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ADVANCED NON-LINEAR TIME-DOMAIN MODELING OF BASE-ISOLATED NUCLEAR POWER STRUCTURES

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ABSTRACT

Seismic base isolation has been implemented in structures all over the world to mitigate the effects of earthquake excitation, however, modern isolators have not been applied to nuclear structure designs despite the large seismic demands for which plants must be qualified. This study uses ALE3D software to analyze three-dimensional, isolated, finite element plant models atop soil columns subjected to design basis seismic inputs in the time domain. A variety of realistic superstructure meshes, coupled and decoupled isolation models, and soil columns are analyzed to illuminate potential design and response issues resulting from certain isolation implementations.

The analyses presented suggest a few caveats regarding seismic base isolation implementation for nuclear power structures. Most plants experienced some uplift/tension in bearings as well as torsion of the isolated superstructure. These quantities were larger for friction bearings, suggesting that care should be taken to consider the magnitude of uplift at a particular site and possible prevention techniques. There is significant correlation between the peak uplift/tension in bearings and the peak global displacement and torsion quantities. The analyses also demonstrated that soft soils induce larger displacement demands on bearing units. The result is that different sites must have independent isolation designs, as one isolation design is not applicable to all sites even for a modularized superstructure design. Finally, the study showed that superstructure slenderness has a negative effect on bearing uplift/tension and subsequent torsion.

INTRODUCTION

Seismic base isolation (SBI) is a mature technology capable of significantly reducing the lateral acceleration response of structures. Domestic interest in SBI application for nuclear power plant (NPP) structures began in the 1980s (Tajirian et al., 1990). Recently, questions regarding three-dimensional SBI system performance including soil-structure interaction (SSI), as well as redundancy, stability, and capacity of the SBI bearings in response to high-amplitude multi-directional seismicity have hindered its implementation. This paper investigates the use of ALE3D, a proprietary finite element software from LLNL, to analyze the non-linear seismic response of 3D isolated NPP models that simultaneously consider the soil column, SBI units, and the superstructure subjected to design basis earthquake (BDE) ground motions in time domain. The results of these simulations are used to assess the efficacy of SBI implementation for NPPs and characterize global response parameters.

BASE ISOLATION

In classical structural engineering, seismic damage is mitigated by designing structures with stiff, bulky members capable of surviving the large seismic forces, or by using flexible members that reduce the forces imparted on the structure. The former paradigm can lead to large, costly structures that impart high accelerations on non-structural components, whereas the latter causes large drifts under earthquake excitation, possibly leading to serviceability issues. SBI is able to combine the two concepts to simultaneously limit superstructure accelerations and minimize interstory drifts.

Structural Dynamics Theory

SBI systems implement a laterally flexible layer between the foundation and superstructure, effectively decoupling them in the lateral direction. For a single-degree-of-freedom (SDOF) system in which mass, m, is held constant, the horizontal dynamic vibration period, T, is inversely proportional to the square root of the lateral stiffness, k. Consequently, even for multiple-degree-of-freedom (MDOF) systems, a large reduction in stiffness will result in a significant lengthening of the fundamental horizontal period. By lengthening the fundamental period, the spectral response moves away from the acceleration-sensitive region, effectively reducing the accelerations transferred to the superstructure from the primary mode.

Additional dynamic advantages arise from the use of SBI. In particular, the response of elastomeric bearings is concentrated in the isolation layer, with minimal interstory drift in the superstructure. Thus, higher mode participation is "filtered-out" since the fundamental mode shape matches the seismic force influence vector. For sliding bearings, the friction layer limits the amount of force that can be transferred to the superstructure. In this case, the bearings are said to act as a "fuse".

Types of Seismic Isolation Bearings

Documented proof of SBI as an engineering concept dates back to the early 20th Century (Kelly, 1986). Early suggestions for isolation material included using talc, sand, or rollers. As the study of base isolation and materials progressed, two distinct SBI concepts were invented: elastomeric and sliding bearings. Although similar in purpose, the mechanics, dynamics, and forms of damping differ between the two bearing types.

