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Probabilistic Seismic Response of Soil-Pile Foundation-Structure System: A Substructure Based Analysis

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Abstract. Seismic soil structure interaction (SSI) analysis helps to evaluate collective response of whole structural system. SSI modelling under seismic loading seems to be the prime factor which has a significant influence on accurate prediction of seismic response of structure. Though it was well evidenced that direct modelling approach renders solution with higher degree of accuracy but this approach was not popular due to computational cost and inefficiency. In addition, variability in subsoil properties, modelling and load uncertainty has led the problem further complex. In this context, present study is attempted to carry out probabilistic seismic analysis of pile foundation supported building structure embedded in in-homogenous soft clay modelled through computationally efficient and simplified substructure based modelling approach. First, monotonic pushover analysis is performed by applying lateral load on pile head which is modelled using beams on nonlinear Winkler foundation (BNWF) approach. The stiffness of equivalent pile-soil impedance springs are calculated from this analysis and, frequency dependent dynamic stiffness of the foundation is also obtained following an accepted analytical model. Finally, seismic response of the whole structure idealized as single degree of freedom (SDOF) oscillator is calculated incorporating equivalent impedance soil springs at the base with application of input earthquake motions. Monte Carlo simulation (MCS) is adopted to perform the probabilistic analysis incorporating material (i.e. soil properties). The outcome of the study helps to assess the influence of variability of system parameters on seismic response of pile supported structure in a relatively simplified manner.

Keywords: Substructure based analysis; SPSI system; Pushover analysis; Material uncertainty; Monte Carlo simulation.

1 Introduction

Seismic soil structure interaction (SSI) analysis offers collective response of whole structural system. SSI modelling under seismic loading seems to be a prime factor which has a significant influence on accurate prediction of the seismic response of the structure. Though it was well evidenced that direct modelling approach renders solution with higher degree of accuracy but this approach was not popular due to computational cost and inefficiency. While, sub-structure approach has an advantage towards SSI modelling and computational efficiency in order to predict the seismic response of structure. However, this approach was found to be limited to linear behaviour of soil-foundation system. On the other hand, several other factors such as, variability in subsoil properties, modelling and load uncertainty has led the problem further complex. In this context, present study is attempted to carry out probabilistic seismic analysis of pile foundation supported building structure embedded in in-homogenous soft clay modelled through computationally efficient and simplified substructure based modelling approach. Monotonic pushover analysis is performed on soil-pile group foundation system which is modelled using beams on nonlinear Winkler foundation (BNWF) approach and subsequently stiffness of equivalent pile-soil impedance springs are calculated. Then frequency dependent dynamic stiffness of the foundation is also obtained by the help of NIST 2012 guidelines. At last, seismic response of the superstructure idealized as SDOF oscillator is calculated incorporating equivalent impedance springs with application of six IS 1893 (Part-1) 2016 spectrum consistent artificial ground motions and two real ground motions at base and 5 % of critical damping (dashpot) which is reasonable for concrete structures. Monte Carlo simulation (MCS) is adopted to perform the probabilistic analysis incorporating material (i.e. soil properties) and load variability. The outcome of the study helps to assess the influence of variability of system parameters on seismic response of pile supported structure in a relatively simplified manner.

2 Statement of the Problem

A representative structure with fundamental period of 0.6 sec resemble a 6 storied reinforcement cement concrete building supported on pile foundation consisting of 2 ×3 bays with plan area 22.5 m × 15 m and elevation 22.5 m situated in very soft clay soil medium is considered in the present study. The size of each bay is 7.5 m×7.5 m. The total load is calculated by considering the live load of 7.2 kN/m^2 acting at each floor along with dead load. The superstructure weight is calculated as 15364.35 kN and the load acting on each central column is calculated as 1097.45 kN. Hence, the present study idealizes a 2 × 2 pile group of size 4 m × 4 m, thickness of the pile cap 0.5 m, length of the pile 18 m, diameter of the pile 0.8 m with *L/d* ratio of 22.5 and spacing to diameter i.e. *s/d* ratio of 2.5 under one of the central column by the help of BNWF SSI approach and developed an identical substructure based SSI model for the same pile group.

