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Simplified *p-y* curves under dynamic loading in dry sand

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ABSTRACT

The behavior of soil-pile systems subjected to dynamic loads in dry sand was investigated using experimental tests. The focus of this study was to quantify the simplified dynamic *p-y* curve for pseudo-static analysis from model tests under various loading frequencies. A framework for determining the *p-y* curve on the basis of 1 g shaking table tests was introduced. Based on the results of the model tests, the dynamic *p-y* curve is highly dependent on the relationships between the natural frequency of the soil-pile system and the loading frequency, the acceleration amplitude and the relative density of soil. The simplified dynamic *p-y* curve is a hyperbolic function. The initial slope and the ultimate soil resistance were proposed as functions of the properties of the pile and soil. The proposed *p-y* curves were determined to be more appropriate than the existing *p-y* curves for representing the soil-pile interaction for a single pile under dynamic loads in dry sand.

1. Introduction

Recently, the number of large-scale earthquakes of magnitude 6.0 or greater, and their resulting tsunamis, has increased worldwide, resulting in many human injuries and substantial property damage. Seismic design has become increasingly important to reduce the threat that earthquake-induced structural deformation and damage poses to communities and property. To analyze the performance of piles under dynamic loads such as earthquakes, a number of analytical and numerical approaches have been proposed based on either linear elastic or viscoelastic models [1–6]. However, pile foundations behave nonlinearly under dynamic loads due to the plastic deformation of the soil, soil-pile separation and slippage. Therefore, nonlinear analysis is required to analyze the behavior of piles under dynamic loads.

Lumped mass models, which feature nonlinear springs and dampers along the pile, as well as gapping mechanisms, were developed to reproduce strong nonlinear effects [7–10]; however, relating the characteristics of the discrete elements to the general soil parameters is difficult [11]. Alternatively, the lateral load transfer curve method, often referred to as the *p-y* curve method, has been studied to define soil stiffness at a particular depth. This method is based on a numerical solution of a physical model based on a beam with a Winkler foundation. *p-y* curves have been established based on full-scale pile head loading tests under static or cyclic loads [12–18].

Pseudo-static analysis, which is a method of converting dynamic loads into the equivalent static loads using inertial loads, is widely used in the seismic design of pile foundations. *P-y* curves, which were proposed by Reese et al. [14] and the API (American Petroleum Institute) [17], are most frequently used for pseudo-static analysis in practical engineering applications. However, *p-y* curves do not properly consider soil stiffness or soil inertia effects under seismic loads because the *p-y* curves are derived from field tests by applying static and cyclic loads at the pile head [19–25].

Considerable work has been conducted by many researchers on laterally loaded piles by considering the dynamic loading conditions to overcome the limitation of the existing *p-y* curves. Ting et al. [26] noted that the secant slope of the dynamic *p-y* curve is highly dependent on the loading frequencies in dynamic pile load tests. On the basis of the API *p-y* curve, Boulanger et al. [27] proposed that the *p-y* element equations can be conceptualized as elastic, plastic and gap components in series. However, the model is too complicated for pseudo-static analysis in practical engineering applications.

The NCHRP (National Cooperative Highway Research Program) [28] described that the dynamic behavior of the soil-pile interaction is closely associated with the pile diameters, shear wave velocities of the soil and loading frequencies. Additionally, the NCHRP suggested dynamic *p-y* curves by using a numerical analysis method that related the static *p-y* curves to the dimensionless frequency. However, the verification of the dynamic *p-y* curves was conducted under restrictive conditions by statnamic tests in which a lateral load was applied on a pile head. El Naggar and Bentley [29] observed that the soil resistance under dynamic loading increases due to the damping effect. They also noted that the dynamic *p-y* curves depended on the loading frequency. Yang et al. [21] and Yoo et al. [30] proposed dynamic *p-y* backbone

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(a)

(b)

(c)