Elastomeric bearings introduce thin, alternating layers of rubber and steel that provide vertical stiffness and stability but are horizontally flexible. As the bearings displace laterally, the vertical stiffness decreases, reducing the height of the bearing. Various types of elastomeric bearings differ in how damping is introduced to the system. Low-damping rubber (LDR) bearings have low critical damping values in the range of 2-3% (Kelly and Konstantinidis, 2011), and thus, the horizontal response is nearly linear. High-damping rubber (HDR) bearings use rubber vulcanized with fillers (e.g. resins or carbon black) that provide equivalent linear viscous damping of 10-20% (Naeim and Kelly, 1999). Non-linear response and low-cycle softening of HDR bearings are caused by the interaction of filler chains and by strain crystallization. Finally, lead-plug rubber (LR) bearings use low-damping rubber and vertically oriented plugs in the bearing that yield during deformation to provide damping. High-amplitude displacement has been shown to induce temperature increase in the lead material, softening the plug and reducing damping.

Sliding bearings use a low-friction interface between the foundation and superstructure that limits the transmittable force. To provide recentering force, most sliding bearings have spherical surfaces. The radius of the sliding surface controls the dynamic period of the structure and the coefficient of friction of the surface controls the damping ratio. Various sliding bearings differ by the number of sliding surfaces. The original friction pendulum (FP) bearing design has a single sliding surface that provides linear behavior after static friction has been overcome. The triple friction pendulum (TFP) bearing is composed of five pieces with four sliding planes. The radius and friction coefficient of each slider can be designed to provide different performance goals under various excitation levels.

Analytical Models of Seismic Isolation Bearings

The analyses presented herein consider 4 separate SBI models. These models include two sliding and two elastomeric models. The first sliding model is a robust TFP bearing proposed by Becker (2011) and termed "spring3" in ALE3D. This model is a two-dimensional, non-linear, kinematic model that tracks the location of each individual slider on its respective sliding surface. The research is extended to three dimensions by adding a vertical geometric constraint that simulates the coupled spherical motion but also allows for uplift. At the time of this paper, this model was not fully completed, and thus, results for this model are left as future work.

The simplified FP model termed "spring2" in ALE3D was presented in Nagajaraiah (1991) and serves as the numerical basis for friction bearings in commercial software such as SAP2000. This model uses a hysteretic model in conjunction with a pendulum component. Although this model does couple the vertical and horizontal response through scaling by the normal force, it does not constrain the vertical motion induced during horizontal displacement.

The simplified LR model termed "spring4" in ALE3D was presented in Nagajaraiah (1991) and serves as the numerical basis for elastomeric bearings in commercial software. This model uses a hysteretic model for the lateral response, with a completely decoupled vertical response.

The robust LR model termed "spring5" in ALE3D uses the Kalpakidis et al (2010) model for the lateral response. This model is similar to "spring4", but additionally accounts for reduction in lead plug strength as a result of temperature change induced by multiple load reversals. The model presented in Warn and Whittaker (2006) is used to determine the vertical stiffness of the bearings as a function of lateral displacement, and therefore couples the vertical and horizontal response.

All models were validated using dynamic load cases from the accompanying reports. The results of those analyses are presented in Keldrauk (2012).

Seismic Isolator Design and Layout

The isolation systems were designed to have the same quantity and layout for the same superstructures, regardless of bearing type. The target effective isolation period, T_{eff}, and equivalent damping, z_{eq}, were set at 3 sec and 15% respectively. With these dynamic properties, the mean lateral acceleration and displacement demands were 0.27 g and 61 cm, respectively. A safety factor of 2 was implemented, making the MCE displacement 120 cm.

Design superstructure weights were increased by 20% to account for live loads. The final overbearing forces for the cylindrical and rectangular plants were 420 MN and 2770 MN, respectively. The isolation system for these two structures consisted of 32 and 220 bearings, respectively.

