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Seismic codes based equivalent nonlinear and stochastic soil structure interaction analysis

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Abstract: The aim of this study is to consider the effects of the variation of shear modulus ratio (G/G_0) and damping ratio (ξ) of soil, obtained by a linear iterative method based on the design spectra of seismic codes, the soil environment in terms of uncertainties in shear modulus using Monte Carlo simulations and the foundation damping (ξ_f) of flexible base for analyses of the Soil-Structure Interaction (SSI) problems. A squat structure with circular shallow foundation resting on a soil layer over a homogeneous half-space is studied by using cone model and considering seismic zone effect on structural response. Firstly, after showing the effects of the correction of G and ξ on impedance functions and the responses of soil-foundation-structure system, a study is carried out to compare these effects to those of the modelling of uncertainties in shear modulus as random variations. Secondly, a comparative analysis on design response spectra and base shear forces was carried out for four seismic codes (Algerian Seismic Rules RPA99-2003, Eurocode 8-2004, International Building Code IBC-2015 and Indian Code IS-1893-2002) considering the three cases of SSI: SSI effects (initial G and ξ), nonlinear SSI (corrected G and ξ) and stochastic SSI (random G with COV = 20%) compared to the fixed base case. Results show that the correction of G and ξ, according to the equivalent nonlinear method in all the cases, leads to a remarkable decrease in peak responses but show a huge amount of reduction in the second study for IBC-2015 and IS-1893-2002 codes compared to the other codes.

Keywords: Shear modulus; damping; uncertainties; linear iterative method; seismic code.

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1 Introduction

Observations based on the effects of previous strong earthquakes on structural damage have concluded that the seismic response of many engineering structures could be affected by deformability of the underlying foundation. Currently, structures become higher and larger and their seismic response may be affected by the medium on which they are founded. In addition, the response of a structure under seismic excitation is affected by the interaction between three connected structural parts: the structure, the foundation and the soil underlying and surrounding the foundation. Furthermore, the interaction between soil, foundation and structure sub-systems cannot be disregarded because the problem becomes increasingly important especially for structures placed on relatively soft soil in seismically active areas. The effects of soil-structure interaction (SSI) on the seismic response of structural systems are significant when the shear wave velocity of the supporting soil is less than 600 m/s.[1–3] Hence, the effect of SSI should be taken into consideration in order to ensure SSI safety and economic efficiency of the structure during an earthquake.[3]

On the other hand, the capacity of the soil to support various kinds of loadings depends on the strength and stability of the supporting soil materials. Consequently, not only the behaviour and responses of soils and foundations are important, but how soils and foundations affect seismic wave propagations and the subsequent loading transferred to main structures are also of great interest.[4]

Various methods have been developed to implement SSI during the last fifty years and the problems can be solved by two main methods: the direct method and the substructure method. Current design codes provide guidelines to include its effects in seismic design of structure[5–7] analysing it by the substructure method.

Furthermore, the soil exhibits spatial heterogeneities, which are among the main sources of structural damage and disfunctioning of the built systems.[8] This heterogeneity or variability resulting from its deposition history is in many cases imperfectly known and large errors

might appear during the identification of the parameters of the soil model. So, it is necessary to use probabilistic approaches to provide reliable design of the structure. In fact, the spatial variability of soils is a major source of SSI problems and the dynamic SSI problem should be modelled into a stochastic from word.[9–11] However, to what extent can uncertainty in soil parameters affect the structure response to address the SSI problem?

The response of a structure to a seismic excitation might increase or decrease depending on the characteristics of the ground motion and dynamic properties of the structure and the underlying soil.[3,12] Under earthquake excitation, the strain would invariably be larger than that measured during tests where the soil non-linearity plays an important role in the dynamic response of structures and soils, since each seismic record has different levels of shear stress in soil deposits.[13] Even for a moderate earthquake, the strain range increases; degradation in soil stiffness becomes significant and has a major contribution to the overall response. It is obvious that strain induced in soil will depend upon dynamic loading, geological conditions of a site, stress history of soil and a number of other factors.[4]

Widely, in order to evaluate the seismic demand of structures, it is assumed that the structure is founded on a rigid soil. However, real soil profiles are featured by complex shapes and nonlinear behaviour. Adequate information on the dynamic soil properties, in particular dynamic shear modulus and damping coefficient, is essential for accurate calculation of soil response and better analysis of SSI problems. Both shear modulus and damping coefficient are the most important material properties to characterize the dynamic behaviour of soils[14] and both of these properties are affected by shear strain amplitude, effective stress level, void ratio and number of loading cycles in the case of clean sands. However, in the case of clay, they are affected by the number of loading cycles, over-consolidation ratio and plasticity index.[15] Consequently, the shear modulus and damping coefficient of a soil medium exhibiting nonlinear behaviour should be sufficiently reflected in SSI analysis.[2,3]

Over and above, in the dynamic analysis of SSI, soils are represented by linear, equivalent linear or nonlinear models. In linear analysis, the shear modulus and damping ratio are assumed constant. However, the relationship between cyclic secant shear modulus and cyclic shear strain amplitude is widely used in design practice to evaluate the seismic response and site effects through subsurface soils.[16] According to EC8-2004,[5] if the ground acceleration is equal to/or greater than 0.1 g, shear modulus must be multiplied by an average reduction factor. So, to what extent can correction in soil parameters (G and ξ) due to seismic event affect the structure response to address the SSI problem?

To answer the questions raised above, a nonlinear and stochastic analysis of SSI effects for shallow foundations is carried out in the present study. The main emphasis is on the effects of the correction of shear modulus and damping ratio values due to the nonlinear soil behaviour under seismic excitation on the response of the SSI system in terms of impedance functions and structure displacements. The soil under the structure is considered homogenous and is modelled as a discrete model based on the concept of cone model. The model cannot only give the impedance of the soil but can also model the soil responses.[10,11] Furthermore, the response spectrum being a very important tool for seismic analysis or structural design and providing a very handy tool for engineers to quantify the demands of earthquake ground motion on the capacity of buildings to resist earthquakes, a comparative analysis is carried out on design response spectra and base shear forces for four seismic codes (Algerian Seismic Rules (RPA99 -2003),[17] Eurocode 8 (EC8-2004),[5] International Building Code (IBC-2015)[18] and the Indian Seismic Code (IS-1893-2002)[19] considering three cases: the initial case of damping ratio or fixed base, the case of uncertain shear modulus of soil and the case of total foundation damping for SSI (before and after the correction of shear modulus and damping ratio).

2 Nonlinear and stochastic formulation of SSI problem

2.1 Simulation of the nonlinear behaviour

The nonlinear stress-strain behaviour of a soil can be expressed in two ways, with an increase in the shear strain amplitude: (i) degradation of shear modulus G and (ii) an increase in the damping coefficient ξ.