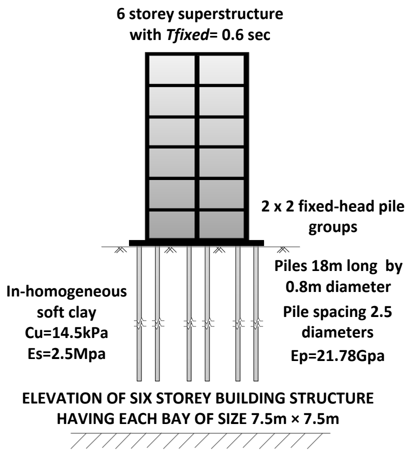

Fig. 1: Prototype structure.

3 Idealization and Modelling of Structural system

3.1 Superstructure-pile foundation-soil modelling

Beams on nonlinear Winkler foundation (BNWF) model as adopted elsewhere are used to model the dynamic pile soil interaction behaviour. Modelling of pile is performed by using elastic beam column element. In fact, as per modern design codes, piles are not designed to remain linear elastic, especially under the MCE earthquake level. Hence, nonlinear pile behaviour may be considered as future scope of study. Based on a convergence study, pile is discretized into 20 elements and the discredited length of each pile element is 0.9 m. The embedded pile length is 18 m. 6 translational degrees of freedom are considered at each node of the pile. The mass acting on the central column is presumed to be lumped at the top of free head of the column. The equivalent soil springs are attached at each node of the pile in all translational directions (equal degree of freedom) which incorporates pile-soil interaction in all three directions. The pile is assumed to be embedded in in-homogeneous very soft clayey soil medium. The nonlinear spring behaviour is modelled using dynamic nonlinear $p{-}y$, $t{-}z$ and $q{-}z$ curves for clay.

Table 1. Properties of Pile and Soil.

Pile data	Soil data (Clay)
Diameter, *d* (m), 0.80	Soil consistency, Very soft
Length, *L* (m), 18.00	Un-drained cohesion, *Cu* (kN/m^2), 14.50
Young's modulus, *Ep* (kN/m^2), 21.78×10^6	Saturated density, γ_{sat} (kN/m^3), 17.00
Poisson's ratio, 0.17	Young's modulus, *Es* (kN/m^2), 2500.00
	Poisson's ratio, 0.40

3.2 Proposed soft clay soil modelled

In many studies, the design of pile foundation is given by considering the homogeneous soil medium (that means soil strength parameter does not vary with depth i.e. constant) beneath the structure which is unrealistic for soft clay as compared to stiff clay. Gazetas (1984) proposed three soil models in his study of seismic response of end bearing single piles, these are $E = Es \times (z/D)$ [linearly varying in-homogeneous soil medium], $E = Es \times \sqrt{(z/D)}$ [parabolic in-homogeneous soil medium], $E = Es$ [homogeneous soil medium] where, Es = Young's modulus of soil, z = depth of the soil medium from the ground surface, D = diameter of the pile and shown the comparison between them. On the other hand, Budhu and Davis (1988) showed that field data suggest a linear relation between un-drained shear strength (Cu) and vertical effective stress (σ_z) for very soft (normally consolidated) clays as $Cu = 0.3 \times \sigma_Z$. For many practical cases, it is observed that results are much more correlated by considering the Gazetas (1984) linearly varying in-homogeneous soil medium. So, in this study, we consider linearly varying in-homogeneous soil medium proposed by Gazetas (1984) as $E = Es \times (z/D)$ and considering linear relationship among Cu and Es, we proposed linearly varying in-homogeneous soil profile for clay as $Cu \times (z/D)$ which is more realistic for actual situation. Also show the comparison between homogeneous soil medium, linearly varying in-homogeneous soil medium of Gazetas (1984) and Budhu and Davis (1988).