The design of the individual bearings was predicated on displacement demand, target effective period, target equivalent damping, and general SBI design practices. The final LR bearing had an 80 cm radius, with a lead core radius of 17 cm. The bearing consists of 31 rubber layers, each with a thickness of 1.0 cm. The total bearing height is 40 cm. The FP bearing has a sliding surface radius of 3.05 m and a friction coefficient of 0.07. Relevant material properties for each bearing are listed in Keldrauk (2012).

SUPERSTRUCTURE MODELING

Geometry

Two superstructure designs were analyzed herein: a cylindrical plant representative of a typical conceptual design, and a rectangular plant representative of an Advanced Boiling Water Reactor (ABWR). The geometry of these plants was presented in (Xu et al., 2006). The cylindrical plant is slender and light-weight in contrast to the rectangular plant which is more stout and heavy. The cylindrical plant is 26.0 m in diameter and 46.0 m tall. A central wall serves to break the structure into two cavities for the reactor and power conversion vessels. The rectangular plant has a square footprint 60.0 m in length and 78.0 m tall. The structure has 3 interior floors including the base mat and interior walls that form a central vessel in the structure. The plants have total dead-load weights, including equipment, of 350.5 MN and 2305 MN, respectively. They are used to examine the difference in response between light and heavy isolated structures, as well as to observe the global response of NPPs with SBI including overturning, uplift, and torsion.

Material

All structural members were composed of idealized reinforced concrete. As defined in Xu et al, the concrete has a compressive strength, f'_c = 27.6 MPa, weight density, w_c = 23.6 kN/m^3, Young's modulus, E_c = 26.4 GPa, and Poisson's ratio, ρ = 0.20. Additionally, steel reinforcement with a yield strength, f_y = 413.7 MPa, was added in all three directions at a constant volume ratio, ρ_s = 1.0% (no explicit rebar locations or sizes was required). In ALE3D, this material is modeled using a non-linear, inelastic DTRA model with a rebar option.

Equipment

The response of plant equipment as well as its interaction with the superstructure is considered by implementing eight lumped mass stick (LMS) models at various locations within the superstructure. The LMS models were designed by Quazi Hossain (2012) and scaled by Keldrauk (2012) to meet the equipment weight specified in Xu et al (2006). Stiff beams were added at connection points between the LMS and structure in order to spread the gravity load imparted by the heavy equipment. Despite these beams, a small amount of concrete inelasticity was observed at connection points. Similar beams were added at isolator connection points to the superstructure and the foundation.

GEOTECHNICAL SPECIFICATIONS

Soil Characterization

The soil column is modeled with a square footprint twice the width of the superstructure used, and 80 m tall. Soil layer properties were chosen to match those presented in Xu et al, including weight density, γ = 17.28 kN/m3 and Poisson's ratio, ρ = 0.3. Three soil stiffnesses are analyzed with associated shear wave velocities, β, of 1000 m/s, 625 m/s, and 250 m/s (IBC soil classifications B, C, D, respectively). The shear wave velocity is assumed to be constant over the height of the soil column. Finally, a case with no soil is analyzed where ground motions are applied directly to the bottom of the SBI elements, such that SSI is not considered. The no soil (N) case serves to show the accuracy of analyses not considering SSI.

Applied Ground Motions

Safety-critical facilities such as NPPs pose a higher risk to the environment, and are therefore designed to a Uniform Hazard Spectrum (UHS) specified at a mean hazard annual frequency of exceedence, H_D = 1x10^{-4}. The 30 three-component scaled ground motions from probabilistic seismic hazard analysis (PSHA) for the Diablo Canyon NPP (Abrahamson and Gregor, 2012) are used as inputs at the bottom of the soil column. These ground motions are listed in Keldrauk (2012). Note that for some combinations of superstructure, isolator type, and soil column, only 3 motions, referred to as the basic motions, are applied to the models.