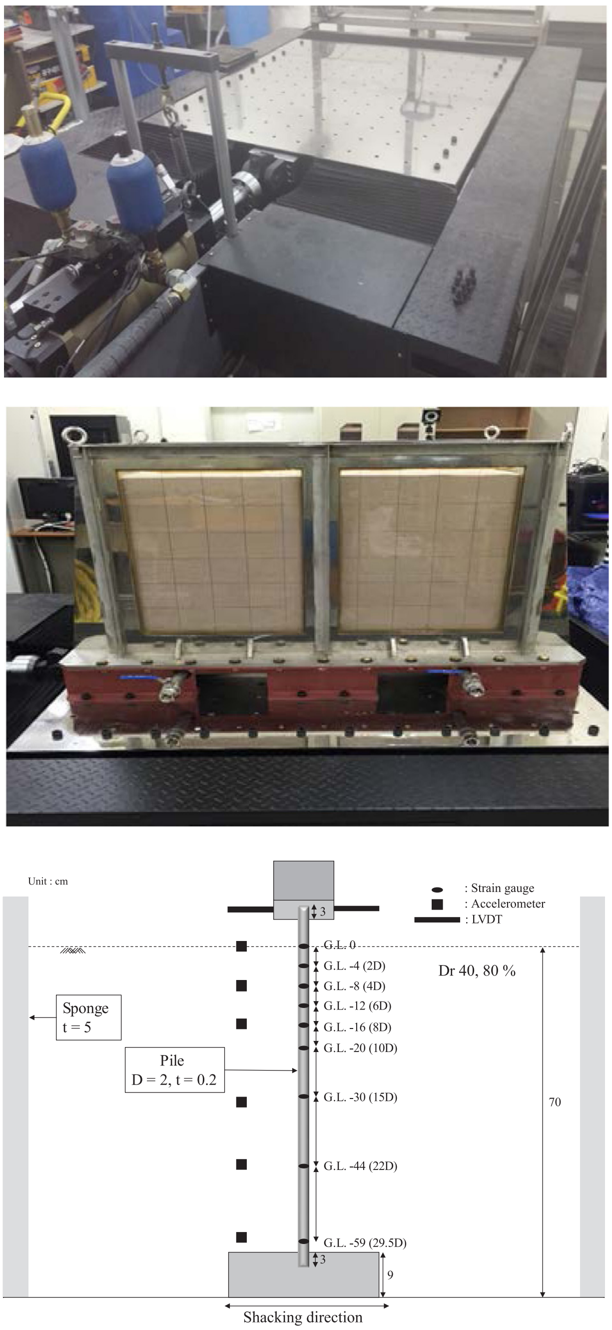

Fig. 1. 1 g shaking test devices: (a) 1 g shaking table; (b) soil box; (c) sectional view of 1 g shaking table test.

curves using 1 g shaking table tests and centrifuge tests, respectively. However, these proposed curves require further verification, and the curves were proposed within a limited range of loading frequencies. Therefore, it is necessary to study dynamic *p-y* curves for pseudo-static analysis considering various ranges of natural frequencies and loading frequencies of soil-pile systems.

The objective of this study is to develop simplified dynamic *p-y* curves for pseudo-static analysis by using model tests under various loading frequencies based on the natural frequencies of soil-pile systems. To achieve the objective, a series of 1 g shaking table tests was performed. The application of this study was evaluated by numerical analysis. The results of the proposed *p-y* curves were compared with those of the existing *p-y* curves.

2. 1 g shaking table test

2.1. Testing apparatus

The dynamic tests on model piles were conducted by using a 1 degree of freedom shaking table under 1 g conditions. Fig. 1(a) and Fig. 1(b) show the 1 g shaking table and the soil box. The shaking table has a platform width of 1500 mm and a platform length of 1500 mm. The maximum sample weight, acceleration and frequency are 2 t, 1.1 g and 50 Hz, respectively. A soil box 1200 × 600 × 800 mm in size was constructed with transparent polycarbonate and aluminum plates. Sponge pads 50 mm thick were placed on the sidewalls of the soil box to reduce the reflection waves in the shaking direction.

Compared with centrifuge tests, 1 g shaking table tests are simpler, can handle larger model sizes and can more easily measure experimental results. However, 1 g shaking table tests cannot be used to produce in situ stress levels due to low confining stresses. To minimize this disadvantage, the model pile was made considering the similitude law proposed by Iai [31]. The diameter and thickness of the model pile are not applicable to the similitude law due to the limitations of the material. Flexural rigidity governs the behavior of a pile subjected to lateral loads; thus, the flexural rigidity of the model pile was scaled to satisfy the similitude law. The model pile was made of aluminum alloy (6061-T6) and had a hollow circular section with an outer diameter of 2.0 cm. The embedment depth was 64 cm (long pile condition, Table 1 [32]). The properties of the model pile are summarized in Table 2.

The 1 g shaking table test device is illustrated in Fig. 1(c). The model pile was fixed to the aluminum plate to simulate a rock-socketed pile. A variety of instruments were installed to monitor the displacements, bending moments and accelerations during the testing. The displacement of the pile was measured by two LVDTs (linear variable differential transformers) located on either side of the pile cap. Strain gauges were installed along the pile to measure the bending moments. Accelerometers were installed on the surcharge mass and within the soil to measure the natural frequencies of the soil-pile system and the free field displacement.

2.2. Test soil

Jumoonjin sand classified as SP (poorly graded sand, according to the Unified Soil Classification System), with a specific gravity G_s 2.65, an effective grain size D_{10} = 0.38 and a uniformity coefficient C_u = 1.59, was used for the 1 g shaking tests. The maximum and minimum dry densities of the test soil were 16.2 kN/m^3 and 13.6 kN/m^3, respectively. The soil compacted by vibrations using the shaking table. The target relative densities were set at 40% for the loose state and 80% for the dense state. The amount of sand needed to obtain the target relative density was prepared for the volume of soil box calculated by the embedment depth of the pile and the size of the soil box. To prepare homogeneous soil, the soil was divided into layers and vibration compaction was performed by layers. The soil was subjected to vibration compaction to a target height corresponding to the target relative density. The number of soil layers, the vibration frequency and the acceleration were determined by a trial and error method to find the optimal value for the target relative density. The particle size distribution curve and the properties of the test soil are shown in Fig. 2 and

Table 1
Pile conditions [32].

Pile	Sand	$\eta = \left(\frac{\eta_h}{EI}\right)^{\frac{1}{5}}$
Short pile	$\eta L < 2.0$	L = Pile length (m)
Immediate pile	$2.0 \le \eta L \le 4.0$	η_h = Constant of subgrade reaction
Long pile	$\eta L > 4.0$	E = Elastic modulus (kN/m^2)
		I = Moment of inertia (m^4)