Seismic Response of Nuclear Reactor Buildings Incorporating Nonlinear Soil-Structure Interaction

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SUMMARY:
The inelastic behavior of the soil-foundation interface may significantly alter the seismic response of a structure leading to greater energy dissipation, foundation uplift, large foundation deformations, and so on. In this study, the seismic response of an internal shearwall of a nuclear reactor with and without the consideration of these issues is investigated. The effect of soil-structure interaction (SSI) is incorporated using a Beam-on-Nonlinear-Winkler-Foundation (BNWF) model. Various Engineering Demand Parameters (EDP) such as moment, shear force, settlement and foundation rotational demands are evaluated for different foundation compliances. The analysis is carried out within an open-source finite element framework using a large number of ground motions, adopting the incremental dynamic analysis approach. It is observed that consideration of SSI leads to increase in the force demand and reduction in the displacement demand of the shearwall-foundation-soil system.

Keywords: Shearwall, Soil-structure interaction, Incremental dynamic analysis, Seismic response

1. INTRODUCTION

Possibility of structural damage to nuclear reactor structures due to earthquakes constitute a special safety problem, since exposure of the atmosphere or cooling waters to radiation from the fission reactions poses a major health threat to people in the vicinity of the reactor and risk of a biological hazard over a much larger area for several generations (Newmark and Hall, 1969). Due to their low aspect ratio (height/length), reinforced concrete shearwalls, present either in the form of a rectangular box or an individual wall, are used as an intrinsic part of nuclear reactor structures for resisting lateral load due to wind, blasts, earthquakes or waves. A thorough evaluation of the seismic response of shearwall structures is thus indispensable for obtaining a realistic picture of the seismic performance of nuclear power plants.

In general, the seismic performances of shearwalls are characterized by the inelastic flexural or coupled shear-flexural responses of the wall, the nonlinear behavior of the supporting soil-foundation system and SSI. The responses are also affected by the variability in material properties, structural and foundation lateral strength, stiffness, deformation ductility and uncertainty inherent in the seismic ground motions (Tang and Zhang, 2011). The rocking, settlement and translation induced in the foundation under seismic loads may alter the overall dynamic characteristics of the structure including global stiffness, period and spectral demands (Raychowdhury and Hutchinson, 2011) as a result of modified flexibility and damping of the soil foundation structure system (inertial interaction effects). The flexibility of foundations may also modify the ground motion transmitted to the structures (kinematic interaction effects). Displacements to accommodate large rocking and settlement caused by capacity mobilization of the foundation may cause overstress of the structural elements.

Current design practice incorporates linear SSI by increasing the fundamental period and hysteretic damping of the system (ASCE/SEI 7-05, FEMA 368). FEMA 440 (FEMA 440) and ASCE/SEI 7-05 (ASCE/SEI 7-05) recommends a foundation model that uses Winkler springs with elastic stiffness given by Gazetas (1991). Averaging response spectra of motions recorded on soft soil without proper normalization of periods and considering ductility demand in fixed-base structures as a decreasing function of structural period, as suggested by traditional design procedures may also lead to errors (Mylonakis and Gazetas, 2000). A careful consideration of the increase in the fundamental period is required in force based design approaches (Marzban et al., 2011). However, the present design practice is still reluctant to account for the nonlinear SSI, mainly due to the absence of reliable numerical modeling tools.

For structures on soil, the feedback from the structure into the soil and the energy absorption at the structure-foundation interface is considerable, reducing the forces transmitted to the structure (Newmark and Hall, 1969). Foundation rocking and uplifting can produce critical values of base shear for short period structures and slender structures on a relatively soft soil site (Yim and Chopra, 1984). For nonlinear structures, SSI effects due to rocking can result in significantly larger ductility demands under certain conditions of the motion and the structure (Mylonakis and Gazetas, 2000). The SSI effects depend on structure-to-earthquake frequency ratio, foundation-to-structure stiffness ratio, damping coefficient of foundation impedance, foundation rocking and the development of nonlinearity in structures (Zhang and Tang, 2009). Force demands and drift demands (excluding rocking and sliding modes) reduce when nonlinear SSI is considered while the settlement due to foundation yielding is well within the permissible limits suggested by design codes (Raychowdhury and Hutchinson, 2011). These observations necessitate a thorough evaluation of SSI effects on the seismic response of structures using a foundation model that can capture the precise effects of the nonlinear soil-foundation interface behavior.

