

EARTHQUAKE RESPONSE ANALYSIS IN THE CONTEXT OF THE KARISMA BENCHMARK PROJECT

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ABSTRACT

This paper focuses on outcomes of our collaborative work as consultants of the Swiss Federal Nuclear Inspectorate (ENSI) that participated in KARISMA benchmark project organized by IAEA. Structural, soil, and soil-structure-interaction (SSI) models of a reactor building at the Kashiwazaki-Kariwa Nuclear Power Station were developed, and the SSI modeling results were compared to actual signals recorded during the Niigataken-Chuetsu-Oki Earthquake (NCOE). Using the SASSI (System for Analysis of Soil-Structure Interaction) sub-structuring method, a standard-practice approach for SSI analysis, reasonable floor response spectra (FRS) were predicted. Using variations of the same models, non-linear behavior of the site and the reactor building were analyzed separately with increasingly amplified NCOE input signals (up to six times the original signal). Using the SASSI modeling approach, the dynamic stiffness and damping of the site were found to be relatively insensitive to high excitation in the low frequency range (<20 Hz). This lack of sensitivity may be partly due to the fact that the model does not capture extreme non-linear soil behavior, e.g. liquefaction, which would likely occur at such high excitation levels. Non-linear pushover analysis predicted that the reactor building should maintain global stability until excitation exceeds six times the provided NCOE accelerations.

INTRODUCTION

In 2008 the International Atomic Energy Association (IAEA) launched a benchmark project entitled KARISMA (**K**Ashiwazaki-Kariwa **R**esearch **I**nitiative for **S**ismic **M**argin **A**ssessment). This project was created in response to the Niigataken-Chuetsu-Oki Earthquake (NCOE), which occurred on July 16, 2007, off the coast of Japan. The earthquake affected the Tokyo Electric Power Company (TEPCO) Tokyo Kashiwazaki-Kariwa Nuclear Power Station located just 16 km from the epicenter. As consultants of the Swiss Federal Nuclear Inspectorate (ENSI), we conducted soil-structure-interaction (SSI) analysis to predict the seismic response of one of the reactor buildings under NCOE excitation. Figure 1 shows a schematic of the modeled scenario; a stiff, rectangular structure deeply embedded in soft soil.

PREDICTION MODELS

The reactor building and site were modeled independently prior to integrating them together into a SSI model. Results from these independent models were first compared between our two teams, as well as the other participants, to avoid modeling errors and ensure consistency. Comparing results with actual signals recorded during the NCOE, the SSI model predictions were then validated. These recorded signals, as well as all modeling input information, were provided by the organizers of the KARISMA benchmark project, IAEA, in cooperation with the power station operators, TEPCO, Japan (IAEA, 2012).

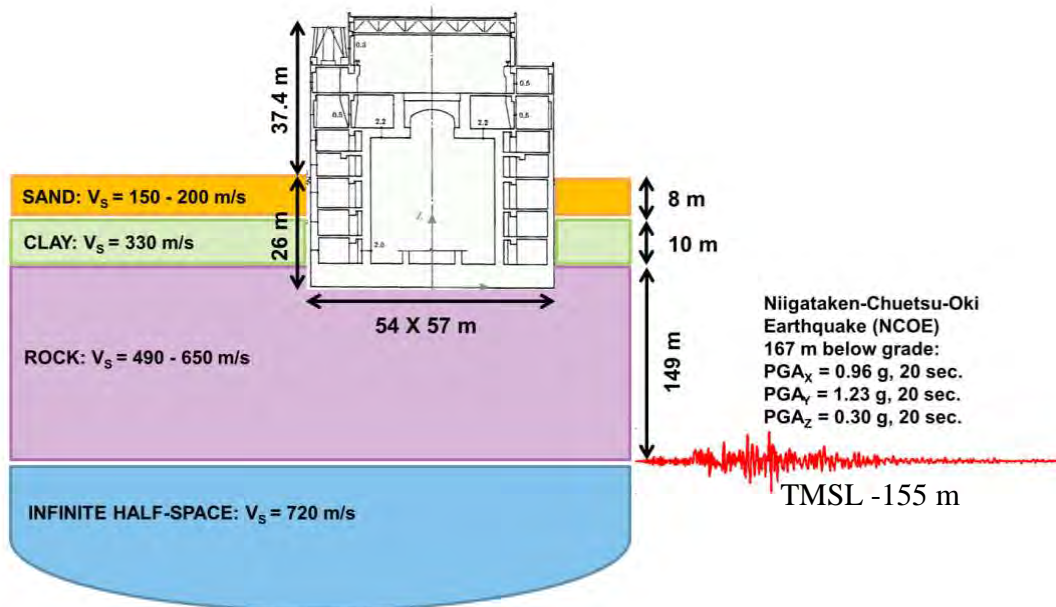


Figure 1. Kashiwazaki-Kariwa Nuclear Power Plant Reactor Building Modeling Scenario.