The soil nonlinearity can be taken into account by many models.[20–25] In the equivalent linear method, the nonlinear stress-strain behaviour of a soil is approximated by a secant stiffness G_{sec} and an equivalent damping ξ_{eq} that are compatible with the strain in the soil induced by the ground shaking. Okada et al.[26] studied the seismic pile response of a structure-pile-soil system to evaluate the strain-dependent nonlinearity of the ground using an equivalent linearization method.

In the present study, the shear modulus and damping coefficient are modified using a linearized iterative method

based on design response spectra (Sa) for four seismic codes (RPA99-2003,[17] EC8-2004,[5] IBC-2015[18] and IS-1893-2002[19]). All the steps of the method are summarized in **Appendix 1** according to Chowdhury and Dasgupta.[27]

2.2 Stochastic based formulation

The soil properties can be delineated using deterministic or probabilistic model. They greatly depend on the soil deposition conditions and loading history, and may be derived from a common set of in situ or laboratory test data.[28,29] In other words, soil is nonhomogeneous and its formation is an outcome of a variety of random geological processes,[30] and consequently, the stochasticity of soil properties remains largely unknown. Several methods are used to treat stochastic problems where Monte-Carlo simulation (MCS)[31,10] provide a robust conceptually simple way to account rationally for various uncertainties.[32–34] The most used probability distribution function to estimate uncertainty in geotechnical properties for performance based earthquake engineering are uniform, normal, lognormal, gamma and exponential.[35] The lognormal distribution is commonly used to model certain types of data that appear in several fields of engineering. Since this includes most, if not all, engineering (mechanical) systems, particular properties of the lognormal random variable (such as non-negativeness and skewness) and of the lognormal hazard function (which increases initially and then decreases) make lognormal distribution a suitable fit for some engineering data sets. However, the lognormal distribution can have widespread application; a generalized lognormal distribution can be used to provide better fits for many types of experimental or observational data.[36]

On other hand, structural engineering design is replete with uncertainties, some of which are obvious and some of which many engineers may never have considered as stated by Bulleit.[37] Otherwise, most shallow foundations supporting structures are generally not circular in shape. So equivalent circular foundations, where foundation radii are computed separately for translational (r_a) and rotational (r_m) deformation modes,[38] are used.

In addition, the effective height of the structure (h) shall be taken as 0.7 times the structure height for multi-story structures, although in reality, there are irregular structures in plan and/or elevation (Fig. 1a). Thus, the equivalent foundation radius and the effective height of the structure may be assumed subject to uncertainties and may be considered as random variables.[11]

In this study, the effects of soil nonlinearity by correction of the shear modulus and the damping ratio together with soil stochasticity are considered. Several thousands of random draws are generated for soil according to the lognormal distribution with a coefficient of variation (COV) for G taken as 20% according to the literature.[39–42]

The lognormal distribution is written as follows:

$$f(x)=\begin{cases}\dfrac{1}{x.\sqrt{2.\pi.\sigma_{\ln.x}{}^2}}\exp\left(-\dfrac{1}{2.\sigma_{\ln.x}{}^2}(\log x-\mu_{\ln.x})^2\right), & for.0.\langle x\langle\infty\\ 0 & otherwise\end{cases} \quad (1\text{-a})$$

Mean and variance are given as:

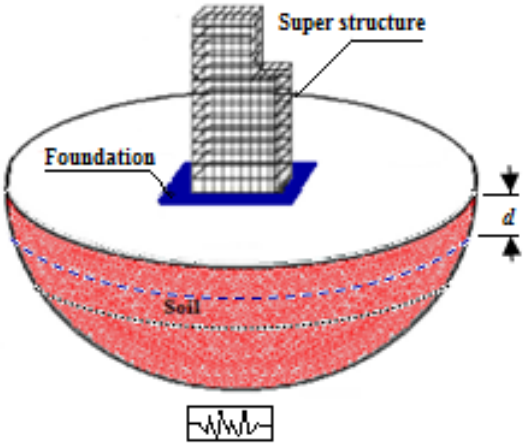
$$E(X)=\exp\left(\mu_{\ln.x}+0.5.\sigma_{\ln.x}{}^2\right),$$
$$V(X)=\exp\left(2\mu_{\ln.x}+2.\sigma_{\ln.x}{}^2\right)-\exp\left(2\mu_{\ln.x}+\sigma_{\ln.x}{}^2\right) \quad (1\text{-b})$$

Where, $\mu_{ln,x}$ and $\sigma_{ln,x}$ are the lognormal mean value and standard deviation, respectively.

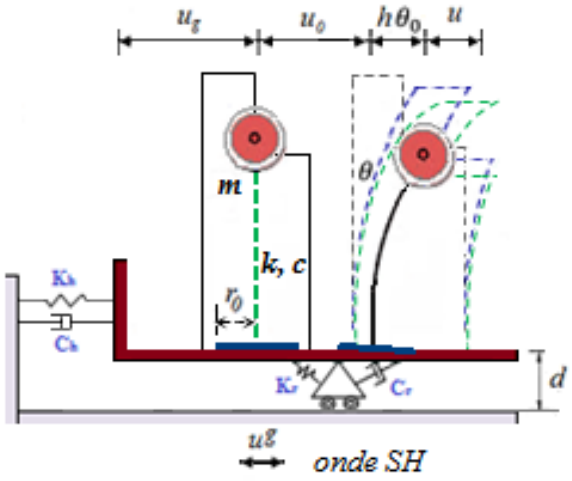
3 Modelling of the SSI problem

The schematic view of a SSI system and the equivalent system of single degree of freedom (SDOF) are shown in Figs. 1a and 1b, respectively. The superstructure is attached to the ground by springs-dampers elements, which are[43] able to move in two directions to give translational and rotational motions. The structure is described by its stiffness *k*, mass *m*, equivalent height *h* and damping *c* (Fig. 1b). The SSI problem is solved in the frequency domain using the Cone model, which is a simple model for foundation vibration analysis based on the wave propagation.[11,44] Also, cone model is well suited for evaluating the impedance functions.[42,45–47]

SSI has both kinematic and inertia interaction effects on a structure, but for shallow foundation, only the inertia effect is considered[10] to calculate the dynamic response of a structure supported on a shallow foundation. The soil-foundation interface is modelled by discrete-elements including horizontal and rocking equivalent linear springs and viscous dashpots with frequency-independent coefficients. The parameters of this interface element (impedance functions) are obtained by the simple physical cone model. The cone model is simple one-dimensional model for foundation vibration analysis, which consists of replacing the soil deposit, for



(a)



(b)

Figure 1: Modelling of SSI: (a) schematic view of SSI system, (b) equivalent SDOF system.

each degree of freedom of the foundation by a truncated semi-infinite elastic cone .The cone model was originally developed by Ehlers (1942) to represent a surface disk under translational motions, and since that time, the model shows multi uses and development in dynamic soil structure interaction problems.[41–43,45–47] The concept of cone model has been rearranged and extended by Wolf[48–52] to include a complete range of dynamic excitations and physical situations.[53]