In a parametric study, Ghersi et al. (2000) observed an increase of the rocking stiffness of the foundation with increasing relative density and decreasing L/B ratio, more so for nonlinear soil conditions. A degradation of the same was noted for low relative densities, high soil deformations and increasing foundation rotation level. Wen et al. (2002) selected a 21-storey shear wall-structure to investigate the effects of site condition and epicentral distance on the seismic response of structures. They observed that probability of damage was higher for softer sites and far field earthquakes. Tang and Zhang (2011) explored the effect of SSI on the damage probability of a mid-rise slender shear wall with a flexible foundation. SSI effect on the maximum inter-story drift was found to be most sensitive to the soil friction angle. Marzban et al. (2011) conducted a nonlinear static analysis to study the behavior of shear wall frames with SSI using the BNWF model. The inter-storey drift, ductility demands and panel shear in the shear wall were found to decrease when foundation yielding is considered. Ductility and period of the structure were found to vary for different site classes. These studies highlight the need to study the effect of variation of ground motions intensities, soil properties and foundation configurations on the response of the shearwall-foundation-soil system.

In this study, the Beam-on-Nonlinear-Winkler-Foundation (BNWF) modeling approach (Raychowdhury and Hutchinson, 2009) is used to address these issues. The response of shearwall structures for fixed base, elastic base and nonlinear base conditions is evaluated by conducting nonlinear time history analysis using a large number of ground motions adopting the incremental dynamic analysis (IDA) approach. Various Engineering Demand Parameters (EDP) such as moment, shear force, drift and ductility demands of the structure and absolute settlement and sliding demands of the foundation are selected to study the effect of inelastic SSI. Of particular interest is the study of the effect of ground motion intensity on the EDPs.

2. NUMERICAL MODELING

The open source, object oriented software (OpenSees, 2008), which provides a finite element platform to numerically model and simulate the response of soil, structures or soil-foundation-structure systems to horizontal loads, cyclic loads and seismic loads is employed in the present study. A concise overview of the shearwall and soil modeling approach, the structural properties, soil properties and the analysis procedure adopted is provided in the following subsections.

2.1. Shearwall modeling

Several numerical models have been developed to model the static and dynamic behavior of reinforced concrete shearwalls (Vulcano et al., 1988, Bolander and Wight, 1991, Ayoub and Filippou, 1998, Kubin et al., 2008, among others). Special continuum elements like shell elements or modified frame elements that can capture the shear-flexure behavior of shearwalls or the hysteretic behavior of reinforced concrete members under cyclic or dynamic loading have been developed (Kubin et al., 2008, Bolander and Wight, 1991). Simple macroscopic elements like fiber beam–column elements and Multiple-Vertical-Line-Element (MVLE) have also been used to model shearwalls (Vulcano et al., 1988). The relative simplicity and computational efficiency of the macroscopic models as compared to the more complex continuum element models make them better suited for practical implementation.

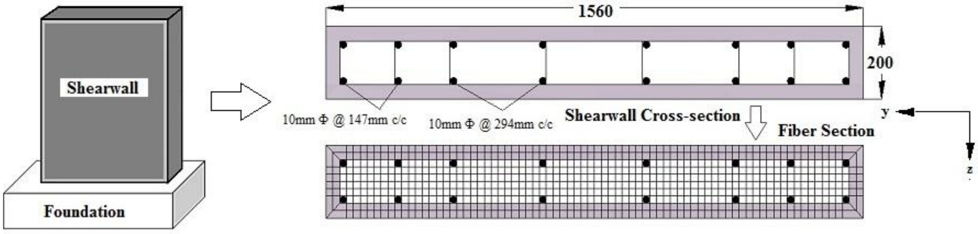

Figure 2.1. Geometric, reinforcement and fiber section modeling details of the shearwall

In this study the force-based distributed-plasticity fiber beam-column element, implemented in the OpenSees platform (OpenSees 2008) is used to model the reinforced concrete shear wall. For the fixed base condition, the foundation is modeled using the same element. The shearwall, its original reinforced concrete section and its fiber discretization done in OpenSees is illustrated in Fig. 2.1. The uniaxial Kent-Scott-Park concrete material model with degraded linear unloading/reloading stiffness and zero tensile strength (*Concrete01* Material in OpenSees) is adopted to represent the concrete. To simulate the steel reinforcement, a uniaxial bilinear steel material model with kinematic hardening and optional isotropic hardening (*Steel01* Material in OpenSees) is used.