Reactor Building Fixed-Base Analysis

Fixed-base analyses of the reactor building were performed in order to verify the accuracy of the SOFiSTiK and SAP2000 finite element (FE) models. Using the same information provided for the reactor buildings, independent models were developed as shown in Figure 2. Table 1 shows a comparison of the predominant Eigen modes of the B&H/ENSI 3D FE model (shown below) and the SPI preliminary stick model (not shown). Slight variation is due to the fact that the SPI stick model had point masses and limited degrees of freedom. Also, there is variation due to incomplete information provided for the reactor building, e.g. the location and thickness of auxiliary walls. Modeling teams had to make independent assumptions when information could not be provided.

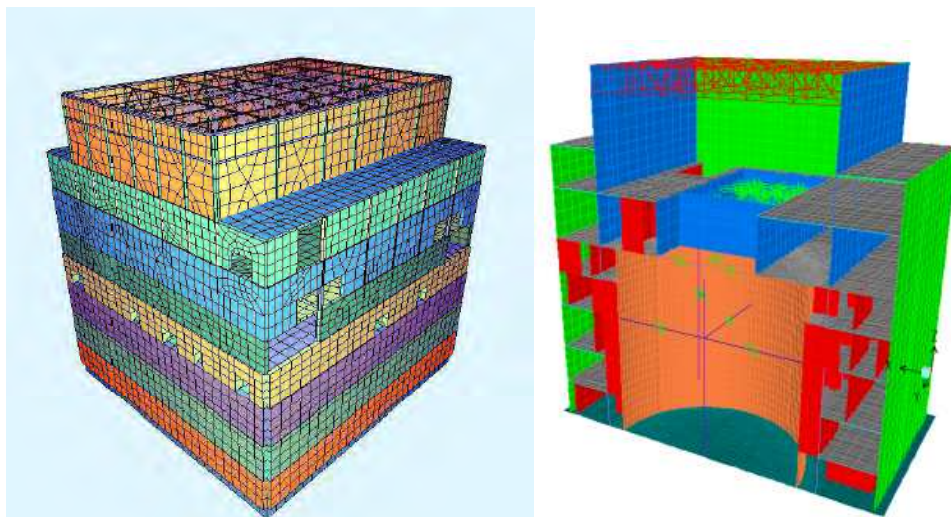


Figure 2. Reactor Building FE Models (Left: SOFiSTiK FE Model by SPI, Right: Section View of SAP2000 FE Model by B&H/ENSI).

Mode	Period		Frequency		Modal Mass Participation					
	(second)		(Hz)		X		Y		Z	
	SPI*	B&H	SPI*	B&H	SPI*	B&H	SPI*	B&H	SPI*	B&H
1	0.21	0.21	4.84	4.84	55.8%	57.0%	0.0%	0.2%	0.0%	0.0%
2	0.19	0.19	5.24	5.33	0.0%	0.8%	57.6	36.8%	0.0%	1.3%
3	-	0.17	-	5.76	-	0.4%	-	15.7%	-	2.8%
4	0.07	0.09	13.85	10.59	0.0%	0.8%	0.0%	0.1%	69.9%	7.2%

Table 1. Comparison of First Significant Eigen Modes.

* A modal analysis was not conducted for the SPI fixed 3D FE model; SPI values are taken from the modal analysis of a reactor building stick model

Ground Motions and Site Response

The assumed soil profile consists of a deep stratum of sand, clay, and soft rock. Figure 1 shows the low-strain shear wave velocity profile provided by the benchmark organizers. One-dimensional site response analyses were performed using the motions shown below in order to develop strain-compatible shear moduli and damping ratios. These motions, provided by the KARISMA organizers, were calculated from recorded signals and specified at the defined outcrop of engineering bedrock (OEB). The OEB is 155 m below the Tokyo mean sea level (TMSL) and 167 m below grade.

