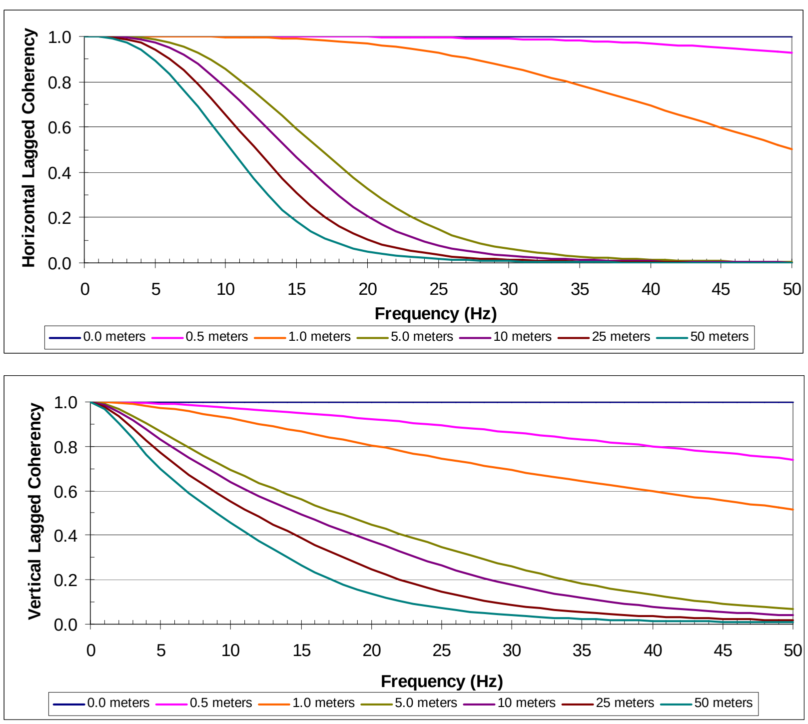

Figure 32: Typical Coherence Functions (Source: EPRI 2007)

The SSI analysis using incoherent ground motions incorporated in ^{MTR}/SASSI consists of the following four steps.

- Step 1: Compute coherence matrix $[\gamma(\omega,d)]$ of incoherent ground motion from prescribed coherency function $\gamma(\omega,d)$
- Step 2: Determine a few significant eigenmodes of coherence matrix (i=1,2, .. m)
- Step 3: Solve SSI response for eigenmodes of Step 2
- Step 4: Combine the results using SRSS method ($\eta_\theta(\omega)=1$)

To evaluate the effects of incoherent ground motion input for seismic response of NPPs founded in hard rock, a test problem obtained from Ref. [15] was analyzed using ^{MTR}/SASSI. The results in terms of the computed acceleration response spectra at the tops of the NI basemat, Steel Containment Vessel (SCV), Auxiliary Shield Building (ASB) and Containment Internal Structure (CIS) are shown for the x-response in Figure 33, for the y-response in Figure 34 for the z-response in Figure 35.

Based on an analysis of the results, the following conclusions can be drawn:

- In general, consideration of the effects of incoherent ground motions on structures with large footprints can result in significant reductions in high frequency components of the response.
- However, incoherent ground motions introduce additional rocking and torsional components in the foundation response that can increase the response of the structure at certain locations.