2.2. Soil-foundation interface modeling

Soil is a heterogeneous, anisotropic medium with nonlinear force displacement characteristics and complexities added by the presence of a fluctuating water table. Consolidation and varying pressure distribution at the foundation soil interface also influences the soil behavior. Thus an efficient and accurate model with computational validity is required for the modeling of the soil media (Dutta and Roy, 2002). The present study adopts a nonlinear Winkler-based modeling approach to model the soil-foundation interface. The BNWF model (Raychowdhury and Hutchinson, 2009) is selected for its ability to aptly capture soil yielding and degradation (material nonlinearity) and uplifting, gapping and sliding of the foundation (geometric nonlinearity). Variable stiffness and spacing can be assigned to the vertical soil springs thus increasing the versatility of the model. The BNWF model adopted has been validated by comparing the model responses with the results obtained from experiments on square and strip foundations (Raychowdhury and Hutchinson, 2009, 2011 and Raychowdhury, 2008).

The model has been developed within the OpenSees platform (OpenSees, 2008) by assigning *DispBeamColumn* and *zeroLength* elements to the shallow foundation and the soil springs respectively. The soil is modeled by a system of closely spaced, independent, nonlinear springs coupled with dashpot and gap elements. The array of vertical q-z springs distributed along the base of the footing are intended to capture the rocking, uplift and settlement, that is, the vertical and rotational resistances of the footing. The vertical and rocking impedances of the foundation are implicitly accounted for by the differential movement of the q-z springs. In the horizontal direction, the p-x springs capture the passive soil resistance and the t-x springs account for the frictional sliding resistance of the footing. In the BNWF model, the vertical and lateral stiffness are represented by expressions given by Gazetas (Gazetas, 1991). The constitutive relations associated with the q-z, p-x and t-x springs are represented by material models originally developed for piles by Boulanger et al. (Boulanger et al., 1999) and later modified by calibrating the nonlinear backbone curves of the materials against shallow foundation tests. The material models are named *QzSimple2*, *PxSimple1* and *TxSimple1* within OpenSees. A visco-elastic component that represents the 'far-field' behaviour and a plastic, drag and closure component that captures the 'near-field' displacement are present in each of the material models. A gap component accounting for soil-foundation separation is present in the *QzSimple2* and *PySimple1* materials.

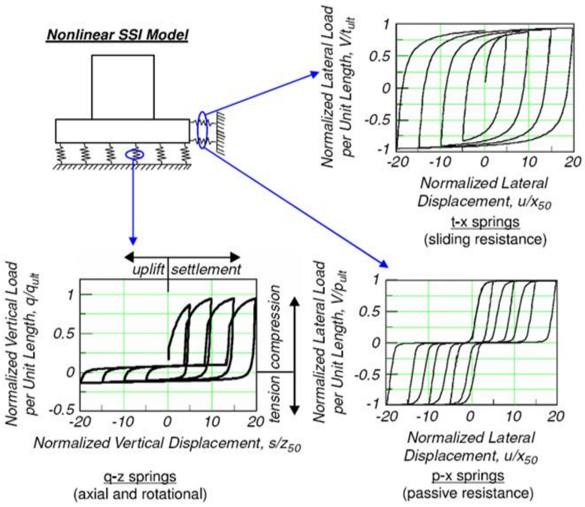

Figure 2.2. Representation of the BNWF model and its material hysteretic responses (Raychowdhury and Hutchinson, 2009)

The *QzSimple2* material has an asymmetric hysteretic response with a backbone curve defined by an ultimate load on the compression side and a reduced strength in tension to represent the soil's low tensile strength. The *PxSimple1* material is characterized by a pinched hysteretic behavior to account for the phenomena of gapping during unloading while the *TxSimple1* material model is characterized by a large initial stiffness and a broad hysteresis response. The distribution of the springs and the material hysteretic responses are shown in Fig. 2.2.