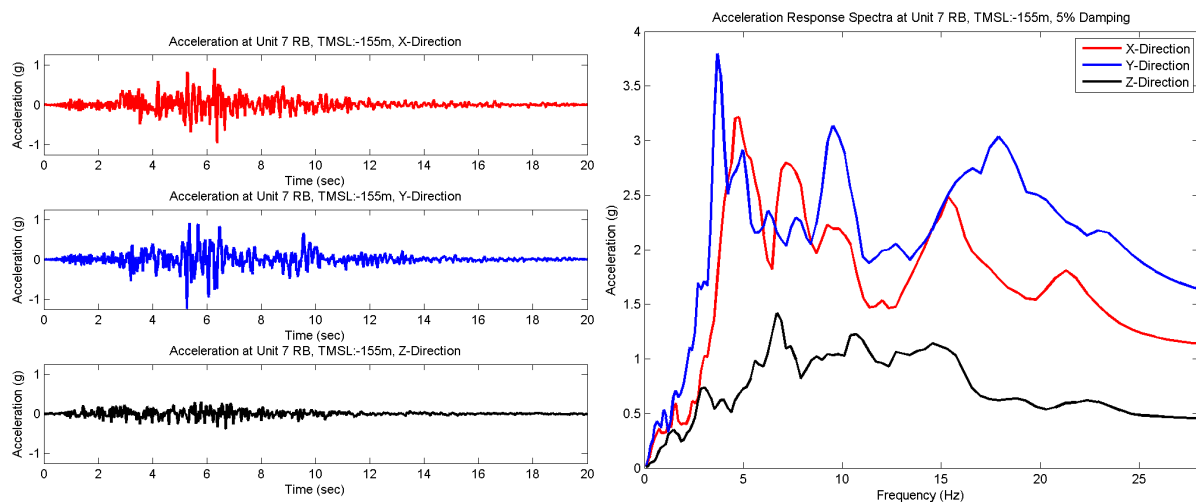


Figure 3. Acceleration Time Histories and Acceleration Response Spectra (ARS) (5% damping) of Input Motions.

SSI Parameters

A hybrid approach was used to generate the SSI model parameters, in which the embedded foundation was assumed to act as a rigid body (Johnson et al., 2010). Using this approach, the unconstrained (flexible) impedance matrices were generated from quarter symmetry models of the excavated soil volume. Discretization of the ACS SASSI foundation models is shown in Figure 4.

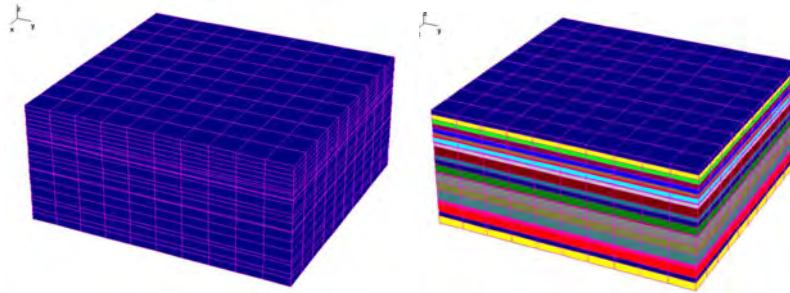


Figure 4. ACS SASSI Quarter Symmetry Models (Left: Rigid Foundation, Right: Excavated Soil).

Scattering functions and foundation impedances were then generated in post processing by applying rigid-body constraints within the embedded foundation region. Figure 4 shows the foundation impedance functions (assuming a rigid foundation) normalized and plotted against dimensionless frequency $a_0 = \omega V_s / r$, where V_s is the average shear wave velocity ($V_s = 500$ m/s) and r is half of the respective foundation dimension ($r = 27$ m, 28.5 m).