3.3 Pile-soil interaction model in clay

The spring properties are modelled by nonlinear *p-y* behaviour under cyclic loading proposed by American Petroleum Institute (API 2005).The ultimate resistance for nonlinear spring in case of very soft clay is found out using the following parameters,

$$Pu = 3C + \lambda H + J \times (CH/D) \quad for \;\; H < H_R \tag{1}$$

$$Pu = 9C \quad for \;\; H \geq H_R \tag{2}$$

$$H_R = 6D/\{(\lambda D/\{(\; + J\} \tag{3}$$

Where P_u = ultimate resistance in (lbs/in or kN/m), C = un-drained shear strength (lbs/in^2 or kN/m^2), λ= Effective soil weight in (Ib/in.^{3}or kN/m^3), H = depth in (m), H_R = depth below soil surface to bottom of reduced resistance zone in (m), J = 0.25-0.5 (dimensionless empirical constant) and D = average pile diameter from surface to depth in (m).However, the final ultimate resistance in lbs or kN for clayey soil is calculated as follows,

$$Pu \; final = (Pu \times element\,length) \tag{4}$$

Further, in case of clay the displacement relationship is presented for 50% of ultimate resistance as follows;

$$Y_{50} = 2.5\varepsilon_{50} D \tag{5}$$

$$\varepsilon_{50} = \left(\frac{0.1796}{log_{10}^{Cu}}\right)^{(1/1.1841)} \tag{6}$$

Where Y_{50} parameter is expressed as a function of un-drained cohesion (Cu) based on the suggested experimental relationship between ε_{50} and Cu as presented by (Evans et al. 1982) D = average pile diameter from surface to depth in (m).

Furthermore, many seismic codes (e.g. Indian seismic design guideline, IS 1893-Part -I 2002) suggest 5 % of critical damping is reasonable for concrete structures. Therefore, to strike a balance between rigour and accuracy, 5% of critical damping in each mode of vibration of pile-soil and superstructure system is considered for deterministic analysis regardless of structural support condition.

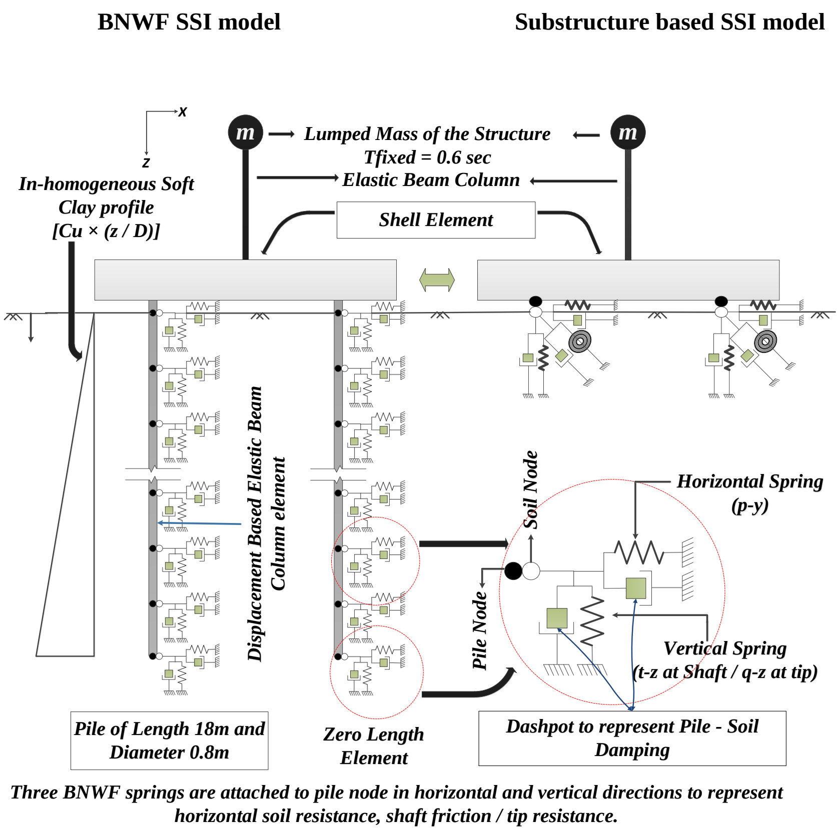

Fig. 2. Idealized of soil-pile foundation structure BNWF model and substructure based SSI model.

