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1-g Experimental investigation of bi-layer soil response and kinematic pile bending

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ABSTRACT

The effect of soil inhomogeneity and material nonlinearity on kinematic soil–pile interaction and ensuing bending under the passage of vertically propagating seismic shear waves in layered soil, is investigated by means of 1-g shaking table tests and nonlinear numerical simulations. To this end, a suite of scale model tests on a group of five piles embedded in two-layers of sand in a laminar container at the shaking table facility in BLADE Laboratory at University of Bristol, are reported. Results from white noise and sine dwell tests were obtained and interpreted by means of one-dimensional lumped parameter models, suitable for inhomogeneous soil, encompassing material nonlinearity. A frequency range from 0.1 Hz to 100 Hz and 5 Hz to 35 Hz for white noise and sine dwell tests, respectively, and an input acceleration range from 0.015 g to 0.1 g, were employed. The paper elucidates that soil nonlinearity and inhomogeneity strongly affect both site response and kinematic pile bending, so that accurate nonlinear analyses are often necessary to predict the dynamic response of pile foundations.
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1. Introduction

The seismic behavior of pile foundations constitutes a classical problem of soil-structure interaction. Dynamic loads on piles are not only the result of inertial forces induced by oscillating superstructure (*inertial* interaction), but also of deformations of the soil surrounding the pile caused by the propagation of the seismic waves, regardless of the presence of a superstructure (*kinematic* interaction).

Studies on dynamic soil–pile interaction have been carried out over the years, mostly by means of numerical approaches such as the finite-element [1–7] and the boundary-element method [8–14].

Among simplified procedures [15], the dynamic Winkler model [16–20] has provided a reasonably accurate, versatile and economic alternative to the aforementioned rigorous approaches. Furthermore, the so-called *p*–*y* curves, originally developed for nonlinear pile–soil interaction under large static or low-frequency cyclic loads, have been extended to the dynamic regime in terms of lumped-parameter formulations encompassing both stiffness and damping using beds of springs and dashpots attached in parallel [21–28]. In recent years, pseudo-static methods, which constitute an essential tool in engineering practice, have been established for the seismic design of piles [29–31]. Similarly, simplified closed-form expressions for the evaluation of kinematic pile bending have been devoloped [7,32–36].

On the other hand, experimental studies in the field and the laboratory are more limited, primarily because of cost and complexities in carrying out and interpreting such tests. Following early experiments by Novak and co-workers [37,38], Finn and Gohl [39,40] performed centrifuge tests under earthquake loading on model piles. Shaking table scale model tests on single pile and pile groups were presented by Meymand [41] and large scale shaking table tests on pile-structure models were studied by Tokimatsu et al. [42]. Shirato et al. [43] performed large scale shaking table experiments on a 3×3 pile group, while shaking table tests on a soil–pile–structure model subjected to seismic excitation were performed by Chau et al. [44]. Moccia [45] conducted a large number of shaking table tests on a single pile embedded in layered soil at University of Bristol, in an experimental campaign that preceded the one at hand.

In this paper, small scale model shaking table tests, carried out at the BLADE Laboratory in University of Bristol within the framework of the Seismic Engineering Research Infrastructures for European Synergies (SERIES) project [46,47], are presented and discussed. Tests were performed on both single and grouped piles