Most of the published results using cone model are confined to the determination of the dynamic response of the foundation in the form of complex impedance functions, which represent the dynamic stiffness of the soil medium surrounding the foundation. The real part of the impedance function, denoted dynamic stiffness, reflects the stiffness and inertia of the supporting soil, and its dependency on frequency is solely attributed to the

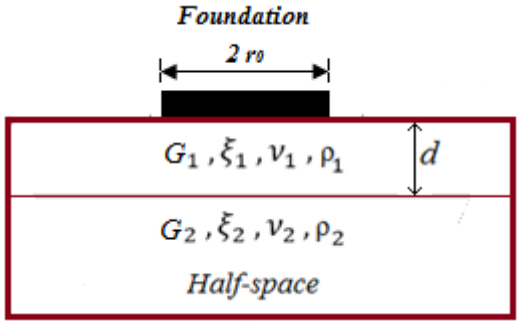


Figure 2: Model of a rigid circular footing on a soil layer underlain by elastic half-space.

influence of frequency on inertia. While, the imaginary part represents the energy dissipation in the system generated as a result of the wave propagation away from the foundation (radiation damping).

To study the impedance function of a shallow foundation resting on the surface of a soil layer underlain by a half-space, the foundation was replaced by a rigid massless foundation of radius r_0 as illustrate in Fig. 2. The layer with depth d has a shear modulus G_1, Poisson's ratio ν_1 , mass density ρ_1 and hysteretic damping ratio ξ_1. The corresponding parameters of the half-space are G_2, ν_2, ρ_2, and ξ_2.

The impedance functions for both horizontal and rocking motions can be written in terms of the dimensionless frequency ($a_0 = \omega r_0 / \nu_s$, where: ν_s is the shear wave velocity of soil) as:

$$K(a_0) = K\left[k(a_0) + ia_0.c(a_0)\right] \tag{2-a}$$

where the static stiffness K (K_h for horizontal motion and K_r for rocking motion), are as follows:

$$K_h = \frac{8.G_1 r_0}{2-\upsilon_1}, \qquad K_r = \frac{8.G_1 r_0^3}{3(1-\upsilon_1)} \tag{2-b}$$

$k(a_0)$ and $c(a_0)$ are the dimensionless dynamic stiffness and damping factors.

The total horizontal foundation displacement (u_0^t) and the total displacement of the SSI system (u^t) (Fig. 1(b)) are expressed, respectively, by

$$u_0^t = u^g + u_0 \tag{3-a}$$

$$u^t = u^g + u_0 + h\,\theta_0 + u \tag{3-b}$$

Where, u_g is the ground displacement.

The displacements of the coupled system are:[10,11,44]

$$\begin{cases} u_0(\omega) = \dfrac{\omega_s{}^2.(1+2\xi i)}{S_{\xi_x}.m}u(\omega) \\ h\theta(\omega) = \dfrac{\omega_s{}^2.(1+2\xi i)}{S_{\theta_x}.mh^2}u(\omega) \\ (1+2\xi i - \dfrac{\omega^2}{\omega_s{}^2} - \dfrac{\omega^2.(1+2\xi i)}{S_{\xi_x}.m} - \dfrac{\omega^2.(1+2\xi i)}{S_{\theta_x}.mh^2})u(\omega) = \dfrac{\omega^2}{\omega_s{}^2}.u^g(\omega) \end{cases} \quad (3\text{-c})$$

Where $S_{\xi_x}(\omega)$ and $S_{\theta_x}(\omega)$ are horizontal and rocking components of the impedance functions, respectively. ω/ω_s is the frequency ratio and $\boldsymbol{i}$ is the complex number.

A FORTRAN computer program is realized in order to assess the effects of the stochasticity (shear modulus G with COV =20%) and the nonlinear behaviour of soil (corrected shear modulus and damping ratio) on the foundation and superstructure responses.

The steps followed to realize the program are shown in **Appendix II**.

4 Results and Discussions

The correction of dynamic soil characteristics (G and ξ) is performed based on an iteration method,[27] which uses response spectra as explained in **Appendix I**.

The mean values of soil parameters before the correction are selected such that G_1/G_2 = 0.544 and the damping ratio $\xi_1 = \xi_2$ = 5%. The other parameters are mass density ratio ρ_1/ρ_2 = 0.85, Poisson's ratio $\nu_1 = \nu_2$ = 0.25 and depth to radius ratio d/r_0 = 1. Two values of the coefficient of variation (COV) for shear modulus parameters are assumed here (10% and 20%). Then, by using the linear iterative method[27] and RPA99-2003[17] code design spectrum, the soil characteristics (G_1, ξ_1) are corrected according to **Appendix I** and subsequently: G_1/G_2 = 0.309 and ξ_1/ξ_2 = 1.32.

4.1 Effects of the correction of shear modulus and damping coefficient on SSI response

The impedance functions for horizontal and rotational motions are determined for a circular foundation of radius r_0 on the surface of a soil layer resting on a half-space.

The results in terms of impedance functions (written in terms of real and imaginary parts versus the dimensionless frequency) for horizontal and rotational vibration modes are shown in Fig. 3. This figure shows the influence of the correction of shear modulus and damping coefficient on the real and imaginary part of impedance functions (spring and damping coefficients). It appears from this figure that the correction of the shear modulus and the damping coefficient due to the imposed seismic motion is very remarkable. Both real and imaginary parts of impedance functions changing as G and ξ values are corrected.

The change is apparent for all the frequencies from 0 to 8. This result is consistent with the results of other researchers like those of Lutes et al.,[39] when they used a non-classical modal-analysis-based formulation to quantify the variation of stochastic response in an SSI system with both uncertain soil properties in the soil–foundation system and uncertain structural properties in the superstructure on structural response for seismically excited soil-structure interacting (SSI) systems, those of Mittal et al.,[54] when carrying out a parametric study on the effect of seismic zone on structural response of tall chimney by incorporating strain dependent shear modulus and determining the effect of variability in soil type and those of Zafarkhan and Dehkordi[55] while presenting the effect of variability in soil type (different shear modulus) and structure height on soil-structure system responses.

The displacements of the structure u (ω) and the mass with respect to the free field (u_0 (ω) + $h\theta_0$ (ω) + u (ω)) and the total displacement of the base ($u_g(\omega) + u_0(\omega)$) are plotted versus the excitation frequency normalized by that of the structure ω/ω_s in Fig 4. The displacements before and after the correction of G and ξ are compared.

The displacement relative to the free field of the structure shows a decrease of about 33%, while the mass displacement and the total displacement of the base increase in absolute values by about 4% and 6%, respectively. But on average, all the displacements show the same dimensionless frequency shift of 18%. Fattah et al.[56] studied the effect of sand density and whether the sand is dry or saturated on its dynamic response for the foundation-soil system. They found that for dry and saturated conditions, the maximum amplitude of displacement decreases with increasing the relative density of sand and contact area of footing and increase with increasing the amplitude of loading. The maximum displacement amplitude response of the foundation resting on dry sand models is more than that on the saturated sand by about 5.0–10%.