3.4 Nonlinear pushover analysis of pile foundation

By performing monotonic pushover analysis, generates different static-pushover curves which plot different strength-based parameters against deformations. Here, strength-based parameters are lateral load, vertical load, and rocking moment and corresponding deformations are lateral displacement, vertical settlement and rotation, respectively. From these curves, also generates three different stiffness curves against deformation, these are lateral stiffness, vertical stiffness and rocking stiffness curve.

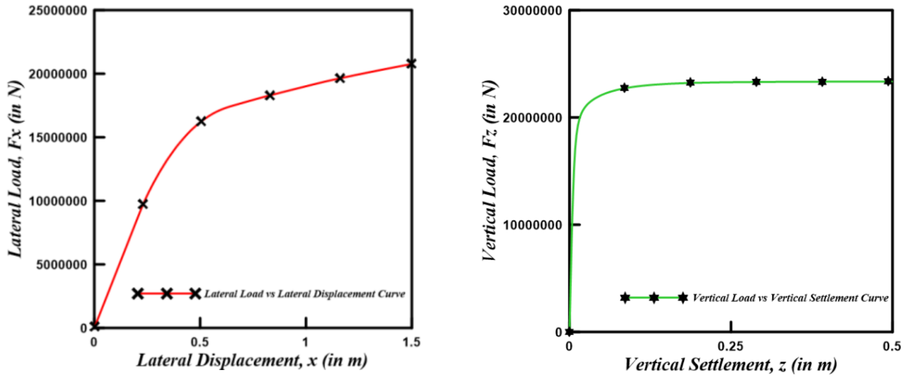

Fig. 3. Static-pushover curves for Lateral load vs. displacement and Vertical load vs. settlement.

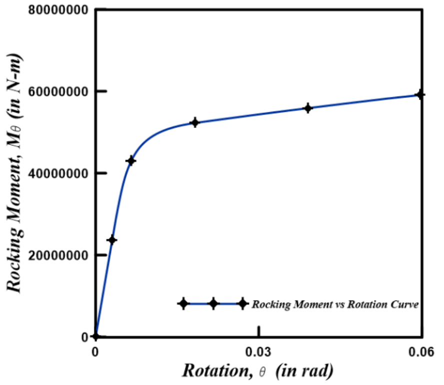

Fig. 4. Static-pushover curves for Rocking moment vs. rotation.

Table 2. Dynamic stiffness modifier (α_j^p) for different modes of vibration by NIST 2012.

Dimensionless frequency	Dynamic stiffness modifier		
	α_X^p	α_Z^p	α_θ^p
$a_{o,ssi}^p$	0.98606886	1.192217825	1.00 (assumed)
$a_{o,soil}^p$	0.998511756	1.117703207	1.00 (assumed)

Table 3. Dynamic stiffness of group piles for different modes of vibration by NIST 2012.

Dimensionless frequency	Dynamic stiffness		
	$K_X^{Dynamic}$	$K_Z^{Dynamic}$	$K_\theta^{Dynamic}$
$a_{o,ssi}^{p}$	41720891.21	3280679160	7747000000
$a_{o,soil}^{p}$	42247354.15	3075633950	7747000000

3.5 Uncertainty modelling and probabilistic analysis

Probabilistic analysis is performed using Monte Carlo simulation (MCS) technique. The Probability distribution function, PDF is taken as Log-normally distributed function. The range of variation is selected based on the suggested range for *COV* of undrained cohesion (*Cu*), (10%–50%) as proposed by Phoon and Kulhawy (1999). The soil parameters used in deterministic analysis is considered as mean parameter in case of variability modelling.

