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An improved replacement oscillator approach for soil-structure interaction analysis considering soft soils

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ABSTRACT

This paper aims to improve the effectiveness of the replacement oscillator approach for soil-structure interaction (SSI) analysis of flexible-base structures on soft soil deposits. The replacement oscillator approach transforms a flexible-base single-degree-of-freedom (SDOF) structure into an equivalent fixed-base SDOF (EFSDOF) oscillator so that response spectra for fixed-base structures can be used directly for SSI systems. A sway-rocking SSI model is used as a baseline for assessment of the performance of EFSDOF oscillators. Both elastic and constant-ductility response spectra are studied under 20 horizontal ground motion records on soft soil profiles. The effects of frequency content of the ground motions and initial damping of the SSI systems are investigated. It is concluded that absolute acceleration spectra, instead of pseudo-acceleration spectra, should be used for EFSDOF oscillators in force-based design of SSI systems. It is also shown that using an EFSDOF oscillator is not appropriate for predicting the constant-ductility spectra when the initial damping ratio of the SSI system exceeds 10%. Based on the results of this study, a correction factor is suggested to improve the accuracy of the replacement oscillator approach for soft soil conditions.

1. Introduction

The preliminary design of typical building structures in current seismic design codes and provisions is mainly based on elastic spectrum analysis, where the inelastic strength and displacement demands are estimated by using modification factors, such as the constant-ductility strength reduction factor R_μ (i.e. reduction in strength demand due to nonlinear hysteretic behaviour) and inelastic displacement ratio C_μ [1–3]. The spectral shapes of elastic response spectra and modification factors in most seismic design codes and provisions (e.g. [3,4]) are derived by averaging the results of response-history analyses performed on single-degree-of-freedom (SDOF) oscillators using a number of earthquake ground motions [5–7]. In engineering practice, the frequency content of a ground acceleration motion at a soft soil site is often characterized by a predominant period [8] as an influential parameter for estimating the seismic response of buildings.

It is well known that spectral accelerations for soft soil sites attain their maximum values at specific periods T_P, which correspond to the resonance between the vibration of buildings and the amplification of seismic waves travelling upwards through various soil deposits [9]. However, most current seismic codes adopt design acceleration spectra that are smoothed by the averaging of a number of spectra whose peak ordinates may occur at significantly different values of T_P. As a consequence, averaging these dissimilar spectra leads to a flatter spectrum for soft soil profiles than for rock and stiff soil sites, while disregarding the frequency content of the ground motions [7].

Xu and Xie [10] developed the concept of a Bi-Normalized Response Spectrum (BNRS) by normalizing the spectral acceleration S_a and the period of the structure T by the Peak Ground Acceleration (PGA) and the spectral predominant period T_P of each ground excitation, respectively. Based on analyses performed using 206 free-field records of the Chi-Chi earthquake (1999), they found that the BNRS curves were practically independent of site class or epicentre distance, and thus represented a good substitute for the code-specified design spectra that are based on simple averaging of spectral values. In a follow-up study, Ziotopoulou and Gazetas [7] demonstrated that BNRS can preserve the resonance between soil deposits and excitations, thereby reflecting more realistically the effects of the frequency content of the ground motion.

Comprehensive studies have been carried out in the past three decades to calculate values of constant-ductility strength reduction factor R_μ and inelastic displacement ratio C_μ for fixed-base structures

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[11,12]. It has been shown that R_μ and C_μ usually reach their maximum and minimum values, respectively, at the predominant period of the ground motion T_g, which is defined as the maximum ordinate in the relative velocity spectrum calculated for an elastic SDOF system having a 5% damping ratio. It has also been observed that, in the vicinity of T_g, maximum inelastic displacements are sometimes smaller than the elastic displacement demands. It should be noted that the predominant period is mainly a characteristic of soft soils.

The studies discussed above all assumed that the structures were rigidly supported, adopted a viscous damping ratio between 2 and 5%, and disregarded the effects of soil stiffness and damping within the soil domain, also known as soil-structure interaction (SSI) effects. However, it is well known that SSI can significantly affect the seismic response of superstructures, especially those on soft soil profiles [13,14]. Khoshnoudian et al. [15] and Khoshnoudian and Ahmadi [16,17] investigated the effects of SSI on the seismic performance of nonlinear SDOF and multi-degree of freedom (MDOF) systems and proposed empirical equations to predict the inelastic displacement ratios. However, the results of their studies were mainly based on pulse-like near-field earthquakes, and therefore, may not be directly applicable for other types of earthquake ground motions.

For design purposes, an SSI system is usually replaced by an equivalent fixed-base SDOF (EFSDOF) oscillator (also called replacement oscillator) having an elongated period of T_{ssi}, an effective initial damping ratio of ξ_{ssi} and an effective ductility ratio of μ_{ssi}. Inelastic and linear EFSDOF oscillators were adopted by Mekki et al. [18] and Moghaddasi et al. [19], respectively, by using inelastic spectra and equivalent linearization to facilitate a design procedure for nonlinear flexible-base structures. Similarly, Seylabi et al. [20] developed a linear EFSDOF oscillator based on equivalent linearization. Since previous studies have shown that using inelastic response spectra can provide more accurate design solutions for nonlinear systems compared to equivalent linearization (e.g. [21,22]), the current study is focused on inelastic EFSDOF oscillators.