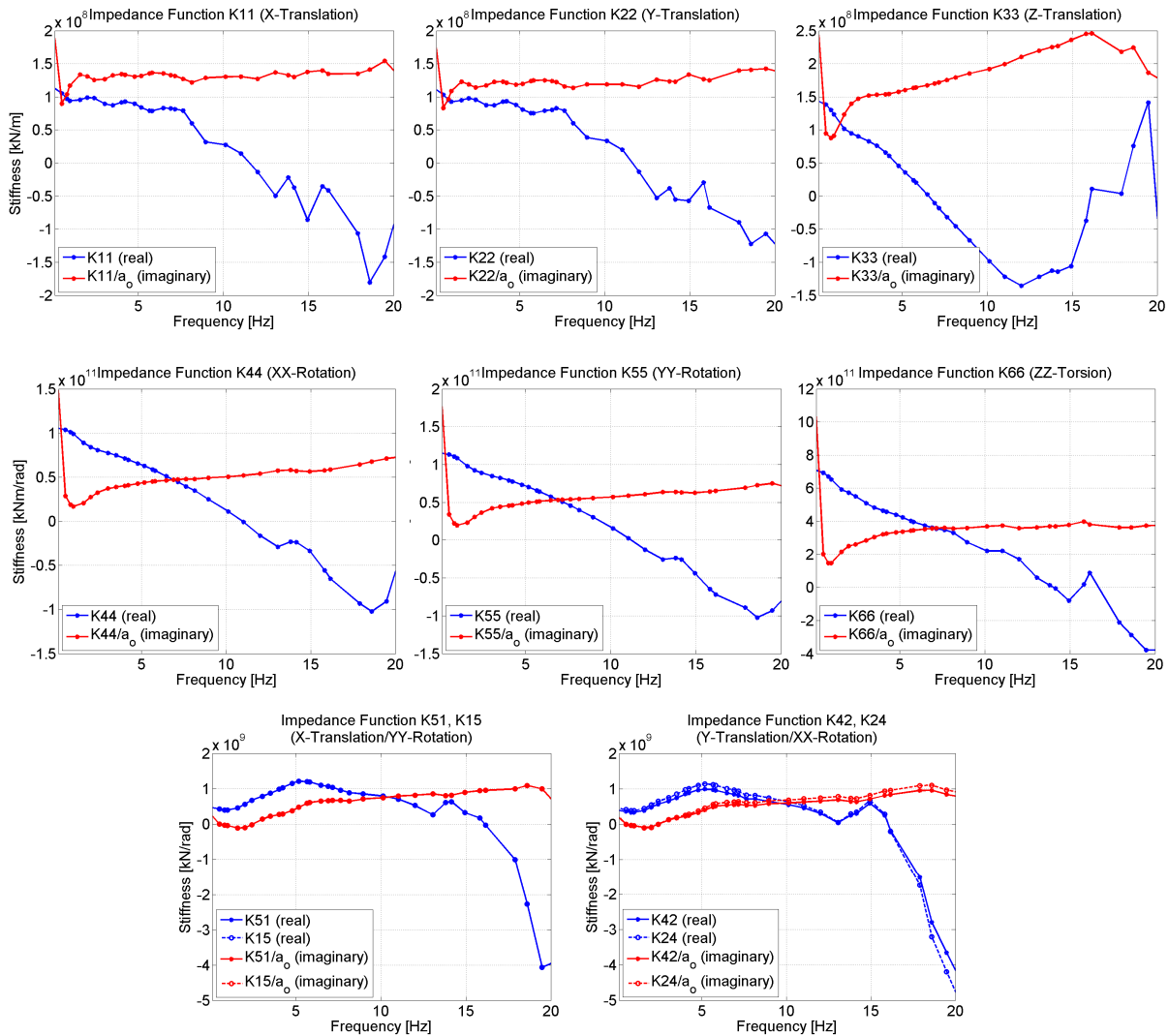


Figure 4. Translational, Rotational, and Coupled Impedance Functions.

The scattering function calculations assume vertically propagating SH and P waves. The following scattering functions relate the foundation input motion to the control motion. In this case, the control motion was defined as the in-column motion at the foundation level.

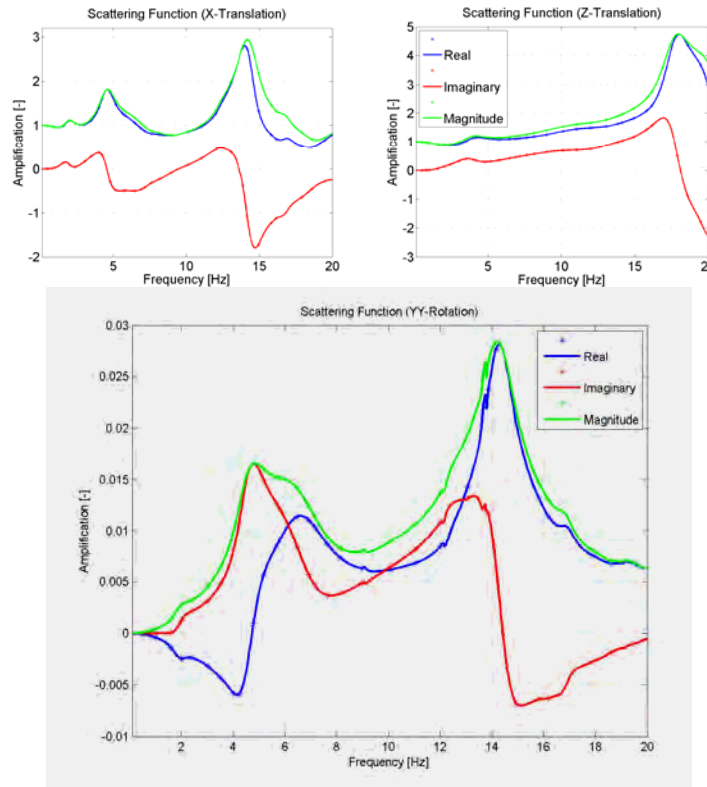


Figure 6. Example Scattering Functions.