The log-normally distributed random field is given by,

$$Cu(\tilde{x}_i) = exp\{\mu_{\ln Cu}(\tilde{x}_i) + \sigma_{\ln Cu}(x).G_i(\tilde{x})\} \tag{7}$$

Where *Cui* are the i^{th} realization of *Cu* and *Gi* is a zero mean, unit variance Gaussian random number, $\mu_{\ln Cu}$ and $\sigma_{\ln Cu}$ are described as,

$$\sigma^2_{lnCu} = ln(1 + \frac{\sigma^2_{Cu}}{\mu^2_{Cu}}) = ln(1 + COV^2_{Cu}) \tag{8}$$

$$\mu_{\ln Cu} = lnCu - 0.5\sigma^2_{lnCu} \tag{9}$$

Where, $COV_{Cu} = \sigma_{Cu} / \mu_{Cu}$ is the coefficient of variation of *Cu* and μ_{Cu} and σ_{Cu} are the sample mean and standard deviation of *Cu*, respectively. Present study considers COV_{Cu} as 10%, 30% and 50%. It is to be noted that the stiffness of soil springs are estimated from the realizations of *Cu*. It is assumed that stiffness of the soil springs follow log-normal distribution due to linear relationship among spring stiffness, *Cu*.

3.6 Ground motions

Six IS 1893 (Part-1) 2016 spectrum consistent artificial ground motions and two real ground motion are applied to the substructure based model in both deterministic and probabilistic SSI analysis to obtain moment, shear force and displacement at various location of the SSI model. The artificial ground motions are generated by SeismoArtif 2018 software. These six ground motions are selected by taking 0.13 times, 0.22 times, 0.32 times, 0.34 times, 0.50 times and 0.54 times of the maximum PGA (Peak Ground Acceleration) of the IS spectra. Hence, the six different artificial ground motions are generated of PGA 0.325g, 0.56g, 0.785g, 0.84g, 1.25g and 1.36g covering low to high range of ground motion. Also two different real ground motions are applied. These are recorded when earthquake occurred at Loma Prieta, California (1989) [recorded station: Anderson Dam (L Abut)] and Imperial Valley, Mexico (1979) [recorded station: Cerro Prieto].

3.7 Dynamic method of analysis

Nonlinear dynamic analysis is performed to calculate the elastic and inelastic response of the structure and foundation under applied ground motion loading. In built algorithm available in OPENSees (version 3.2.0) is used to carry out the analysis. The solution of equation of motion of the system for the analysis is done by using Newmark's β-γ step by step integration method. It involves consideration of constant average acceleration over each incremental time step. Modified Newton-Raphson method is employed to perform the iteration for each incremental step. Newmark's parameters are chosen as $\beta = 0.25$ and $\gamma = 0.5$ are adopted. This ensures unconditional stability. 1/5- 1/20 (0.001s) of the ground motion time step is taken as the time step of integration, which has been found to be suitable from a convergence study. The accuracy and correctness of the programs used has been validated in other cases and are available in literatures. (E.g. Chopra 2008, Clough and Penzien 1995).

4 Results and Discussion

Results obtained from deterministic and probabilistic analysis performed by Monte Carlo Simulation (MCS) for different ground motions are presented in this study. Also, the effect of *in-situ* soil variability of soil design parameters on the dynamic characteristics, nonlinear load deformation behaviour of pile and soil under lateral and dynamic loading (*p-y* behaviour) and response at various location of the pile foundation of different period of structures are primarily investigated in this chapter. The results presented for clayey soil will help to give a broad conclusion on the effect of *in-situ* variability on seismic design of structure supported by pile foundation in non-liquefiable clay (soft) soil.

4.1 Probabilistic seismic response of SSI structure

A convergence study is performed to determine the number of sample realizations for MCS analysis. For instance, 300 realizations of soil springs stiffness are calculated considering 300 numbers of log-normally distributed randomly generated *Cu* values obtained from the above equations. Realizations of fundamental time period of the substructure based SSI system are obtained for different number of realizations of spring stiffness values considering COV_{Cu} 50%, T_{fixed}= 0.60sec and very soft clay (Ep/Es = 8712, L/d = 22.5 and S/d = 2.5) for $a^{p}_{o,ssi}$ and for $a^{p}_{o,soil}$ frequency respectively.