Ground motion records are applied as velocity time histories to the bottom of the soil columns. Non-reflecting boundaries are applied at the sides of the soil column, thereby canceling the elastic portion of a propagating wave hitting the boundary. This simulates an infinite halfspace as long as no plastic deformation occurs. Hourglassing in the soil column was reduced by not applying the non-reflecting boundary condition at corner nodes.

RESULTS

153 different numerical analysis cases comprised of 397 simulations requiring a total of 108,884 processor-hours were run using ALE3D. Detailed peak responses are depicted in Keldrauk (2012). The results are summarized in the following sections.

Isolator Deformation and Uplift Response

The mean values of peak isolator compressive deformation, $dz_{c,max}$, peak tensile deformation/uplift, $dz_{t,max}$, and peak lateral deformation, Δ_{max}, are listed in Table 1 for the simulations atop each soil type subjected to the basic motions. Additionally, this chart shows the percentage of the lateral displacement limit, Δ_{lim} (calculated as 123 cm and 124 cm for the FP and LR cases respectively) that was reached. It should be noted that no N or B test cases exceeded the lateral deformation limit, however the C and D cases exceeded the limits in 6% and 33% of the cases, respectively.

Table 1: Mean isolation peak displacement data for the analyses subjected to the basic motions

Soil Type	$dz_{c,max}$ (cm)	$dz_{t,max}$ (cm)	Δ_{max} (cm)	$\Delta_{max}/\Delta_{lim}$
N	0.97	0.57	57.0	46%
B	1.59	3.73	59.5	48%
C	1.91	6.17	69.1	56%
D	2.04	8.04	107.9	88%

Because the metallic sliding components of friction bearings provide increased compression stiffness that is approximately one order-of-magnitude greater than elastomeric bearings, but offer no tension resistance, the vertical deformation extrema are notably different for the two bearing types. For the B soil cases, the median $dz_{c,max}$ value for the sliding bearings was an average of 90% less than that for the elastomeric bearings. The other soil cases showed only a minor variation from this percentage, but with no set pattern. The median peak uplift for the rectangular superstructure cases with friction pendulum bearings atop soil column B in response to all ground motions was 6.5 times greater than the median tension deformation of the elastomeric bearings for the same cases. The standard deviation of the uplift/tension deformation data for the FP and LR simulations was 1.89 cm and 0.19 cm respectively. This data shows that FP bearings lacking uplift restraint exhibit significantly greater vertical motion than LR bearings.

For the comparative cases with both superstructures using the basic motions, the ratio of FP uplift to LR tension deformation under the same motions is shown to be 0.80 for the N soil cases, but grows to 11, 16, and 18 for the B, C, and D soil column cases respectively. This suggests that softer soil columns increase the tension response of SBI bearings, an effect that is more pronounced for FP bearings lacking tension restraint. Furthermore, the mean tension/uplift peaks increase by 12% in the cases utilizing the more-slender cylindrical superstructure in comparison to the rectangular superstructure.

The data from the rectangular superstructure simulations atop soil column B in response to the full set of motions showed median (standard deviation) $\Delta_{max}/\Delta_{lim}$ ratios of 34% (20%), 32% (16%), and 32% (20%) and maximum ratios of 81%, 73%, and 90% for the FP, simplified LR, and robust LR runs respectively. For these simulations, the maximum ratios are 2.35, 2.56, and 2.90 standard deviations above the respective median ratios, and the failure level is 3.30, 4.25, and 3.40 standard deviations above the median ratios. These numbers mirror suggested code ratios and present the largest discrepancy between the simplified and robust LR model responses, implying that consideration of lead core heating is very important for properly characterizing peak lateral displacements and predicting possible strainbased limit states. Considerable softening of the lead core under high-amplitude seismicity is also the likely cause of the increase in $\Delta_{max}/\Delta_{lim}$ ratio standard deviations from simplified to robust LR simulations.