Best-Estimate SSI Analysis

In order to predict the reactor building's response to the NCOE, participants were provided with several ground motions. For the reference analysis, participants were asked to use the motions shown in Figure 3 as input excitation. For the best-estimate analysis, the SPI team chose to use a recorded free-field motion adjacent to the reactor building of interest. One-dimensional site response analyses were then performed to deconvolute the earthquake signals to the OEB level, which is 167 meters below grade (TMSL -155 m). Using a similar SASSI program, SASSI 2000, floor response spectra (FRS) at several building locations were calculated and compared to recorded data. Figure 7 shows an example comparison between predicted and measured FRS for both the standard and best-estimate analysis.

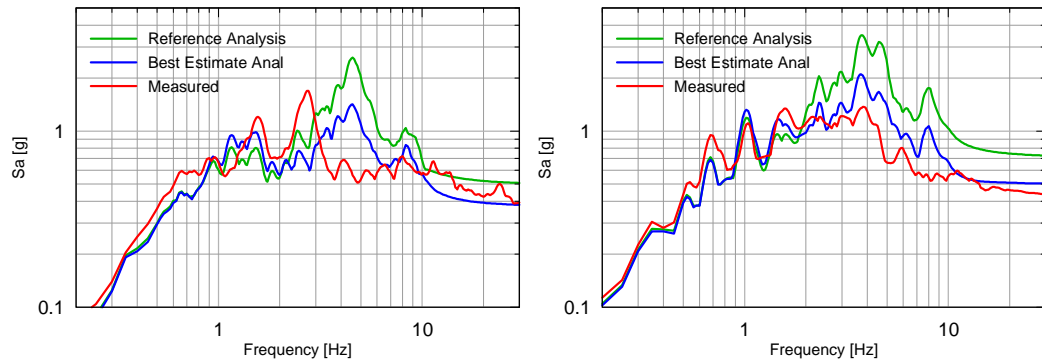


Figure 7. FRS comparison at point FP2, 11.5 meters above grade (Left: X-direction, Right: Y-direction).

NON-LINEAR SOIL AND STRUCTURAL BEHAVIOR UNDER INCREASING EXCITATION

The SSI analysis method used in the first part of this study, SASSI, is a sub-structuring method that assumes linear-elastic behavior. The method does account for some soil non-linearity with strain-dependent shear modulus and damping curves, but it is still a linear calculation. In order to analyze non-linear soil and structure behavior under extreme seismic excitation, the soil and structure were analyzed independently.

Sensitivity of SSI Parameters to Increasing Excitation

A study was conducted using ACS SASSI to examine the variation of strain-dependent soil properties, impedance functions, and scattering functions under increasing seismic excitation. The input NCOE signals were again specified at the OEB, and incrementally amplified to generate four excitation levels, 1xNCOE, 2xNCOE, 4xNCOE, and 6xNCOE. For each excitation level, one-dimensional site response analyses were performed to develop strain-compatible shear modulus and damping ratios, as shown in Figure 8.