2.3. Structure, foundation and soil details

The modeling and assessment of the reinforced concrete shearwall resting on a shallow square footing is carried out in the software OpenSees using the soil and structural element model described in the preceeding subsections. The shearwall considered is a 1:5 scaled model of an internal shear wall of an Indian nuclear power plant building. A description of this power plant can be found in Reddy et al. (Reddy et al., 1997). The shearwall has an aspect ratio of 1.98 with 0.4% and 0.28% reinforcement in the vertical and horizontal direction respectively. The height of the shearwall model is 3m, width 1.56m and thickness 0.2m. The shearwall was analysed for a combination of gravity and seismic loads as per FEMA-273 (FEMA 1997) and ACI318-89 (ACI 1989). The design and reinforcement detailing was done by following the provisions set by IS456 (IS 2000), IS800 (IS 1984) and IS13920 (IS 1993). In OpenSees, the shearwall is represented by thirty force-based distributed-plasticity fiber nonlinear beam-column elements. Every element has five Gaussian integration points. The wall cross-section shown in Fig. 2.1 is discretized into a mesh of fibers with a thickness of 20mm along the vertical and horizontal direction incorporating the *Concrete01* material in OpenSees. The strength properties of the confined and unconfined concrete are listed in Table 2.1. The top and bottom reinforcement bars are represented by straight layers of fibers made of the *Steel01* material in OpenSees with yield strength of 415MPa. The steel properties used are listed in Table 2.2.

The shallow footing is designed for the soil conditions at the given reactor site. The soil properties at the site are unit weight $\gamma = 24.8 kN/m^3$, friction angle $\Phi = 15^0$, cohesion c = 23.045kPa with corrected

N_{60} value from standard penetration tests ranging from 10 to refusal. For the present study, N_{60} value of 10 is adopted to compute the maximum value of shear modulus (G_{max}) of the soil using the expression given by the Eqn. 2.1 (Kramer, 1996).

$$G_{max} = 325N_{60}^{0.68} \tag{2.1}$$

The foundation considered is a 2m by 2m square footing with a depth of 0.4m and an embedment of 1m. Vertical and sliding stiffness are selected based on recommendations given by Gazetas (Gazetas, 1991) and vertical load bearing capacity is calculated after Terzaghi (Terzaghi, 1943) using foundation shape, depth and inclination factors proposed by Meyerhof (Meyerhof, 1963). The static vertical factor of safety, FS_v is calculated based on the previously mentioned bearing capacity and shape factor expressions and is found to be 5.3 for the structure considered. The BNWF model is established with the soil properties, foundation characteristics, and mesh details given in Table 2.3. The soil is represented by q-z springs at a spacing of 2% of the footing length in the vertical direction and one p-x and t-x spring each in the horizontal direction.

Table 2.1. Concrete material model and parameters used in numerical analysis

Material	OpenSees model	Model parameters			
		Compressive strength at 28 days (MPa)	Strain at maximum strength	Crushing strength (MPa)	Strain at crushing strength
Confined concrete	Concrete01	39	0.005	35.25	0.020
Unconfined concrete	Concrete01	30	0.002	0.000	0.005

Table 2.2. Reinforcement material model and parameters used in numerical analysis

Material	OpenSees model	Model parameters		
		Yield strength (MPa)	Young's modulus (GPa)	Strain-hardening ratio
Reinforcement bars	Steel01	415	200	0.03

Table 2.3. Parameters for the BNWF model

	Property	Value
Soil properties	Soil Type	Sand
	Cohesion, c (KPa)	23.045
	Friction angle, Φ (deg)	15
	Unit weight, γ (KN/m^3)	24.8
	Shear modulus, G (KPa)	74×10^3
	Poisson's ratio, ν	0.4
	Radiation damping, c_{rad}	0.05
	Tension capacity, T_p	0.1
Footing properties	Length, L_f (m)	2.0
	Width, B_f (m)	2.0
	Height, H_f (m)	0.4
	Embedment, D_f (m)	1.0
	Young's modulus, E_f (KPa)	21.5×10^6
	Pressure at foundation base due to the weight of superstructure, W_g (KPa)	31.25
	Angle of load application w.r.t the vertical axis, β (deg)	0
Mesh properties	Ratio of stiffness intensity in end region to that in middle region, R_k	2.0
	Ratio of the end region length to L, R_e	0.1
	Ratio of the spring spacing in the middle region to L, S_e	0.2

The shearwall is analyzed for fixed, elastic and nonlinear base conditions. The EDPs obtained from the linear and nonlinear base conditions are compared to the ones obtained from the fixed base case to evaluate the effect of SSI. For the fixed base condition the nodes of the foundation are fixed in all degrees of freedom. For the elastic base condition, the spring force-displacement relationships are considered to be linearly elastic. Sliding is allowed for this condition. Incremental dynamic analysis is performed on the system considered using a large number of ground motions. For the seismic response analysis, five percent Rayleigh damping is assumed for the first two modes of vibration of the model. Newmark's method is used to conduct the transient analysis, with solution parameters $\beta = 0.25$ and $\gamma = 0.5$. To solve the nonlinear equilibrium equations, the modified Newton algorithm is used with a convergence tolerance of 1e−8.