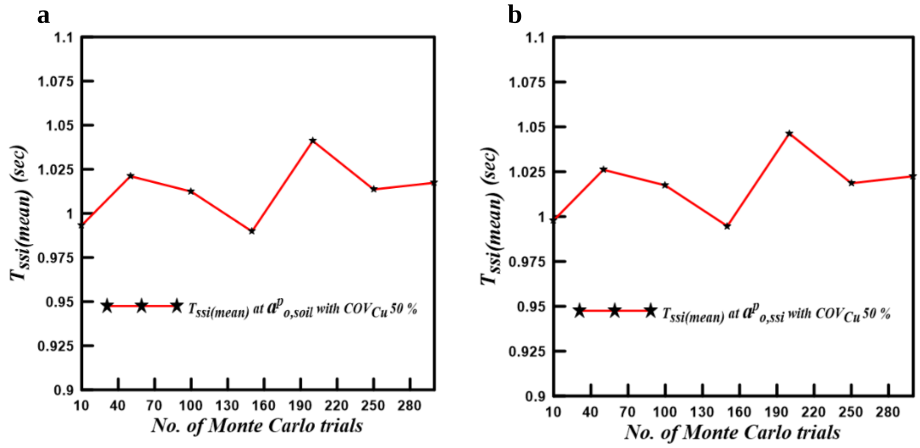

Fig. 5. Variation of time period of the substructure based SSI model with respect to the number of MCS trials T_{fixed} = 0.6s: (a) for $a^{p}_{o,soil}$ and (b) for $a^{p}_{o,ssi}$ with COV_{Cu} 50%, respectively.

The mean and standard deviation of the fundamental time period of the entire system (T_{ssi}) is presented as a function of number of Monte Carlo simulations. It indicates that the fluctuation of (T_{ssi}) is marginal for number of trials beyond 280. Hence, a choice of 300 numbers of iterations is acceptable and adopted in this study for further analysis. However, 300 number of MCS may not valid for nonlinear pile–soil system which may be considered as limitation of present study.

4.2 Influence of shear strength variability of soil on fundamental period of structure

It is observed that the probabilistic normalized period T_{ssi} *(mean)*/T_{ssi} *(det)* is varying with in a range of 1.0043-1.0908 (i.e., minimum to maximum range of variation of 0.43% to 9.08% w.r.t deterministic results) with respect to COV_{Cu} of 10% to 50% in case of very soft clayey soil. However, it is observed that the probabilistic normalized period T_{ssi} *(mean)*/T_{fixed} is varying within a range of 1.53-1.67 (i.e., minimum to maximum range of variation of 53% to 67% w.r.t fixed base results) with respect to COV_{Cu} of 10% to 50% in case of very soft clayey soil. It is also observed that the probabilistic normalized period T_{ssi} *(det)*/T_{fixed} is varying within a range of 1.51-1.53 (i.e., minimum to maximum range of variation of 51% to 61% w.r.t fixed base results) with respect to COV_{Cu} of 10% to 50% in case of very soft clayey soil. Hence, it is shown that by limiting the design (considering fixed base, not incorporating SSI and *in-situ* variability effect) of the structure, we actually eliminate time period effect which is more detrimental towards the structure. However, almost similar effects are observed for both the frequencies.