This variation explains that the structure becomes more rigid and the soil more flexible and shows the interest of taking into account the nonlinear soil behaviour in SSI analysis. The figure clearly shows that

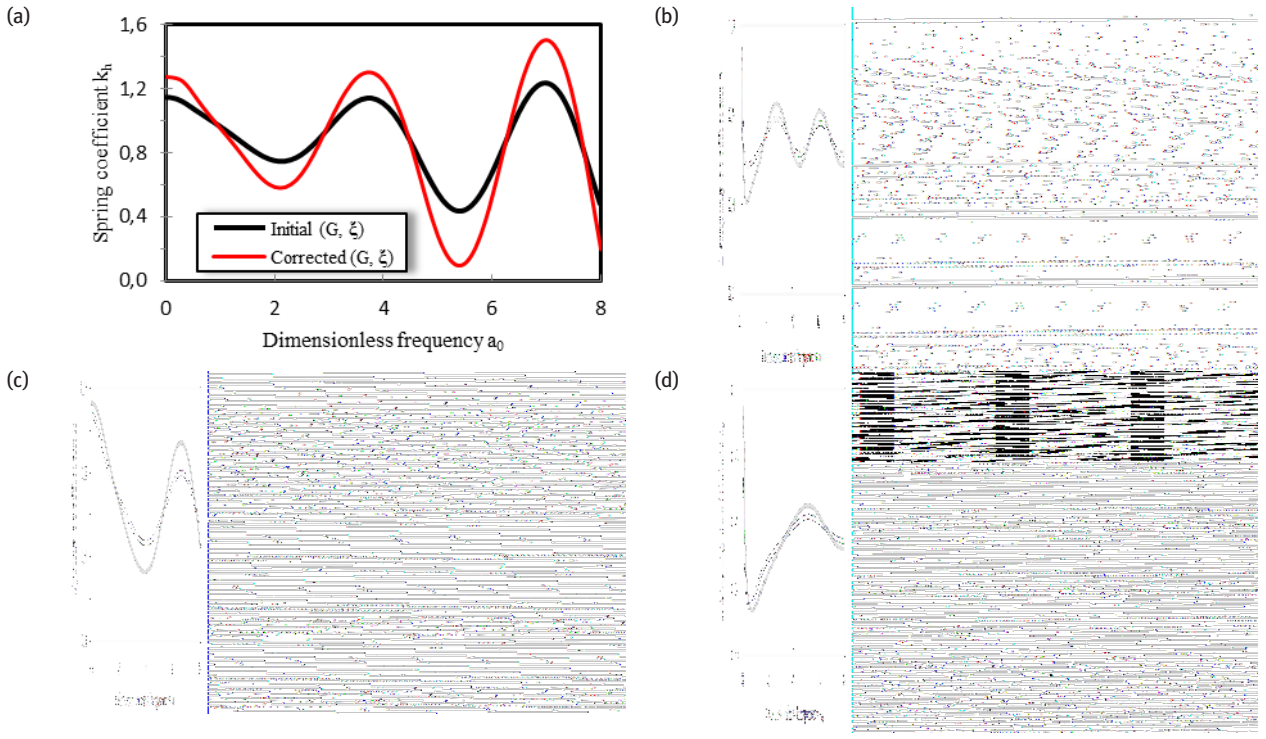


Figure 3: Impedance functions obtained before and after the correction of shear modulus and damping coefficient: (a) and (b) for horizontal motion, (c) and (d) for rotational motion.

SSI alters the frequency content of the structure response. Also, the figure reported herein shows that SSI is more significant when soil nonlinearity case will take place during earthquake compared to the linear case (G and ξ are constants), where the nonlinearity reduces the stiffness of the soil (reduce shear modulus) and increases the hysteretic damping. Similar observation has been made by Li et al.[57] when studying the effect of different soil properties on structure response, and Farghaly and Ahmed[2] in the case of seismic loading. Fattah et al.[58] found that the displacement amplitude of vibration response of foundation system placed on the surface of dry dense sand models is less than that of dry loose sand models; according to the authors, these results are attributed to the increase in the stiffness and the modulus of elasticity of dense sandy soil that makes the soil stiffer and resist vibrations. In 2017, Fattah et al.[59] found that the amplitude of displacement of a single pile embedded in saturated sandy soil due to vertical vibration is more than that of the dry soil, and Fattah et al.[60] explained that this trend is due to the increase in the pore water pressure during dynamic load that causes reduction in the inter-particle forces between the solid particles of the soil skeleton, hence causing an increase in displacement response.

4.2 Effects of stochastic shear modulus and correction of soil characteristics on the SSI response

In this subsection, the stochastic and the nonlinear effects are considered by means of uncertainty of G and correction of *G* and *ξ*, respectively, on the response of the SSI system. A lognormal distribution is used to generate random numbers of shear modulus using Monte Carlo Simulations for coefficient of variation of 20%.

Fig. 5 displays the stochastic as well as the nonlinear displacements of the structure, the mass and the base due to the random variations of shear modulus and the correction of shear modulus and damping, respectively. Fig. 5 shows that the random variations of *G* greatly affect the response of the structure. It is observed that 20% COV of *G* reduces the peak structure displacement (16.6%) by half that is reduced by the correction of *G* and *ξ* (33.1%).

Additionally, the frequency is shifted about 18% due to the correction of *G* and *ξ*, which means that the soil becomes more rigid. However, Fig. 5b indicates that the peak value of mass displacement is reduced by 17.4% due to 20% COV of G compared to the nonlinear as well as to the deterministic ones (initial G). For the displacement of the base (Fig. 5c), for random variations of the shear