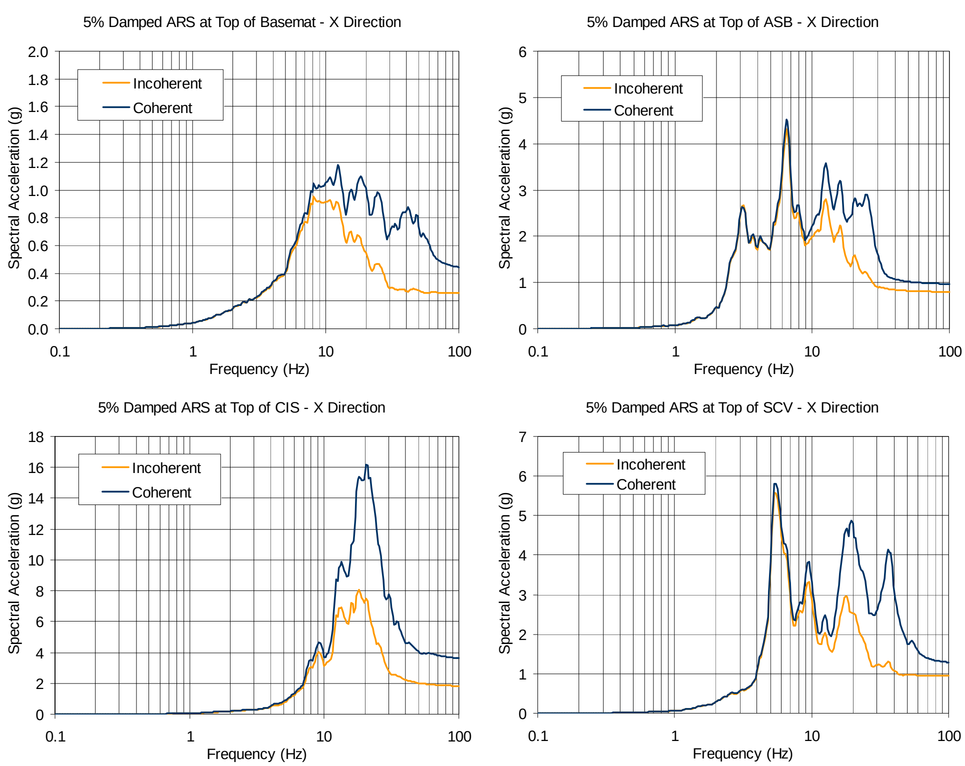

Figure 33: Comparison of Coherent vs. Incoherent In-Structure Response, X-Dir.

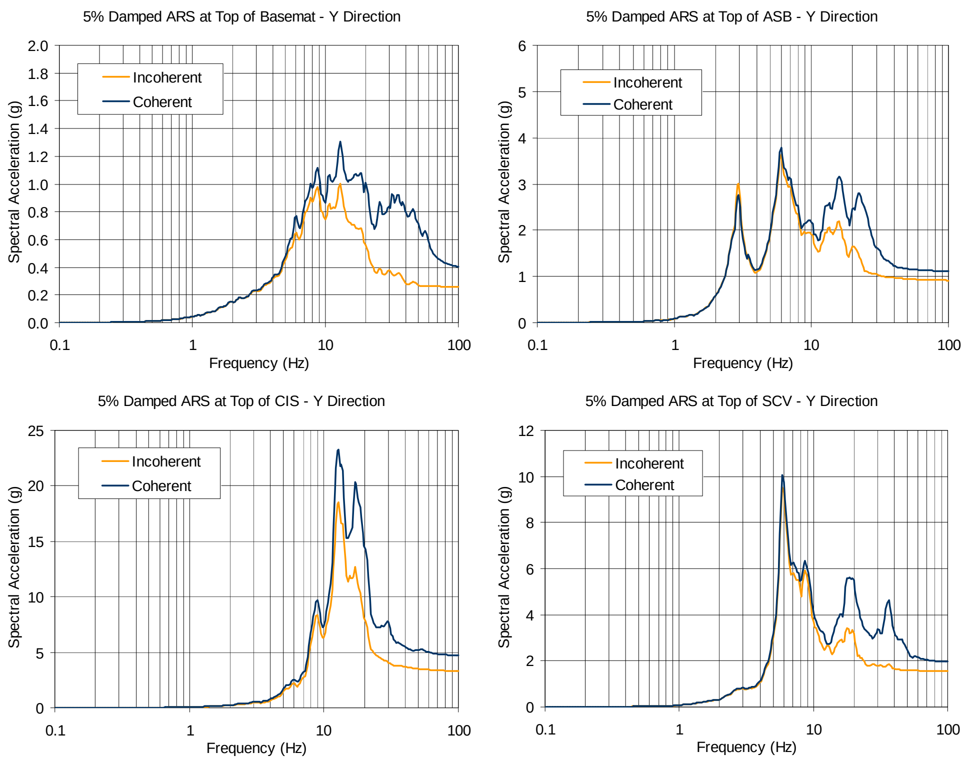

Figure 34: Comparison of Coherent vs. Incoherent In-Structure Response, Y-Dir.