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Nomenclature

a_{base}	input acceleration level
b	inhomogeneity coefficient
$[C]$	damping matrix of soil column
c_S	soil–pile interface dashpot coefficient
C_u	uniformity coefficient
D_{10}, D_{50}, D_{60}	grain size diameters
E_P	Young's modulus of the pile
E_s	Young's modulus of soil
f	frequency
$f_N^{\ 1}$	first mode frequency of the top layer
$f(t)$	viscous force applied at the base of the soil column
$F(\omega)$	transfer function of the soil deposit
$F_1(\omega)$	transfer function of the top layer
G_{MAX}	elastic soil shear modulus
G_{S1}, G_{S2}	shear modulus in top and bottom soil layers, respectively
h_1, h_2	thickness of top and bottom soil layers, respectively
$[K]$	stiffness matrix of soil column
$[K_P]$	stiffness matrix of pile
k_S	Winkler spring coefficient
$[M]$	masses matrix of soil column
$[M_P]$	masses matrix of pile
n	rate of inhomogeneity parameter
t	time
u	free-field soil displacement
V_0	shear wave velocity in the top layer (value at the soil surface)
V_{h1}	shear wave velocity in the top layer (value at the layer interface)
V_{S1}, V_{S2}	shear wave velocities in top and bottom soil layers, respectively
V_{S1}^*, V_{S2}^*	complex shear wave velocities in top and bottom soil layers, respectively
V_S^R	shear wave velocity in the bedrock
y	pile displacement
z	vertical coordinate
α	impedance ratio of soil layers
α_R, β_R	Rayleigh coefficents
γ	soil shear strain
γ_r, β, s	hyperbolic model parameters
δ	dimensionless parameter relating k_s and E_s
κ_1, κ_2	complex wave numbers in top and bottom soil layers, respectively
ξ_S, ξ_1, ξ_2	damping coefficients of soil material
ρ_1, ρ_2	mass densities in top and bottom soil layers, respectively
ρ_R	mass density in bedrock
σ'	confining pressure
τ	soil shear stress
τ_{mo}	shear stress related to a strain of approximately 1%
ω	cyclic oscillation frequency
ω_N, ω_M	Rayleigh damping control frequencies

embedded in a two-layer soil profile. They aimed at assessing the effects of both kinematic and inertial effects, by attaching caps and simple superstructure models on the piles, or testing them under free-head conditions.

The results of these investigations are interpreted by means of a versatile one-dimensional lumped-parameter model, suitable for heterogeneous and layered profiles, encompassing inertial properties and material nonlinearity. The analyses focus on the nonlinear behavior of the soil and the inhomogeneity effects for both site response and kinematic bending of fixed-head piles. Comparisons with simple equations for evaluating kinematic pile bending are also provided.

2. Experimental layout and instrumentation

The shaking table experiments were conducted using a 3 m × 3 m cast aluminium platform weighing 3.8 t, capable of carrying a maximum payload of 15 t. An equivalent shear beam (ESB) container has been employed to carry the model soil deposit (Fig. 1). The ESB consists of 8 rectangular aluminium rings, stacked alternately with rubber sections to create a hollow flexible box of inner dimensions 1190 mm long by 550 mm wide and 814 mm deep [48].

2.1. Soil profile

Two geo-materials were used to constitute the soil profile: Leighton Buzzard sand fraction B (LBB) and Leighton Buzzard sand fraction E (LBE). Experimental tests are available on these sands to allow a precise characterization [49–51]. The Leighton Buzzard Sand fraction B is constituted by coarse rounded particles with a diameter ranging between 0.6 mm and 1.1 mm. The Leighton Buzzard sand fraction E is a uniform fine sand. Main features of LBE and LBB sands are provided in Table 1. A two-layer soil profile was pluviated into the ESB laminar container (Fig. 2). The free surface of the soil deposit was 800 mm above the laminar container floor. The bottom layer was 460 mm thick, made of LBB and LBE in a 85–15% granular mix, respectively, and for this layer a mass density of 1.78 Mg/m^3 had been achieved. The upper layer was 340 mm thick, contained LBE sand, and achieved a mass density of 1.39 Mg/m^3.

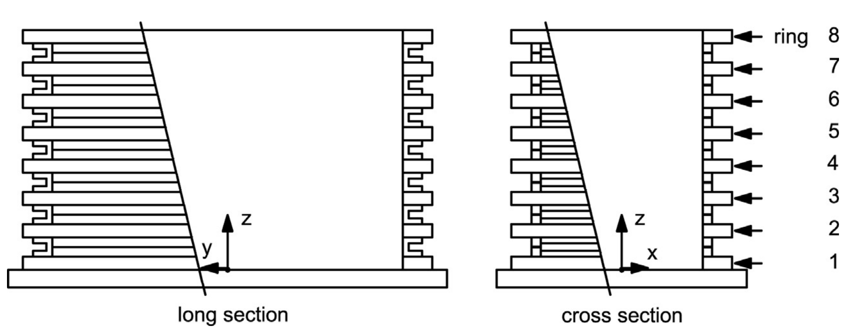

Fig. 1. ESB: equivalent shear beam laminar container.

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