3. RESULTS AND DISCUSSIONS

A gravity load was imposed on the shearwall before the dynamic earthquake analysis was conducted. This is the load that the internal shearwall is subjected to due to the other structural components of the nuclear reactor. Gravity analysis was conducted before any other analysis to account for the effect of settlement of the flexible based shearwall due to gravity. In the subsections that follow, a description of the ground motions selected for the time history analysis, the methodology of the IDA procedure and the analysis results with their interpretations are presented.

3.1. Seismic simulation

A total of 30 ground motions were selected for nonlinear seismic response analysis, 10 from Indian earthquakes selected from the Cosmos database (COSMOS) and 20 from international earthquakes selected from the PEER NGA database (PEER Ground Motion Database). Spectral acceleration at the fundamental time period of the fixed base structure (S_aT_1) is adopted as the intensity measure (IM) for earthquake motions. This IM is selected since it depends on both structural properties (i.e. T_1) and ground motion characteristics (Kurama and Farrow, 2003). The transient analysis is conducted by the incremental dynamic analysis procedure (Vamvatsikos and Cornell, 2002). In an IDA, a selected set of ground motions are scaled to represent increasing intensities and applied as the input motion to the structure-foundation-soil model. The EDPs for each intensity level are studied.

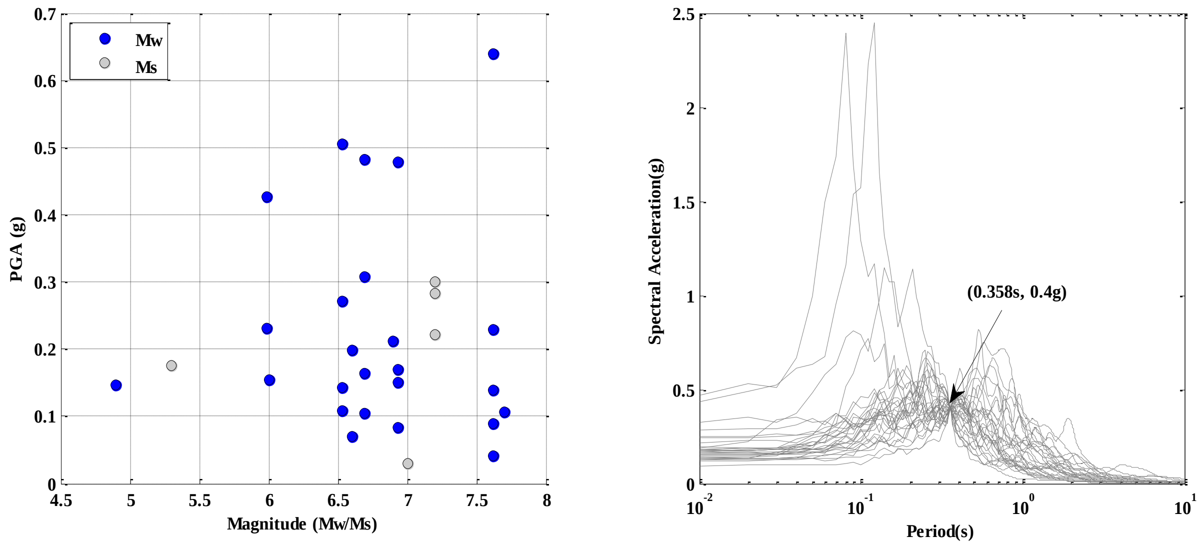

Figure 3.1. Information about selected earthquakes: magnitude vs. PGA of unscaled motions (left) and acceleration response spectrum for motions scaled to $S_aT_1 = 0.4$g for $T_1 = 0.357$sec (right)

The fundamental period for the fixed base shearwall is 0.358s (T_1). The scaling is done such that for each ground motion, the value of spectral acceleration at this time period (i.e. S_aT_1) increases from 0.1g to 1.0g with an increment step of 0.1g. Thus 10 simulations are performed for each of the 30 motions for the fixed, linear and nonlinear base conditions. Fig. 3.1 shows the distribution of earthquake magnitude (M_s/M_w) with peak ground acceleration (PGA) for the 30 unscaled motions selected (left) and the elastic, 5% damped acceleration response spectrum for the motions scaled to S_aT_1 of 0.4g (right).