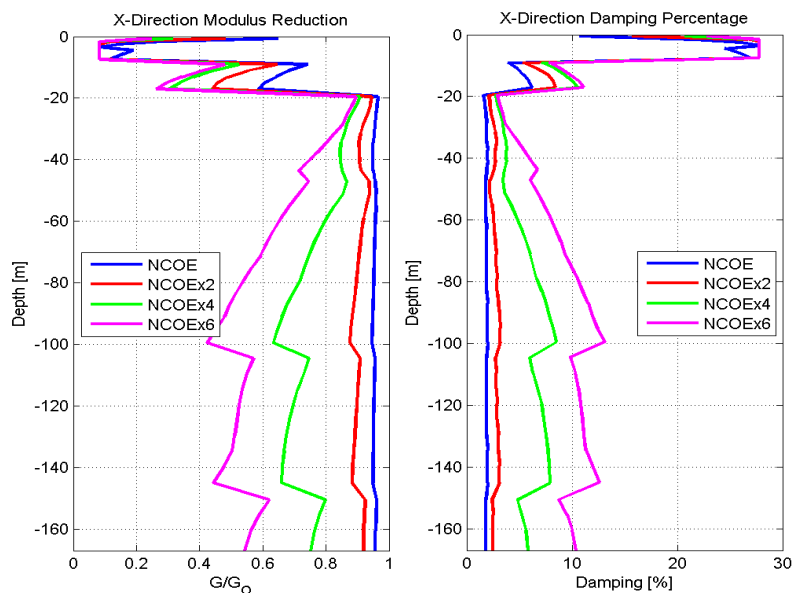


Figure 8. X-Direction Strain-compatible Soil Profiles for Varying Seismic Excitation (Y-direction similar).

The previously described hybrid approach was again used to generate impedance and scattering functions for each level of excitation. Figure 9 shows representative variations in foundation impedance functions (assuming a rigid foundation) in terms of stiffness and damping. Figure 10 shows representative scattering functions that relate the foundation input motion to the control motion, defined here as the in-column motion at the foundation level.

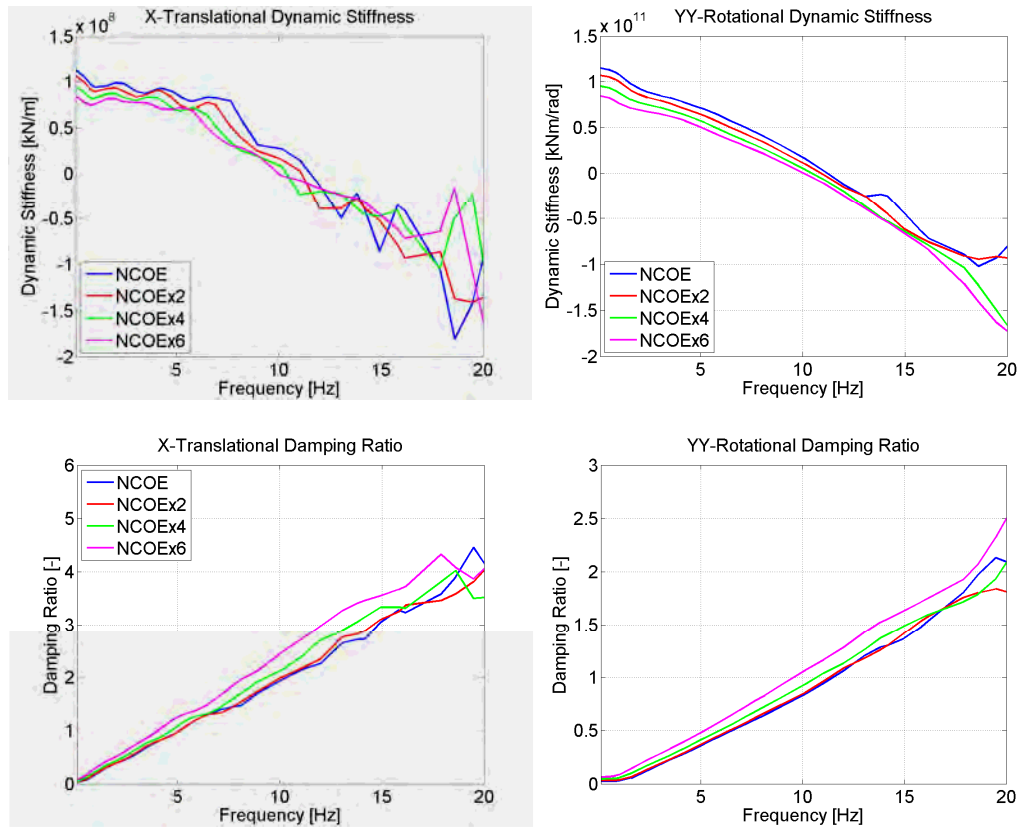


Figure 9. Soil Profiles and Impedance Functions for Varying Seismic Excitation (Y-direction similar).