The effectiveness of the EFSDOF oscillator approach for seismic design of structures located on soft soil sites is evaluated in this paper. A sway and rocking SSI model, which provides sufficient accuracy for modelling the dynamic soil-structure interaction in engineering practice (e.g. [13,14]), is used as a reference to assess the accuracy of the results obtained using the EFSDOF oscillators. The effects of both SSI and frequency content of seismic excitations on elastic and inelastic response spectra are investigated using the adopted SSI models and the EFSDOF oscillators for 20 far-field earthquake ground motions recorded on soft soil sites. The results are then used to improve the EFSDOF oscillator for predicting constant-ductility spectra of flexible-base structures on soft soil profiles. The current study, for the first time, proposes improvements to the replacement oscillator approach and explicitly includes the effect of frequency content of ground motions on soft soils in SSI analysis. The paper provides a description of the adopted SSI model and key design parameters, as well as the EFSDOF oscillator. Limitations of the EFSDOF oscillator approach for highly damped SSI systems are identified and some modifications are suggested to improve predictions. The strengths and potential applications of the improved EFSDOF approach to SSI procedures in performance-based design are also addressed.

2. Soil-structure interaction model

For the SSI model adopted in this study, the superstructure is idealized as an equivalent SDOF oscillator having a mass m_s, mass moment of inertia J_s, effective height h_s, and lateral stiffness k_s. In response to seismic loading, the oscillator is assumed to exhibit elastic-perfectly plastic behaviour as an energy dissipation mechanism, in addition to having a viscous damping ratio of ξ_s in its elastic state. This nonlinear hysteretic model can simulate the seismic behaviour of non-deteriorating structural systems such as buckling-restrained braced

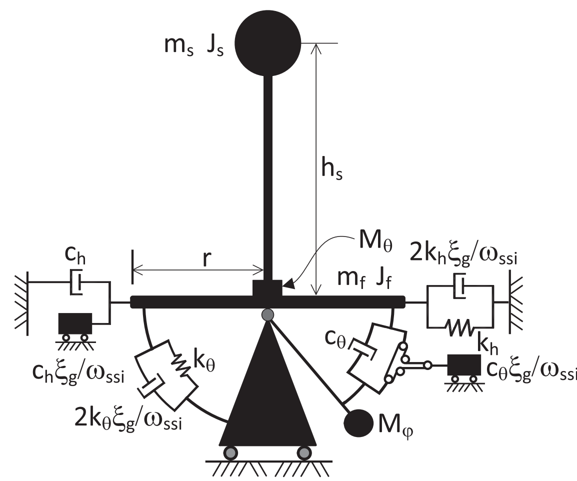

Fig. 1. Soil-structure interaction model.

frames and moment resisting steel frames. The superstructure represents either a single-storey or a multi-storey building corresponding to its fundamental mode of vibration.

The dynamic behaviour of the shallow foundation is simulated using a discrete-element model, which is based on the idealization of a homogeneous soil under a rigid circular base mat as a semi-infinite truncated cone [23]. The accuracy of this model has been validated against more rigorous solutions [24,25]. Fig. 1 shows the SSI model used in this study, which consists of a superstructure and a foundation with sway and rocking components defined by Wolf [25] as follows:

$$k_h = \frac{8\rho v_s^2 r}{2-\nu}, c_h = \rho v_s \pi r^2 \tag{1}$$

$$k_\theta = \frac{8\rho v_s^2 r^3}{3(1-\nu)}, c_\theta = \frac{\rho v_p \pi r^4}{4} \tag{2}$$

$$M_\theta = 0.3\left(\nu-\frac{1}{3}\right)\pi\rho r^5, M_\varphi = \frac{9}{128}(1-\nu)\pi^2\rho r^5\left(\frac{v_p}{v_s}\right)^2 \tag{3}$$

where k_h, k_θ and c_h, c_θ correspond to the zero-frequency foundation stiffness and high-frequency dashpot coefficient for the sway and rocking motions, respectively. The circular foundation beneath the superstructure is assumed to be rigid, with a radius r, mass m_f and centroidal mass moment of inertia J_f. For simplicity, the superstructure is assumed to be axisymmetric with its mass uniformly distributed over a circular area of radius r. Therefore, the moment of inertia J of either the superstructure or the foundation is equal to $mr^2/4$, m being the corresponding mass of the foundation m_f or the superstructure m_s. The homogenous soil half-space is characterized by its mass density ρ, Poisson's ratio ν, as well as the shear and dilatational wave velocities v_s and v_p. An additional rocking degree of freedom φ, with its own mass moment of inertia M_φ is introduced so that the convolution integral embedded in the foundation moment-rotation relation can be satisfied in the time domain. The matrix form of the equations of motion of the SSI model shown in Fig. 1, subjected to a ground acceleration time-history, is given in Appendix A. The authors implemented the nonlinear dynamic analyses in MATLAB [26]; results were obtained in the time domain using Newmark's time-stepping method. In order to solve the nonlinear equations, the modified Newton-Raphson's iterative scheme was utilized. The performance of the linear SSI model was verified against results obtained using the foundation impedance functions [27]; for inelastic structures the model was verified using the central difference numerical integration method [28].

Note that soil incompressibility leads to a high value of v_p (i.e. $v_p \rightarrow$