3.2. Results and interpretation

Various force and displacement demands of the structure and the foundation play a key role in their performance based design. To study the effect of SSI on the shearwall-foundation-soil system, the average of these demand parameters are correlated to the earthquake intensity measure in scatter-plots. The Engineering Demand Parameters (EDP_s) selected for the numerical study are listed in Table 3.1., in which V_{max} = Peak absolute base shear, W = weight of the shearwall, M_{max} = Peak absolute base moment, u_{roof} = Peak roof displacement, H = Height of the shearwall, Δ_y = Displacement at yield point for fixed base structure (from nonlinear static analysis), S_{max} = Peak absolute foundation settlement, ΔS = Differential settlement between two ends of the foundation, u_{fmax} = Peak absolute foundation sliding.

Table 3.1. List of EDP_s selected for the numerical study

EDP_s	Description	Symbol
EDP_1	Normalized peak base shear	$\lvert V_{max}\rvert/W$
EDP_2	Normalized peak base moment	$\lvert M_{max}\rvert/(WB_f)$
EDP_3	Peak roof drift ratio	$\lvert u_{roof}\rvert/H$
EDP_4	Global ductility	$\lvert u_{roof}\rvert/\Delta_y$
EDP_5	Normalized peak settlement	$\lvert S_{max}\rvert/B_f$
EDP_6	Normalized peak foundation sliding	$\lvert u_{fmax}\rvert/B_f$

Code based design requires the evaluation of force based demands of a structure. Fig. 3.2 shows the distribution of the average peak base shear and average peak base moment, i.e. EDP_1 and EDP_2 with the ground motion intensity. Each point represents the average peak values of the demand parameters obtained from analysis using 30 ground motions that have been scaled to a particular value of the IM. It is observed that EDP_1 and EDP_2 reduce on the introduction of foundation flexibility, this reduction being more for the nonlinear base condition. Both these parameters show a clear increase with increase in the intensity of the ground motion. The reduction in base shear demand due to mobilization of the foundation and surrounding soil is enhanced for high intensity motions. For instance, at SaT_1 of 0.6g, a 37.6% reduction in EDP_1 occurs for the nonlinear base as compared to the fixed base, this reduction being 22.7% for the linear base whereas at SaT_1 of 1g, these reductions are 62.2% and 34.4% respectively. From Fig. 3.2., it can be concluded that in case of the base moment demand, these reductions slightly increase with higher IM values. The base shear and base moment for the fixed base configuration acts as the upper envelope for the force demands pertaining to all other foundation configurations. Thus when SSI is considered, reduced force demands will prove to make the structure economic. Consideration of nonlinear SSI is required for an accurate estimate of these demands.