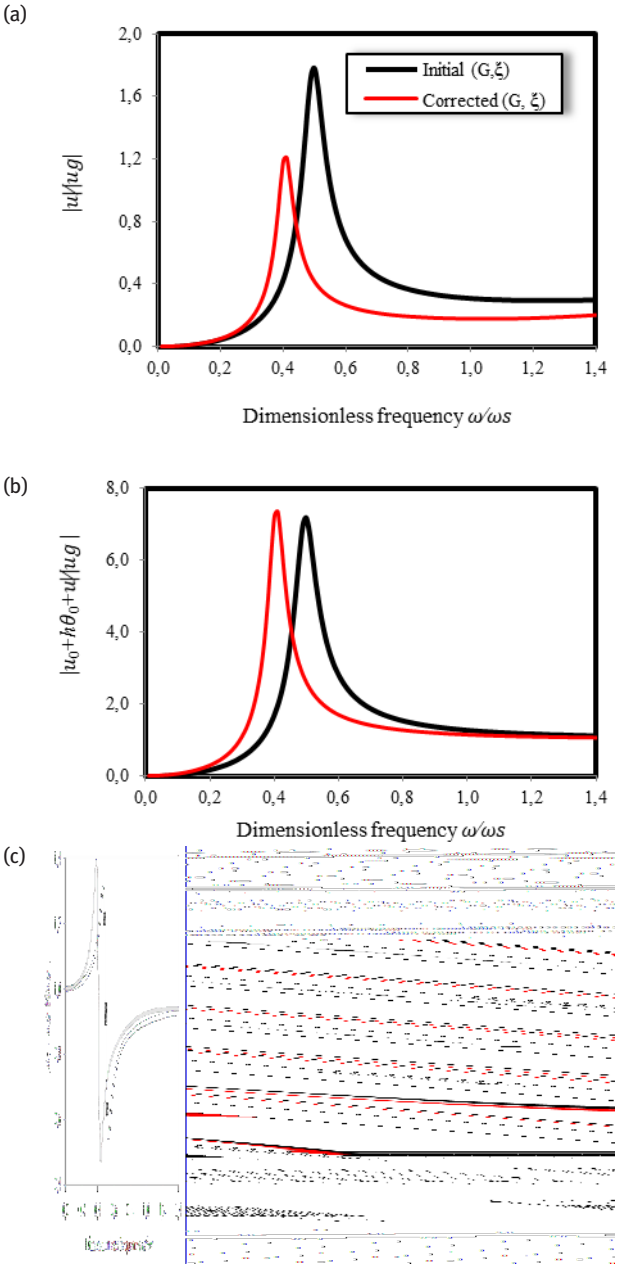


Figure 4: SSI response with respect to the free field before and after the correction of shear modulus and damping coefficients: (a) displacement of the structure, (b) displacement of the mass and (c) total displacement of the base.

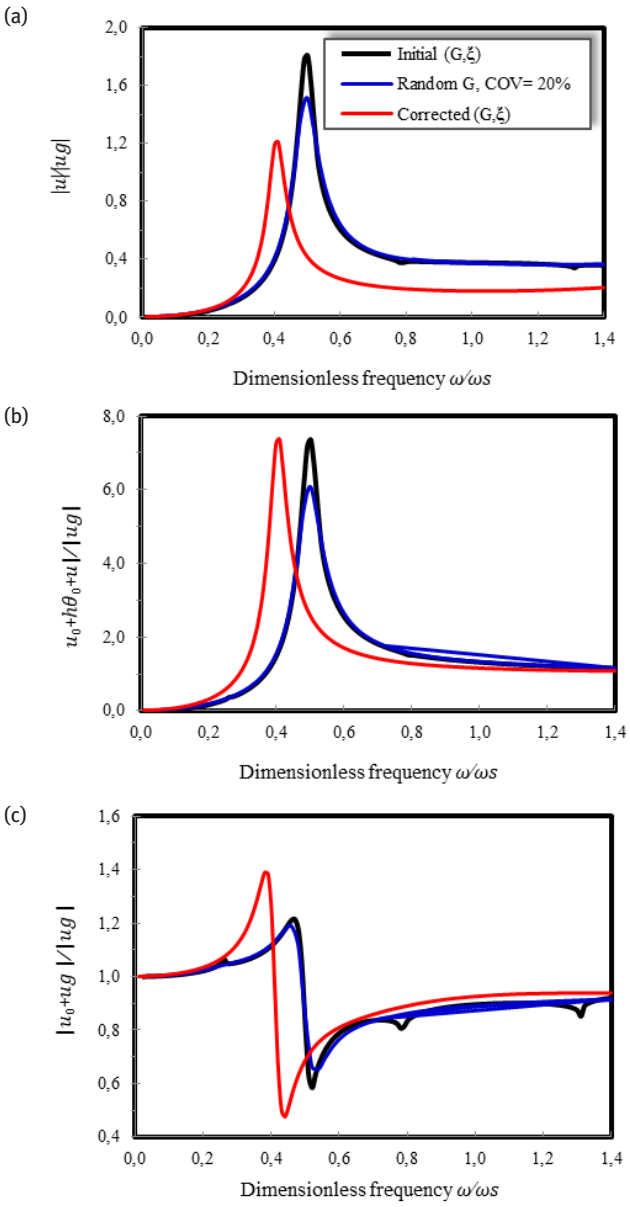


Figure 5: Comparison between stochastic (random G) and nonlinear (corrected G and ξ) SSI responses: (a) structure displacement, (b) mass displacement, (c) base displacement.

modulus (COV = 20%), the displacement remains more or less unchanged compared to the initial case but it reduces by 10.4% due to nonlinear effects. Table 1 summarizes the peak SSI responses and the error margin between stochastic and deterministic response and between nonlinear and deterministic (initial G) response.

It can be seen from Table 1 that, for corrected G and ξ, the frequency ratio (ω/ω_s) goes on decreasing about 18% or time period goes on increasing for the three displacements (Figs. 5a to 5c). This trend means that soil becomes more flexible when shear degradation and damping variation are taken into account. Also, when the shear modulus goes on decreasing and damping coefficient increasing the displacements of structure, masse and the base reduce by 33%, 3% and 10%, respectively, compared to SSI responses for deterministic (initial G and ξ).

Table 1: Error margin (%) in displacements and frequency SSI system due to random G and corrected (G and ξ).

SSI response	Deterministic	Stochastic (20% COV of G)	Nonlinear	Error margin (%)	
				20% COV of G vs. initial G	Corrected (G & ξ) vs initial G
$\|u\|/\|ug\|$	1.81	1.51	1.21	- 16.6%	- 33.1%
$\|u_0+h\theta_0+u\|/\|ug$	7.35	6.07	7.37	- 17.4%	+ 0.3%
$\|u_0+ug\|/\|ug\|$	0.86	0.86	0.95	±00.0%	+ 10.4%
Frequency (ω/ω_s)	0.5	0.5	0.41	±00.0%	-18.0%

4.3 Effects of the correction of the foundation damping on the SSI response

Usually, the seismic calculation of the structure is performed using the response spectrum with a well-fixed damping ratio. However, in reality, there is not only the soil damping but there is also the foundation damping that includes both effects of energy loss from waves propagating away from a vibrating foundation (radiation damping) and hysteretic action in supporting soil (material damping).

Accordingly, the effects of the foundation damping of flexible base on the design spectrum (Fig. 6) of four design codes (RPA99-2003,[17] EC8-2004,[5] IBC-2015[18] and IS-1893-2002[19]) are studied. Furthermore, to further enrich the information, the base shear force of structure is also studied (Fig. 7). The design spectrum and the base shear values are evaluated according to the four cases for each code: (i) fixed base, (ii) SSI effects (initial G and x), (iii) nonlinear SSI (corrected G and x), and (iv) stochastic SSI (random G with COV = 20%). Table 2 in **Appendix III** summaries the ordinates of design spectra and shear base force for the four codes.