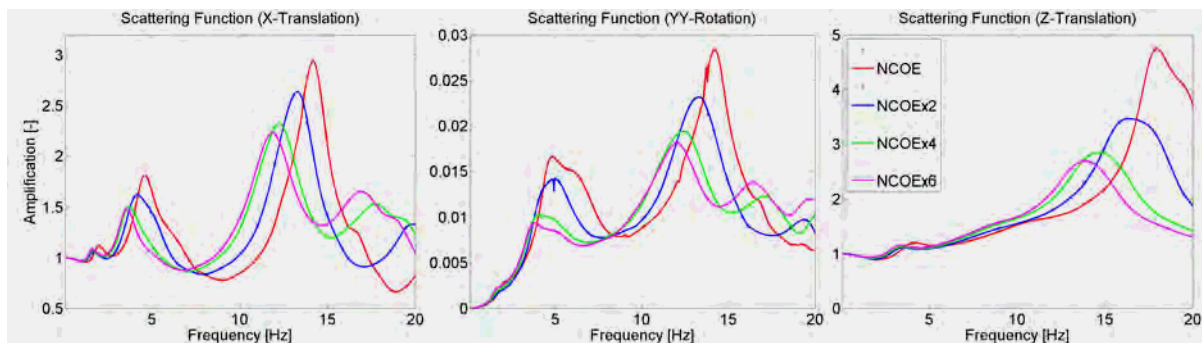


Figure 10. Scattering Functions for Varying Seismic Excitation.

Pushover and Non-linear Dynamic Analyses

A pushover analysis was conducted by SPI using a modified non-linear SOFiSTiK FE model. In this model, the primary reinforced concrete elements, i.e. the main shear walls, pool walls, reinforced concrete containment vessel (RCCV), and auxiliary shear walls were modeled as non-linear shear elements. These elements are consistent with rebar detailing and constitutive material laws provided by the KARISMA benchmark. All other members, i.e. floors, basemat, beams, columns, and steel roof structure) were modeled as linear-elastic. The horizontal load was applied separately in both principle directions as a uniformly distributed acceleration from the bottom of the basemat to the top of the roof. Pushover analyses were conducted for both a fixed-base model and a model considering flexibility of the underlying and surrounding soil. For the latter model, a collection of springs were chosen in accordance with the impedance functions derived for the various seismic excitations (examples shown in Figure 9) and applied over all basemat node points. Identical spring sets were used for the lowest excitation levels (1xNCOE, 2xNCOE, 4xNCOE), and a softer set of springs was used for the 6xNCOE excitation level to account for higher soil strains.

Figure 11 shows the capacity curve of the reactor building, in both X and Y directions, using a flexible boundary to account for SSI. The original force-displacement pushover curve was transformed into an acceleration-displacement response spectrum (ADRS) by simplifying the structure as an equivalent single-degree-of-freedom system. The capacity curve relates displacement of the reactor building 11.5 meters above grade to the acceleration at the basemat. The ductility demand for 6xNCOE was found through an iterative process to be 3.1 in the X-direction and 2.9 in the Y-direction. In Figure 11, the capacity curve is plotted against the elastic and inelastic demand curves, which are based on the 6xNCOE motion applied at the OEB. The results show that reactor building, at a level of 11.5 meters above grade, displaces 11.9 cm in the X-direction and 10.4 cm in the Y-direction during 6xNCOE excitation. Accordingly, the resultant shear force at the basemat is predicted to be 3,922 MN in the X-direction and 3,622 MN in the Y-direction.

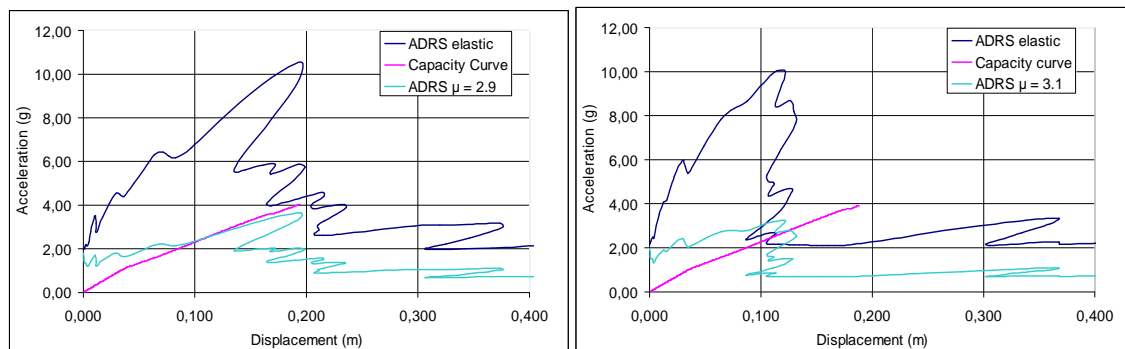


Figure 11. Reactor Building Performance Under 6xNCOE Excitation at TMSL -155 m (Left: X-direction, Right: Y-direction).