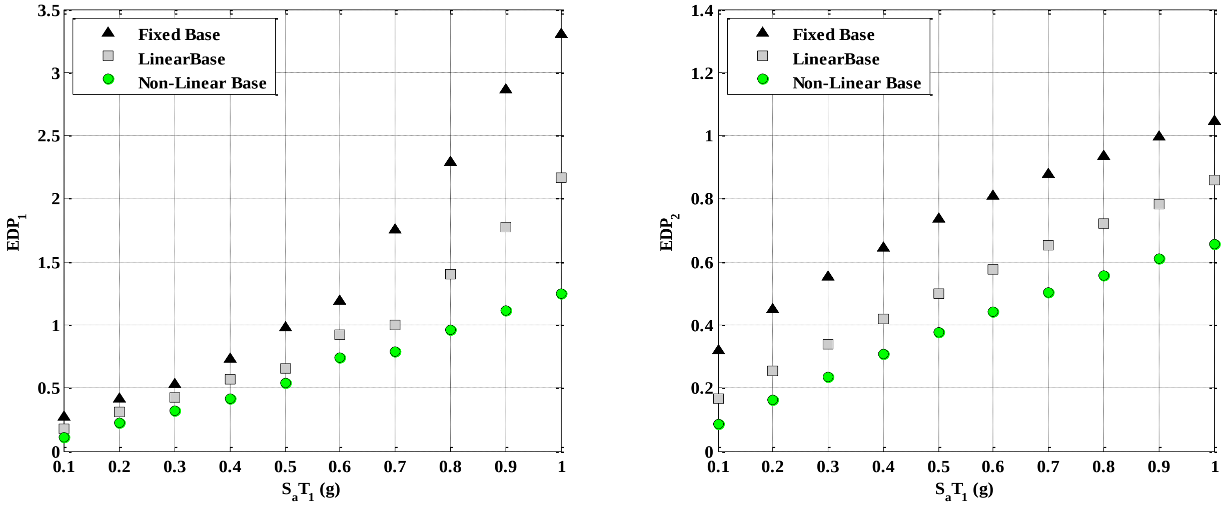

Figure 3.2. Average EDP_1 (left) and average EDP_2 (right) against the incremental intensity levels

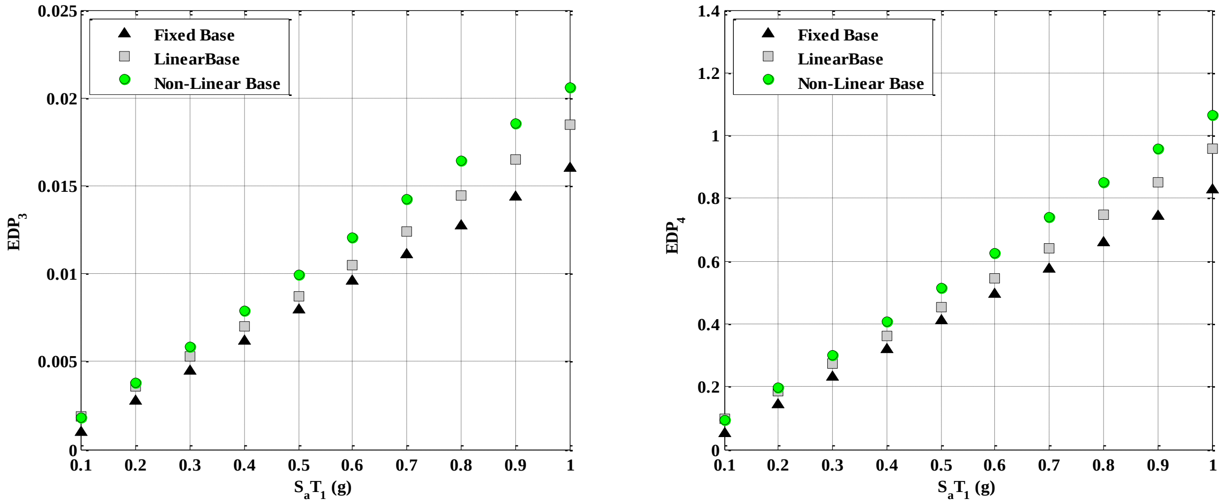

Figure 3.3. Average EDP_3 (left) and average EDP_4 (right) against the incremental intensity levels

