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Seismic response of buried reservoir structures: a comparison of numerical simulations with centrifuge experiments

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ABSTRACT

Centrifuge experiments on seismic performance of relatively stiff underground reservoir structures in dry sand are modeled numerically. The capabilities of numerical simulations with calibrated equivalent linear soil properties in capturing the main features of experimentally measured responses are explored for both low and high amplitude earthquake motions. The scattering effects of the centrifuge container boundaries are also investigated by modeling the same soil deposit resting on an elastic bedrock and extending infinitely laterally, using the domain reduction method. It is observed that the calibrated equivalent linear soil models perform well in predicting accelerations, racking, and bending strains on the buried structure, even for high amplitude motions for which significant soil nonlinearity is expected. While not as accurate, the seismic lateral earth pressures predicted with these models are in fair agreement with direct measurements made with tactile sensors. The mismatches in earth pressures are likely due to local nonlinearities of soil and frictional contact, which were absent from the numerical models. It is also observed that the scattering effects of the container boundaries become more significant closer to the soil surface, and their characteristics are seen to depend on both the side boundaries and the embedded structure's stiffness.

1. Introduction

Seismic response of underground structures is a complex soil-structure interaction problem influenced by (*i*) the structure's geometry, inertia, and stiffness, (*ii*) the soil heterogeneity and nonlinearity, and (*iii*) the input motion characteristics. Existing methods for analyzing the responses of such structures are usually based on simplified analytical or numerical methodologies and their ranges of applicability are not yet adequately validated against physical model studies (see, for example, [1]).

Recently, Hushmand et al. [2] conducted a series of centrifuge experiments at the University of Colorado Boulder to investigate the seismic performance of relatively stiff structures buried in dry sand. Three different simplified box structures were designed to represent the characteristics of prototype reinforced concrete reservoir structures with varying stiffnesses. These structures were restrained from excessive rotational movements at the top and the bottom by their roofs and floors. Investigation of these experimental results showed that commonly used procedures could not adequately capture the loadings and deformations experienced by this class of underground structures for the ranges of stiffness and the sets of ground motions regularly considered in their design [2]. This is mainly because these procedures are usually based either on the assumption of a yielding (e.g., [3]) or a rigid-unyielding wall (e.g., [4]). A yielding wall is expected to deform enough to result in an active or yielding condition in the backfill soil, while a rigid-unyielding wall undergoes no deformation. The structures of interest in this study are expected to deform, depending on their flexural stiffness, but their deformation is restrained. Therefore, these structures do not fall in either of the commonly assumed categories.

Although soil behavior can be highly nonlinear during strong shaking, use of nonlinear soil constitutive models may not always be practical due to general complexities in calibrating their numerous parameters (e.g., [5]). In the present study, we explore the capabilities of calibrated equivalent linear soil models in capturing the seismic response of buried box structures as observed in centrifuge tests. Dynamic soil properties are determined by calibrating the model parameters such that numerically predicted accelerations of the far-field soil column in the centrifuge test match the measurements. We also examine the effects of boundary conditions prescribed in the numerical models on the predicted response of buried structures. For this purpose, we use the so-

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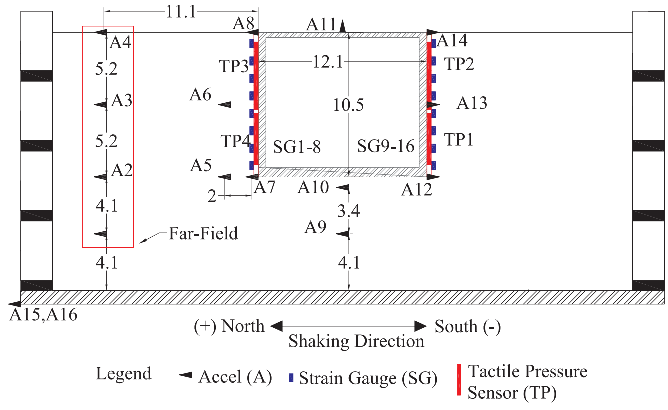

Fig. 1. Layout of the centrifuge tests studied (dimensions in prototype scale meters)[2].

called domain reduction method (DRM) [6,7] to model the same problem in a heterogeneous soil deposit on an elastic bedrock. We also use perfectly matched layers (PMLs) [8,9] as absorbing boundaries to truncate the semi-infinite extent of soil, which is particularly important when extending the numerical simulations beyond the inherently peculiar boundary conditions of centrifuge tests.

2. An overview of the centrifuge tests

The centrifuge test layout and instrumentation are shown in Fig. 1. Dimensions and properties of the model structures used for the experiments are provided in Table 1. The density, Young's modulus and Poisson's ratio of the steel specimen structures were 7870 kg/m^3, 200 GPa, and 0.29, respectively. Dry Nevada sand with the specific gravity of $G_s = 2.65$, minimum and maximum void ratios of, respectively, $e_{\min} = 0.56$, $e_{\max} = 0.84$, median diameter of $D_{50} = 0.13$ mm, and uniformity coefficient of $C_u = 1.67$ was pluviated inside a flexible shear beam container such that an approximately uniform soil layer with a dry unit weight of 15.6 kN/m^3 or a relative density (D_r) of approximately 60% could be achieved.