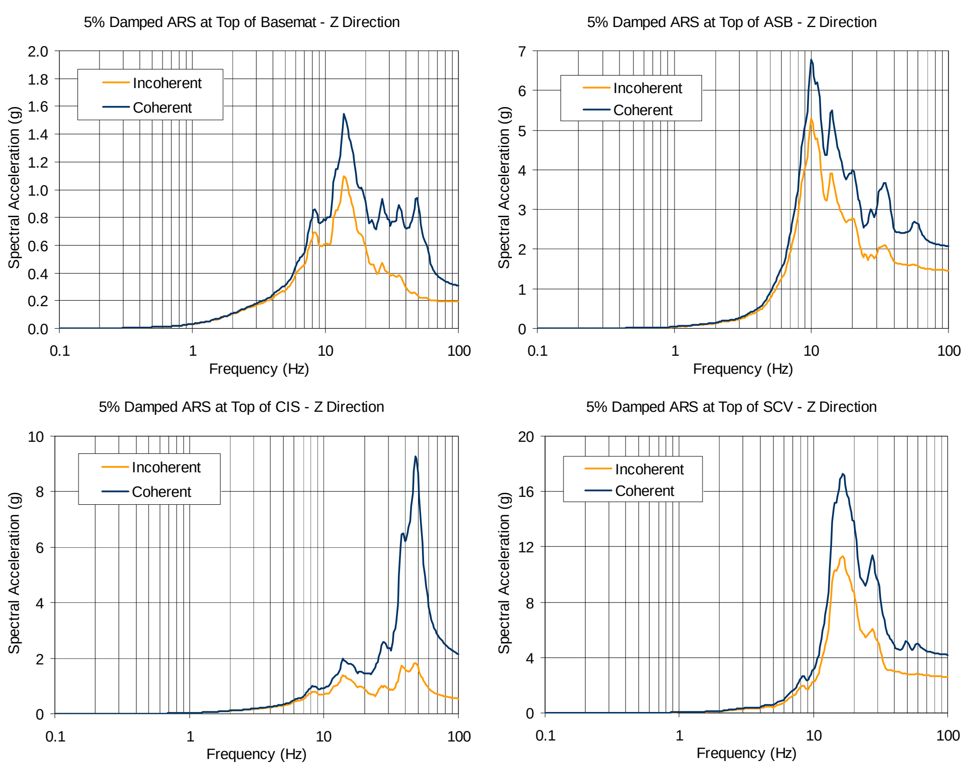

Figure 35: Comparison of Coherent vs. Incoherent In-Structure Response, Z-Dir.

ACKNOWLEDGEMENTS

The computer program SASSI was developed by a team of research students under the direction of the late Professor John Lysmer at the University of California, Berkeley. The code was published in 1981. In the past 30 years significant enhancements to the SASSI code have been made. A more recent advancement of this code includes the capability to analyze large-scale SSI systems, which made it possible to produce some of the results presented in this paper. AREVA NP has been instrumental in supporting the large-scale SSI modeling development as part of its US EPR™ standard design licensing. The support of J. Todd Oswald, Calvin Wong and Daniel B. Fisher of AREVA NP are hereby acknowledged.

Basilio Sumodobila of SC Solutions performed the EPR™ SSI analyses and provided helpful discussions of the results. Dr. Hassan Sedarat of SC Solutions performed the ADINA analyses to develop the dynamic soil pressures on the EPR™ walls as well as providing helpful discussions of the results. Dr. Alexander Kozak of SC Solutions developed the stiffness reduction curves for the proposed method of cracked concrete modeling in SASSI. The contributions of these colleagues are greatly appreciated.

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