Finally, comparison of Δ_{max} data from the rectangular and cylindrical test cases suggests that tall, slender structures may respond less-favorably to earthquake excitation, especially for the soil cases. For said cases, the mean lateral deformation peaks are shown to be 1%, 16%, 55%, and 81% greater for the cylindrical superstructure analyses utilizing the soil N, B, C, and D columns respectively. This points to a possible confluence of soil column and slenderness effects. Although too few comparative simulations were run to make any sweeping conclusions, this issue should be analyzed more-extensively when discussing base isolation applicability.

Global Displacement, Torsion, and Overturning

Mean directional peak displacement amplification factors at the isolation level ($A_{i,x}$, $A_{i,y}$, and $A_{i,z}$) and roof level ($A_{r,x}$, $A_{r,y}$, and $A_{r,z}$) for the comparative cases with all combinations of soil column, isolator type, and superstructure mesh subjected to the basic motions are given in Table 2. These are calculated as the peak displacement at the given location normalized by the peak foundation displacement in the same direction. These values show minimal amplification of vertical displacements, but significant amplification of lateral displacements from the foundation to the superstructure levels. Furthermore, the lateral amplification at the roof is shown to be considerably higher than the lateral amplification at the isolated-slab level, suggesting that overturning rotations can exacerbate peak lateral displacements at the roof level. Finally, the peak displacements appear to increase as soil columns become more flexible. Because of this, a specific SBI design should be associated with soil columns of similar stiffnesses, as softer columns may necessitate isolators with larger deformation capacities and isolation gaps with larger displacement and rotation capacities.

Comparison of superstructural displacement values shows that for the comparative cases where all combinations of soil column, isolator type, and superstructure mesh are analyzed, the simulations utilizing the cylindrical superstructure experienced a 36% increase in peak X displacements, and a 26% increase in peak Y displacements over the rectangular superstructure. Although there was no noticeable change in Z displacements between the two, this data suggests a slenderness effect.

Mean values of the peak torsion, $\theta_{r,max}$, and peak overturning rotation values, $\phi_{rx,max}$ and $\phi_{ry,max}$, at the roof level for all simulations subjected to the basic motions are presented in Table 3. Although some of the angles seem small, remember that a rotation of 1.0° results in over 0.50 m of differential displacement across a 30 m distance. Although there are not enough cases to make definitive conclusions, the values show a clear trend that both torsion and overturning increase as the soil columns become softer.

The bearing type has a considerable effect on peak rotation values, which is likely attributable to the lack of uplift restraint. For the simulations utilizing the rectangular superstructure atop soil column B in response to all ground motions, the median (standard deviation) of the peak isolated-slab and roof torsion, $\theta_{i,max}$ and $\theta_{r,max}$, are shown in Table 4. At both locations, the median value of the peak torsion is at least twice as large for the FP-isolated cases as for the LR-isolated cases. Note that neither structure was specifically designed to minimize torsion. FP bearings are lauded for their ability to mitigate rotation due to structural eccentricity, yet LR bearings were more effective in reducing torsional effects. Analysis of time history records showed that significant torsion in FP structures initiated around the same time as primary uplift in any the outer bearings. Table 5 presents the median (standard deviation) values of the amplification factors for peak directional overturning rotation of the isolated slab ($\alpha_{i,x}$ and $\alpha_{i,y}$) and roof ($\alpha_{r,x}$ and $\alpha_{r,y}$) above those of the foundation slab in the same direction. These values are 13% to 32% larger for the FP cases, suggesting that the FP bearings induce larger overturning in the isolated superstructure than LR bearings.

Finally, the slenderness effect is reinforced by the values of torsion and overturning for the comparative cases utilizing all combinations of soil column, isolator type, and superstructure mesh. The simulations involving the cylindrical superstructure experience peak torsion and overturning values that are, on average, 4.17 times and 6.73 times larger than the same values for the rectangular superstructure cases respectively.