The total soil-structure system damping ratio is calculated as follows:[61]

$$\xi_0 = \xi_f + \frac{\xi}{\left(\overline{T}/T\right)_{eff}^2} \tag{4}$$

The flexible-base damping ξ_0 is contributed from both the structural viscous damping ξ and the foundation damping ξ_f consisting generally of radiation and material damping components. T is the fixed base period and $\overline{T}$ the period of the SSI system. The damping of the flexible base of foundation is a combination of soil damping and radiation such as:

$$\xi_f = \left[\frac{\left(\overline{T}/T\right)^2 - 1}{\left(\overline{T}/T\right)^2}\right]\xi_s + \xi_{rd} \tag{5}$$

The radiation damping ξ_{rd} of the soil-structure system is calculated based on a combination of translational damping ξ_y and rotational damping ξ_{xx}:

$$\xi_{rd} = \left(\frac{1}{\left(\overline{T}/T_y\right)^2}\right)\xi_y + \left(\frac{1}{\left(\overline{T}/T_{xx}\right)^2}\right)\xi_{xx} \tag{6}$$

Figs. 6 and 7 show the reduction in design spectrum and base shear, respectively, due to SSI using Eq. (4) for the three cases (ii, iii and iv) mentioned above, where they are compared to the fixed base case for the four codes.

It appears from Figs. 6a–6d (red line) that when including SSI effects with the foundation damping (flexible base case) of the system, a significant decrease appears in design spectral values. This decrease is of 9%, 10%, 6% and 2% for RPA99-2003,[17] EC8-2004,[5] IBC-2015[18] and IS-2002[19] seismic codes, respectively, compared to the fixed base (black line).

Besides, Figs. 6a–6d (green line) indicate that the same structure responds more close to the previous case when the shear modulus is taken as random variable and only the IBC-2015[18] code shows a remarkable decrease. The design spectral values decrease, compared to the fixed base (black line) for the four codes, by 7%, 9%, 13% and 5%, respectively.

In the last instance, Figs. 6a–6d (blue line) show the effects of the correction of both shear modulus and damping coefficient of the soil on the design response spectra where the correction effect clearly appears by a huge decrease observed in the responses compared to the fixed base and especially for the three codes EC8-2004,[5] IBC-2015[18] and IS-1893.[19] The observed reductions in the response spectra for the four codes are 18%, 36%, 37% and 43%, respectively.

On the other hand, Fig. 7 shows the effect of SSI on the base shear for the four codes for the three cases included: SSI effects (initial G and x) in red line, nonlinear SSI (corrected G and x) in blue line, and stochastic SSI

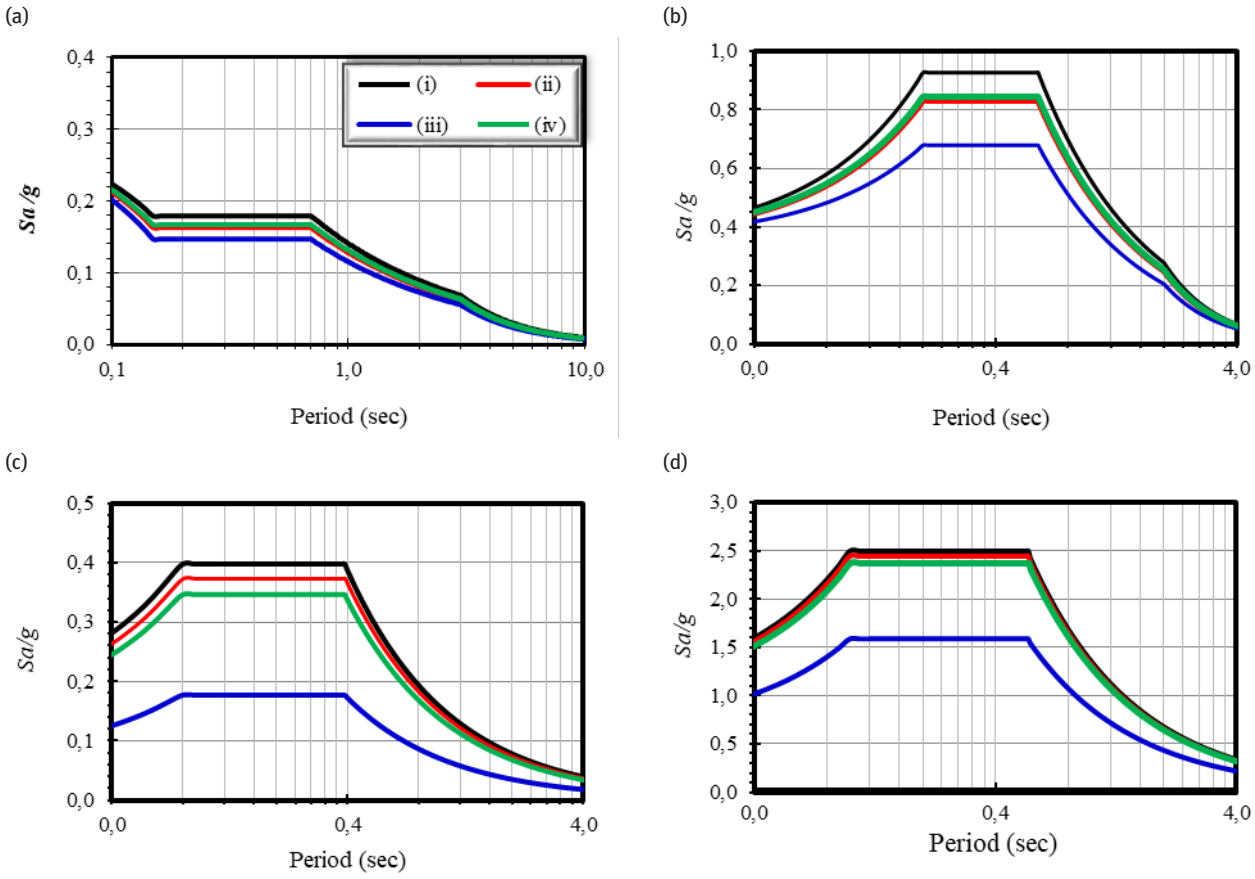


Figure 6: Design response spectrum: (a) RPA99-2003, (b) EC8-2004, (c) IBC-2015 and (d) IS-1893-2002.

(random G with COV = 20%) in green line compared to the fixed base in black line. It is observed that for SSI analysis including flexible base damping, the base shear forces of the structure are noticeably decreased by 9%, 3%, 5% and 4.5%, respectively. In the stochastic case, the results are very close to the previous case results, but only the IBC-2015[18] code shows remarkable decrease, as 7%, 3%, 11% and 7.5%, respectively, compared to the fixed base for the four codes.