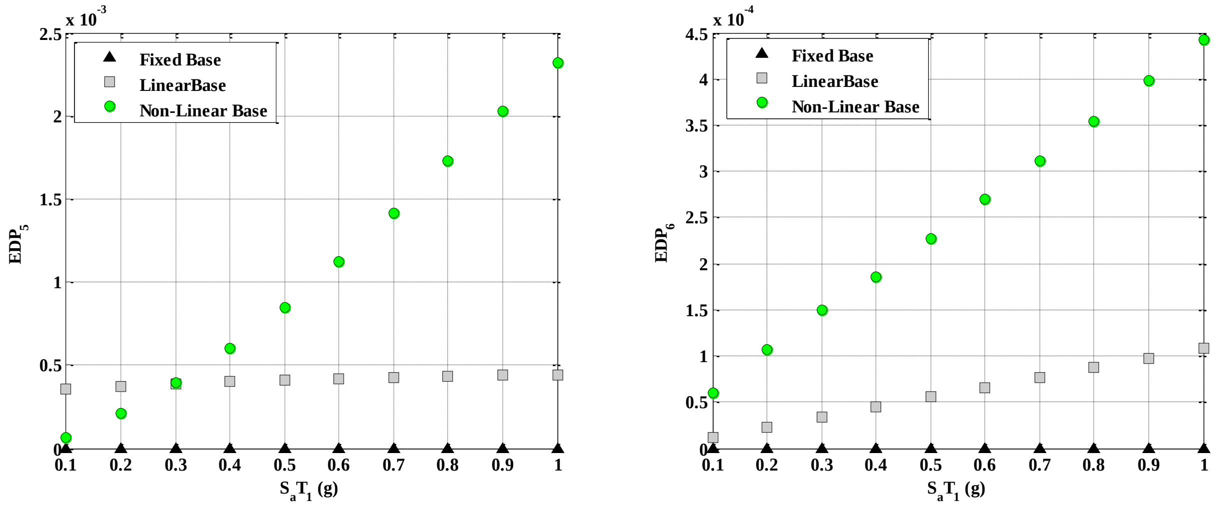

Figure 3.4. Average EDP_5 (left) and average EDP_6 (right) against the incremental intensity levels

Yielding of the foundation can increase the displacements in the superstructure to dangerous levels. The average peak displacement demands of the shearwall, EDP_3 and EDP_4, i.e. the roof drift and global ductility demand respectively are plotted against the selected earthquake IM in Fig. 3.3. Both these demands are observed to be maximum when plastic deformation of the soil is considered, being lesser for the elastic base and least for the fixed base case. Larger roof drift and global ductility are observed with increase in the earthquake intensity as expected. At lower intensity levels the drift and ductility demands are almost comparable for all the base conditions but as the intensity rises, a divergence in the demand trends is noted as the difference in these demands for the different foundation compliance cases increase. Care should thus be taken while considering SSI in engineering design, since it can induce large structural deformations, which if unchecked could lead to considerable damage for ground motions of high intensity.

To obtain a comprehensive view of SSI effects on the shearwall-foundation soil system, in addition to the force and displacement demands of the superstructure, the foundation demands need to be studied. The settlement and sliding of the foundation are evaluated at each earthquake intensity level. In Fig. 3.4 the variation of EDP_5 and EDP_6 are plotted against the selected IM. The average peak settlement (EDP_5) for the nonlinear base is less than that of the linear one for very low intensity earthquakes ($S_aT_1 \leq 0.2g$) whereas as earthquake intensity increases, EDP_5 increases sharply for the nonlinear base condition, remaining more or less constant for the elastic base. This can be explained by the concept that nonlinearity in the soil gets mobilized for earthquakes above a threshold intensity value. As intensity increases, the degree of soil yielding increases, thus inducing a rapid rise in the value of the peak settlement. Linearly elastic foundations greatly underestimate the peak settlement of the shearwall-foundation-soil system, except for very low intensity earthquakes. The average peak foundation sliding (EDP_6) remains maximum for the nonlinear base for the entire range of ground motion intensities as can be observed from Fig. 3.4. For the elastic base, a small but steady rise of foundation sliding is recorded whereas for the inelastic base, a steep rise is noted with incrementing earthquake intensity. For instance, the foundation sliding at SaT_1 of 1g is 3.2 times of its value at SaT_1 of 0.3g for the nonlinear base. As the intensity of ground motions increase, the disparity in the value of foundation sliding for the linear and nonlinear base also increases. This occurs due to the plastic yielding of the soil. Consideration of nonlinear SSI is thus mandatory for safe design of the structure. The settlement and sliding induced in the foundation should be kept within permissible limits for the structure to meet its required performance level.

4. CONCLUSIONS

Sensitive structures like nuclear reactors can cause immense hazard on failure under seismic loads, thus necessitating an analysis procedure that accounts for SSI, varying foundation compliances and ground motions of a wide intensity range. In the present study, the seismic response of an internal shearwall of a nuclear power plant is evaluated in terms of several force and displacement demand parameters using the IDA procedure. The soil-foundation interface is realistically modeled using a distributed array of mechanistic inelastic springs, dashpots, and gap elements. Marked reductions in the base shear and base moment demands and increase in the drift and ductility of the structure are observed when SSI is considered, being more for the nonlinear base condition. The foundation sliding and settlement are found to increase sharply for the nonlinear base condition with increasing ground motion intensity. These detrimental effects of SSI should be carefully monitored for safe design of structures. Issues like the effect of uncertainty in the BNWF model parameters on the shearwall response, the influence of SSI on the damage probability of the system and the interaction of the shearwall with the other components of the reactor structure warrants further investigation.

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