Using the same non-linear SOFiSTiK FE models, non-linear time history analyses were performed for both the fixed-base model and the model using boundary springs to account for SSI. For the later model, foundation damping was derived for each excitation level from the imaginary portion of the impedance functions at the fundamental frequency and was introduced into the model as global Rayleigh damping. For each analysis, the provided input motions (1xNCOE, 2xNCOE, 4xNCOE, and 6xNCOE) defined at the OEB were transformed to foundation motions to be applied at the bottom of the basemat. The transformed X, Y and Z motions were then applied simultaneously for each level of excitation. Figure 12 shows the maximum reinforcement steel strains calculated for two outer walls under 6xNCOE excitation, determined from the model considering SSI foundation flexibility. The strains are

less than 15 % and below acceptable performance levels. For this excitation level, maximum story drifts are 0.6% of the building height above grade. This corresponds to a limit state B as defined by ASCE/SEI 43-05 (ASCE, 2005), and indicates only limited damage.

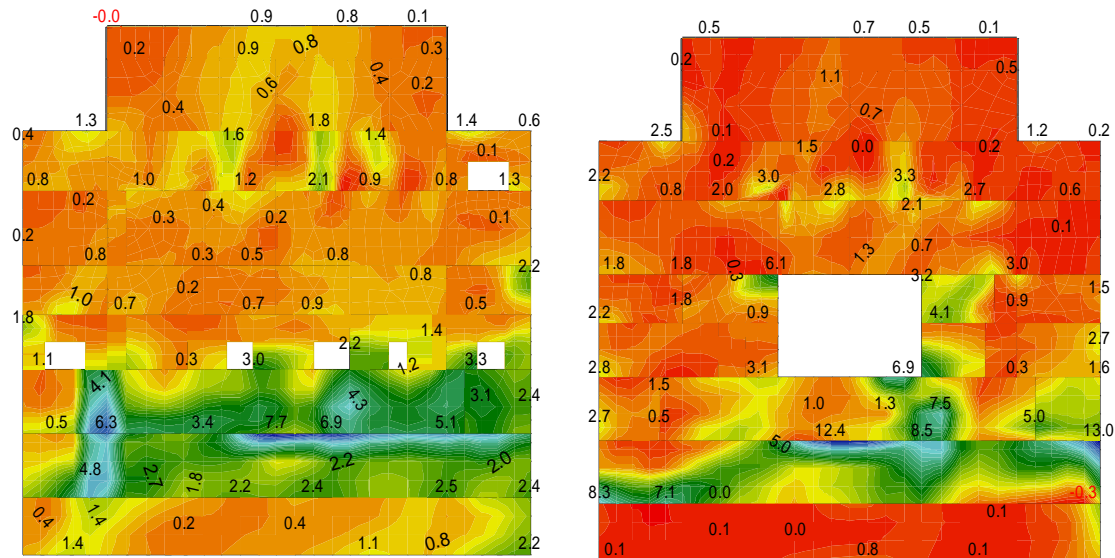


Figure 12. Two Example Exterior Walls Showing Maximum Steel Reinforcement Strain in % under 6xNCOE Excitation at TMSL -155 m.

A High Confidence of Low Probability of Failure (HCLPF) value was derived, and also suggests global stability under extreme seismic excitation. For this calculation, the failure criterion was conservatively assumed to be a story drift limit exceeding 0.006 (based on an ASCE/SEI 43-05). The HCLPF value was defined as the peak ground acceleration (PGA) at OEB. Based on results of the non-linear time history analysis, a HCLPF value of 2.2g was estimated, which is about three times the maximum NCOE excitation.

SUMMARY AND CONCLUSIONS

This study showed that the response of the reactor building and site to the NCOE could be reasonably predicted using a standard-practice approach for SSI analysis, the SASSI sub-structuring method. The analysis had complex models and numerous inputs that required sub-step validation and error checking to ensure an accurate numerical model. For this reason it was important to first validate the structure and the site models prior to engaging in a full SSI analysis. Using the SASSI modeling approach, the dynamic stiffness and damping of the site were found to be relatively insensitive to high excitation in the low frequency range (<20 Hz). This lack of sensitivity may be partly due to the fact that the model does not capture extreme non-linear soil behavior, e.g. liquefaction, which would likely occur at such high excitation levels. Non-linear pushover and dynamic analyses were performed to better understand non-linear behavior of the reactor building under extreme excitation. These analyses found that the reactor building should maintain global stability until excitation exceeds six times the provided NCOE accelerations at TMSL -155 m.

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