Lastly, the effect of the correction of G and ξ of soil on the base shear force for all codes' cases, where remarkable decreases of 18%, 9%, 47% and 46%, respectively, are displayed. For the two last codes, results are similar to those obtained by Mittal and Gajinkar[62] when studying the response of chimney by incorporating strain dependent shear modulus, where they find that the design shear force reduces between 31% to 47% when the soil become more flexible. Whereas Worku,[63] when studying the importance of inertial SSI on the design spectral and on the shear base for EC8[5] and NEHRP-2003,[61] found a reduction between 7% to 39% for two different flexible soil types and four different varying structure heights. On other hand, Jayalekshmi and Chinmayi[64] showed that the design base shear forces obtained as per conventional design practice are higher compared to SSI values and decrease with an increase in foundation stiffness, based on a comparative study on seismic provisions of IS-1893[19] and IBC-2006[65] codes for a multi-storey reinforced concrete framed buildings. Nevertheless, it is important to note that some of the SSI codes cap the maximum allowed base shear reduction to 30%.

5 Conclusions

The present study demonstrated the importance of taking into account of inertial SSI. A comparative analysis on design response spectra and base shear forces was carried out for four seismic codes (Algerian Seismic Rules RPA99 – 2003,[17] EC8-2004,[5] International Building Code IBC-2015[18] and the Indian code IS-1893-2002[19]) considering the three cases of SSI: (*i*) SSI effects (initial G and x), (*ii*) nonlinear SSI (corrected G and x), and (*iii*) stochastic SSI (random G with COV = 20%).

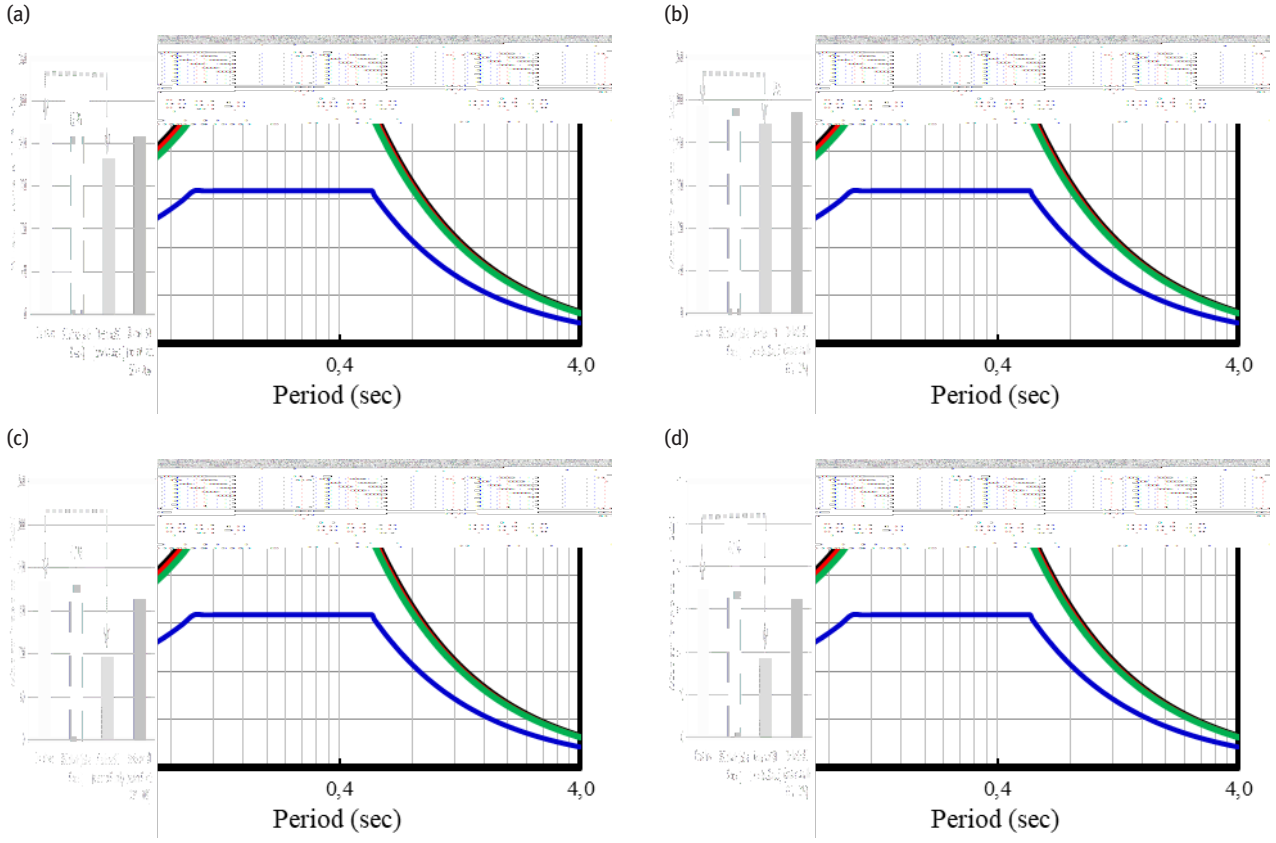


Figure 7: Base shear forces: (a) RPA99-2003, (b) EC8-2004, (c) IBC-2015 and (d) IS-1893-2002.

Firstly, the great influence of the corrected shear modulus and damping coefficient of soil on the impedance functions and the response of structure in the case of SSI analysis was shown. Results showed a shift in the frequency of flexible base by 18% and a decrease in the response of the structure.

Secondly, the influence of the modification of damping of the SSI system for the three cases (SSI effects – initial G and x), nonlinear SSI (corrected G and x) and stochastic SSI (random G with COV = 20%) compared to the fixed base on the design response spectra and shear base forces for the four codes was shown schematically in Figs. 6 and 7. From the results and figures, it was seen that design response spectra and base shear forces for the four codes (RPA99-2003,[17] EC8-2004,[5] IBC-2015[18] and IS-1893-2002[19]) are very sensitive to foundation damping of the flexible base.

A comparative study between seismic codes has been conducted to demonstrate the similarities and dissimilarities between them. The IBC-2015[18] and IS-1893[19] codes show considerable decreased values in design spectra and shear base force compared to the other codes. These results agree with the results in literature, where the shearing strains due to earthquakes ranging between 0.01 and 0.5% reduce shear modulus by 0.9 and 0.2 G_{max}, respectively.

Hence, ignoring the SSI effects in seismic design could lead to large overestimation of design response spectra for buildings. This could lead to uneconomic designs, which can be avoided if SSI is accounted for. Furthermore, the effects of correction of shear modulus and damping ratio on the response spectra of SSI system is very important, and in this case study, the total system damping exceeds 20%. Moreover, results of such a study may encourage to include SSI effects in some seismic code like Algerian Seismic Rules.