Table 2: Mean directional peak displacement amplification factors at the isolation and roof levels

Soil Type	$A_{i,x}$	$A_{i,y}$	$A_{i,z}$	$A_{r,x}$	$A_{r,y}$	$A_{r,z}$
N	1.56	1.42	1.01	1.62	1.48	1.03
B	1.46	1.22	1.12	1.80	1.66	1.17
C	1.60	1.26	1.12	2.03	1.89	1.17
D	2.18	2.28	1.13	2.84	2.89	1.14

Table 3: Mean peak roof rotations for the simulations subjected to the basic motions

Soil Type	Structure Type	$\theta_{r,max}$ (°)	$\phi_{rx,max}$ (°)	$\phi_{ry,max}$ (°)
N	Cylindrical	0.081	0.021	0.025
B	Cylindrical	0.30	0.31	0.35
C	Cylindrical	0.60	0.45	0.47
D	Cylindrical	0.75	0.66	0.62
N	Rectangular	0.0079	0.13	0.0097
B	Rectangular	0.035	0.63	0.057
C	Rectangular	0.059	0.12	0.086
D	Rectangular	0.13	0.30	0.25

Table 4: Median (standard deviation) values of the peak torsion anglesat the isolation and roof levels for the rectangular structure atop soil column B

Bearing Model	$\theta_{i,max}$ (°)	$\theta_{r,max}$ (°)
FP	0.045 (0.019)	0.048 (0.020)
Simplified LR	0.021 (0.0086)	0.023 (0.013)
Robust LR	0.021 (0.0087)	0.025 (0.012)

Table 5: Median (standard deviation) values of the peak directional overturning amplification factors for the rectangular structure atop soil column B

Bearing Model	$\alpha_{i,x}$	$\alpha_{i,y}$	$\alpha_{r,x}$	$\alpha_{r,y}$
FP	1.40 (0.38)	1.62 (0.50)	1.71 (0.41)	1.76 (0.54)
Simplified LR	1.21 (0.080)	1.23 (0.18)	1.49 (0.18)	1.54 (0.18)
Robust LR	1.23 (0.14)	1.24 (0.11)	1.52 (0.20)	1.53 (0.18)

CONCLUSIONS

This study utilized ALE3D software to complete explicit, time-domain numerical simulations of seismically-isolated nuclear facility structures subjected to high-amplitude, scaled seismic excitation. The models were comprised of finite element formulations for the soil column, foundation, isolation bearings, and superstructure to simultaneously achieve a variety of research objectives.

Although not exhaustive of all parameter combinations, the analytical simulations employed were used to make general design recommendations for implementing SBI in new NPP structures. In these studies, potentially harmful levels of uplift/tension, torsion, and overturning were observed in the isolated superstructures. These effects were larger in magnitude for the cases where friction bearings were utilized compared to the cases with elastomeric bearings. Therefore, the use of SBI should include extensive analyses to characterize the effect of uplift/tension and consider means to prevent or minimize it. Such prevention devices would reduce overturning and possibly limit torsion as well, since torsion was shown to be largest in cases that also experienced uplift. Additionally, such devices would prevent impact upon re-contact, which may be the cause of high vertical superstructure accelerations observed in this study.

These analyses suggest that SBI can be effective for a range of soil types, however, softer soil sites will likely induce longer period motion, requiring a bearing design with a larger displacement capacity. Thus, although the use of SBI may enable modularization of plant designs, a separate SBI design is required at each site to assure adequate performance objectives and safety margins are maintained. SBI can be considered an effective filter under various soil classes, but a single SBI design is only valid for a narrow range of soil conditions.

Although only two superstructures were examined, peak lateral displacement, uplift, torsion, and overturning rotation response quantities were larger for the cylindrical structure. This suggests that a slenderness effect may exist for base-isolated superstructures. Additionally, the effects of torsion, uplift and slenderness appear to exacerbate one another when certain combinations of soil type, superstructure type, isolator type, and ground motion are analyzed together.

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