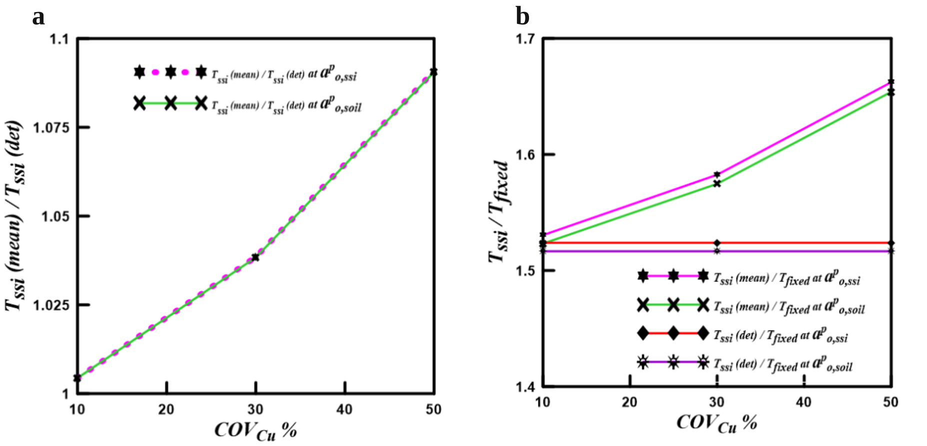

Fig. 6. Variation of the probabilistic normalized time period of the substructure based SSI model with respect to COV_{Cu} of 10%, 30%, 50% in case of very soft clayey soil: a)T_{ssi} *(mean)*/T_{ssi} *(det)* w.r.t deterministic analysis and b)T_{ssi} *(mean)*/T_{fixed} and T_{ssi} *(det)*/T_{fixed} w.r.t fixed base analysis for $a^p_{o,soil}$ frequency and for $a^p_{o,ssi}$ frequency.

4.3 Influence of shear strength variability of soil on shear at pile

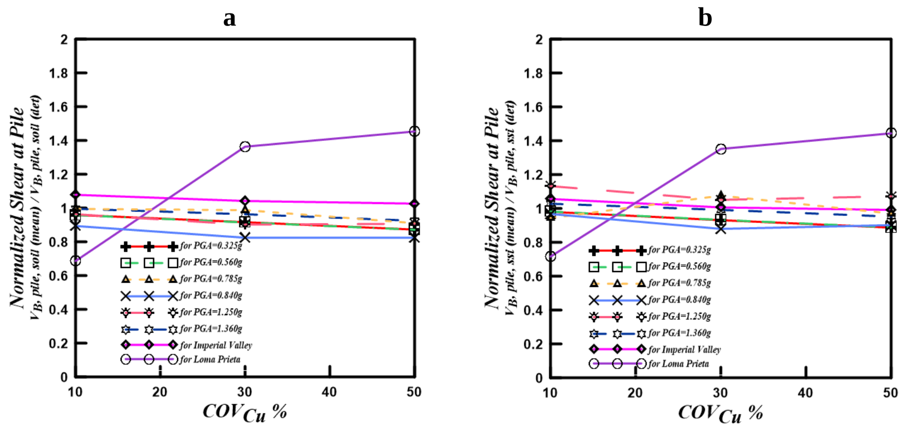

Fig. 7. Variation of the probabilistic normalized shear force at pile with respect to COV_{Cu} of 10%, 30%, 50% in case of very soft clayey soil: a) for $a^p_{o,soil}$ frequency and b) for $a^p_{o,ssi}$ frequency.

An identical response due to influence of shear strength variability of soil on shear force is observed between pile and superstructure. Here, also the results for $a^p_{o,soil}$ frequency and for $a^p_{o,ssi}$ frequency are almost same. Also the variation of responses of different ground motions and for different frequencies with respect to the COV_{Cu} are in between 0-20%. However, the probabilistic analysis of substructure based SSI model for Loma Prieta earthquake shows tremendous increase of shear force of about 40-60% for COV_{Cu} 30-50%. This issue need to be studied in details with substructure based SSI model for more ground motions.

5 Summary and Conclusions

Summarily, in this present study, the fundamental time period estimated using two different modelling techniques (BNWF approach and substructure based approach) idealising different various soil profiles for validating proposed substructure model. Then show a wide range of variation in normalized time period of pile supported structure due to incorporation of *in-situ* variability of soil parameters and modelling uncertainty which is idealized by consideration of certain range of *COV* of *Cu* with respect to the mean *Cu* used in deterministic analysis. Furthermore, the effect of *in-situ* variability of soil on different force and deformation parameters are also studied for different frequencies which also indicates that SSI modelling uncertainty may alter the design response of structure as well as pile foundation. These issues need to be studied in detail by considering a well-accepted statistical modelling of variability of soil, dynamic loading and SSI modelling uncertainty. However, this limited study shows the importance of reliability based design for piled supported heavy structures with an emphasis to carry out a detailed study in this direction.

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