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Appendix I

To arrive at the corrected ***G*** and damping ***ξ*** value, these steps are followed:

1. Identify the bedrock level (d) of the site.
2. Find out the shear wave velocity from the expression: $G = \rho\, v_s^2$.
3. Find out the free field time period of the site for the nth mode from the expression: $T_n = \frac{4.d}{(2n-1).v_s}$.
4. Based on the site response spectra/spectra given in code and damping value as obtained in soil report obtain the acceleration *Sa (RPA 99 – version 2003 [16])*.
5. Obtain shear strain for the soil profile based on the expression: $\gamma_z = -\frac{16.\beta.S_a.d.\rho}{(2n-1).\pi.(\pi+2).G}\sin\frac{(2n-1).\pi.z}{2.d}$.
6. Check if this strain is near or equal to the initial strain (10–3 to 10–4)% .
7. If there exists a significant variation correct G based on the equation $G = G_{max}\,(1+\psi/\psi r)$.
   where:
   - *G is the new value of shear modulus;*
   - G_{max} *is the 1st shear modulus calculate in eq. 2;*
   - ψ *is strain range; and*
   - ψr *is reference strain range*.
8. Find out the ratio G/G_{max}.
9. Obtain new damping ratio based on Zhang's expression: $\xi = 0.333\frac{1+e^{(-0.0145PI^{1.3})}}{2}\left[0.586.\left(\frac{G}{G_{max}}\right)^2 - 1.547\frac{G}{G_{max}} + 1\right]$
   where:
   - *PI is plastic Index*.
10. Repeat the steps as mentioned from ***2*** to ***7*** till the strain is same as the previous cycle.
    - When the value for which the strain becomes constant is the corrected dynamic shear modulus of the soil.

Appendix II

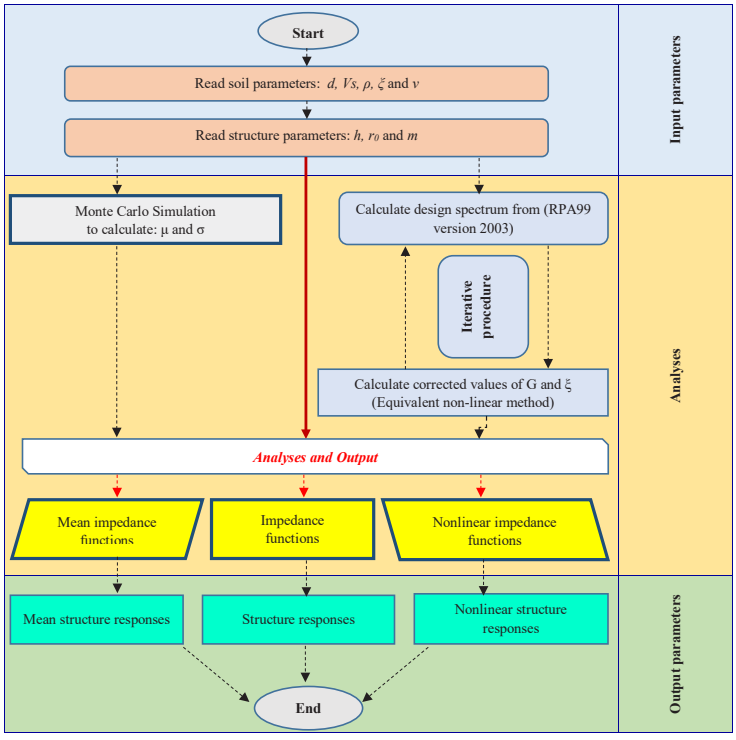

Figure 8: Flow chart of method analysis steps

Appendix III

Table 2: Ordinate of elastic and inelastic design spectra for RPA99-2003, EC8-2004, IBC-2015 and IS-1893-2002.

	$T \le T_B$	$T_B \le T \le TC$	$T \ge T_C$	Seismic base shear
RPA99v2003	$\frac{S_a}{g}=1{,}25.A.\left[1+\frac{T}{T_1}\left(2.5\eta\frac{Q}{R}-1\right)\right]$ $(T_1=T_B)$	$\frac{S_a}{g}=1{,}25.A.2{,}5.\eta.\left(\frac{Q}{R}\right)$ $(T_1\le T\le T_2)$	$\frac{S_a}{g}=1{,}25.A.2{,}5.\eta.\left(\frac{Q}{R}\right)\left(\frac{T_2}{T}\right)^{2/3}$ $(T_2\le T\le T_3)$ $\frac{S_a}{g}=1{,}25.A.2{,}5.\eta.\left(\frac{Q}{R}\right)\left(\frac{T_2}{3}\right)^{2/3}\left(\frac{T}{3}\right)^{5/3}$ $(T_2\le T\le T_3)$	$V=\frac{A.D.Q}{R}.W$ {A:area acceleration coefficient, D:average dynamic amplification factor, Q:quality factor, R:overall behavior coefficient of the structure, W: The effective seismic weight of the structure
EC8-2004	$S_e=a_g.S.\left[1+\frac{T}{T_B}(\eta 2.5-1)\right]$ $S_d=a_g.S.\left[\frac{2}{3}+\frac{T}{T_B}\left(\frac{2.5}{q}-\frac{2}{3}\right)\right]$	$S_e=2.5.a_g.S.\eta$ $S_d=2.5.a_g.S.\eta$	$T_C\le T\le T_D \rightarrow S_e=2.5.a_g.S.\eta.\left[\frac{T_C}{T}\right]$ $T_C\le T\le T_D \rightarrow S_d\begin{cases}=\frac{2.5}{q}.a_g.S.\left[\frac{T_C}{T}\right]\\ \ge\beta.a_g\end{cases}$ $T_D\le T\le 4s\rightarrow S_e=2.5.a_g.S.\eta.\left[\frac{T_C T_D}{T^2}\right]$ $T\ge T_D\rightarrow S_d=\frac{2.5}{q}.a_g.S.\left[\frac{T_C T_D}{T^2}\right]\ge\beta.a_g$	$F_b=S_d(T)\frac{W}{g}.\lambda$ $\begin{cases}\lambda=0.85 \text{ if } T_1\le 2T_C & \text{or}\\ \lambda=1.00 \text{ otherwise}\end{cases}$
IBC-2015	$\frac{S_a}{g}=1+15T$ $(0.00\le T\le 0.10)$	$S_a=S_{DS}$ $(T_0\le T\le T_s)$	$S_a=\frac{S_{DS}}{T}$ $(T>T_s)$	$V=C_s W$ {C_s:The seismic response coefficient, W: The effective seismic weight of the structure
IS-1893-2002	$\frac{S_a}{g}=1+15T$ $(0.00\le T\le 0.10)$	$\frac{S_a}{g}=2.50$ $(0.10\le T\le 0.40)$	$\frac{S_a}{g}=1.00/T$ $(0.40\le T\le 4.00)$	$V=\frac{Z.I.S_a}{2.R.g}.W$ {Z:zone factor, I:importance factor, S_a:average response acceleration coefficient, R:response reduction factor, g:acceleration due to gravity, W:Seismic weight of the structure