Fig. 2 shows the 5%-damped spectral accelerations and the Arias Intensity time-histories of the base motions recorded in the centrifuge. Here, numerical results are presented and compared with experimental recordings for four cases that cover a range of stiffness and ground motion intensities. These are, specifically, “flexible” and “stiff” buried structures that were subjected to “Northridge-L” and “Northridge-H” motions (henceforth referred to as AL and AH). The experiments on the flexible and stiff structures will be referred to as T-Flexible and T-Stiff, respectively. The properties of the Northridge-L and H motions as recorded during the T-Flexible-AL and AH experiments are tabulated in Table 2. The mean frequency is the reciprocal of the mean period [10], and the predominant frequency is the frequency at which the maximum 5%-damped spectral acceleration occurs.

3. Numerical modeling of the centrifuge experiments

In the numerical simulations, only the soil inside the container and the structure are modeled. 8-noded quadratic elements are used for the discretization of both the soil and the structure. The finite element (FE)

Table 1
Dimensions and properties of model structures in prototype scale.

	Thickness			Fundamental frequency
Structure	Base (m)	Roof (m)	Walls (m)	(Hz)
Flexible	0.5	0.28	0.28	1.9
Baseline	0.69	0.37	0.56	3.9
Stiff	1.46	1.12	1.13	9.1

code developed by Esmaeilzadeh et al. (see, [9], for details) is used to solve the plane strain elastodynamic heterogeneous half-space problems at hand. The element size is chosen such that approximately 12 discretized nodes exist within the minimum wavelength [11]. It is assumed that the interface of the structure and soil is perfectly bonded. Since a flexible shear beam container is used for the experiments—-which can mimic free-field conditions for vertically propagating shear waves at its two side boundaries—, periodic boundary conditions are imposed on the horizontal degrees of freedom at the left and right vertical edges of the domain, while their vertical degrees of freedom are fixed. This numerical model is referred to as NM1 in subsequent analyses. Both the structure and the soil are assumed to exhibit linear elastic responses. The properties of the structure are the same as those provided in the previous section. The equivalent linear soil properties are obtained through an optimization-based procedure. Details of this procedure are provided next.

3.1. Optimization of equivalent linear properties for the soil domain

The accelerations recorded by sensors A1, A2, A3, and A4 (cf. Fig. 1) are used to optimize the shear wave velocity profile as well as the equivalent viscous damping of the soil domain. It is assumed that the soil density is constant and is equal to 15.6 kN/m^3. The relationship proposed by Rovithis et al. [12] is used to define the general form of a shear wave velocity profile, as in

$$V_s = V_H\left[b + (1-b)\frac{z}{H}\right]^n \tag{1}$$

where $b = (V_0/V_H)^{1/n}$; n is the dimensionless inhomogeneity factor; z is the downward vertical coordinate measured from the soil surface; and V_0 and V_H are shear wave velocities at $z = 0$ and $z = H$, respectively. Soil damping is approximated using the Rayleigh damping model, as in

$$\begin{bmatrix}\xi_1\\ \xi_2\end{bmatrix} = \begin{bmatrix}1/4\pi f_1 & \pi f_1\\ 1/4\pi f_2 & \pi f_2\end{bmatrix}\begin{bmatrix}a_0\\ a_1\end{bmatrix} \tag{2}$$

where f_1 and f_2 are the control frequencies; ξ_1 and are the associated damping ratios; and a_0 and a_1 are the coefficients to define the viscous damping matrix as a function of mass and stiffness matrices, respectively. Although usually the first- and third-mode frequencies of the soil columns are used for determination of control frequencies in site response analyses, it has been reported that selection of controlling frequencies can influence the response of the system significantly and that this choice should be made such that the system does not experience significant over damping in the dominant range of frequencies [13,14]. As a result, in this study, all four parameters (f_1, f_2, ξ_1, ξ_2) are considered as the optimization parameters.

For any given set of shear wave velocity parameters (V_0, V_H, n) and Rayleigh damping parameters (f_1, f_2, ξ_1,), we solve a one dimensional (1D) wave propagation problem of a soil column subjected to seismic input motion at its rigid base. The acceleration responses at locations A1, A2, A3, and A4 in the far-field soil column are then computed to define the following minimization problem:

$$\min_{\mathbf{x}} \quad f(\mathbf{x}) = \frac{1}{2}\sum_{i=1}^{i=4}\sum_{j=1}^{j=n} E_i^*(\omega_j)E_i(\omega_j) \tag{3}$$

where $E_i(\omega_j) = \sqrt{|A_i^e(\omega_j)|}\,[A_i^e(\omega_j) - A_i(\omega_j)]$; $(.)^*$ denotes the conjugate transpose of its subtended variable; $A_i^e(\omega_j)$ and $A_i(\omega_j)$ are the complex-valued experimental and numerical acceleration responses of the ith sensor at radial frequency ω_j, respectively, in frequency domain; and $\mathbf{x} = (V_0, V_H, n, \xi_1, \xi_2, f_1, f_2)$ is the optimization variable vector. Only the frequency range of 0–10 Hz is considered for this optimization problem.

In total, we solve the aforementioned optimization problem for four separate cases. The resulting optimal parameters are provided in Table 3. The variation of the shear wave velocity profile with depth as well as the variation of the Rayleigh damping model with frequency are