Evaluation of Pile Diameter Effect on Initial Modulus of Subgrade Reaction

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Abstract: This paper presents the results of a study on the effect of pile diameter on the initial modulus of subgrade reaction. A series of ambient and impact vibration tests were performed on four different diameters of cast-in-drilled-hole piles to determine the natural frequencies and damping of the soil-pile systems. The measured natural frequencies were then compared with those estimated from a numerical model. The soil springs in the numerical model were established by implementing two different concepts on initial modulus of subgrade reaction. One is based on Terzaghi's concept in which the modulus of subgrade reaction is independent of pile diameter. The other was based on recent research suggesting that the initial modulus of subgrade reaction may be linearly proportional to pile diameter. It was found that the measured natural frequencies were in good agreement with the computed ones when the diameter-independent modulus of subgrade reaction was employed. In addition, the test results show that the damping ratio of the system varied with pile diameter from 3% for 0.4-m pile to 25% for 1.2-m pile.

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Introduction

Several analytical methods have been proposed that attempt to model lateral pile response, including the elastic continuum (e.g., Spillers and Stoll 1964; Poulos 1971; Banerjee and Davies 1978; Poulos and Davis 1980; Poulos and Hull 1989), soil mesh finite element (e.g., Desai and Appel 1976; Kuhlemeyer 1979; Winicki and Zienkiewicz 1979; Randolph 1981; Brown et al. 1989), and Winkler spring methods (e.g., Hetenyi 1946; McClelland and Focht 1958; Matlock 1970; Reese et al. 1974; Reese et al. 1975; Reese and Welch 1975; Nogami and Chen 1987). The Winkler spring method appears to be the most extensively used due to the mathematical convenience and ease in taking into account soil nonlinearity. In this method, the surrounding soil is replaced by a series of independent springs attached along the pile. The nonlinear spring characteristics can be represented by relationship between soil resistance per unit pile length (p) and pile deflection (y), widely known as *p-y* curves. Several researchers have proposed methods to construct *p-y* curves for various soil types based upon back computation from full-scale test results (e.g., for sand, see Reese et al. 1974; for soft clay, see Matlock 1970; for stiff clay above water table, see Reese and Welch 1975; and for stiff clay below water table, see Reese et al. 1975). However, most of these were formulated based on a relatively small range of pile diameters, and the theory was then extrapolated for use with other diameter sizes. The degree of accuracy in predicting the response of a laterally loaded pile to relatively wide ranges of pile diameters is of interest, especially for very large pile diameters. Though some experimental studies have been completed on large-diameter piles, few studies have reported specifically on the effect of pile diameter on *p-y* curves based on testing of multiple piles at the same site (Reese et al. 1975; O'Neill and Dunnavant 1984; Dunnavant and O'Neill 1985). Therefore, it is beneficial to study the influence of pile diameter on *p-y* curves in order to provide more insight on laterally loaded pile behavior, as well as improve current analysis methods if necessary.

A research project has been undertaken with the aim of evaluating the seismic performance of deep foundations, and mainly focuses on the effect of pile diameter on *p-y* curves. To achieve this goal, cast-in-drilled-hole (CIDH) piles were installed in weakly cemented sand, a typical soil found along the coast of California, and laterally tested under both static and cyclic loading at the University of California, San Diego (UCSD). However, in this paper, only an investigation of the effect of pile diameter on the initial modulus of subgrade reaction, one portion of the *p-y* curve, is discussed. This paper will refer to definitions of various terms in subgrade reaction theory (e.g., modulus of subgrade reaction K; coefficient of subgrade reaction k; and stiffness of subgrade reaction spring K_s) which are often confused in the literature; therefore, they are summarized in Table 1 to make this paper easier to follow.

There are few discussions available in the literature regarding the pile diameter effect on modulus of subgrade reaction. Terzaghi (1955) explained the influence of pile diameter on the coefficient of subgrade reaction by using the concept of a stress bulb to show that the larger pile diameter has the deeper stress influence than the smaller one. Therefore, with an equivalent applied pressure, a larger pile diameter encounters greater displacement resulting in a lower coefficient of subgrade reaction. Terzaghi concluded that the coefficient of subgrade reaction is linearly proportional to the inverse of pile diameter. In other words, the

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Table 1. Summary of Definition and Dimension of Terms Used in Analysis of Laterally Loaded Piles

Description	Symbol	Definition	Dimension
Soil resistance per unit length	p		F/L
Pile deflection	y		L
Pile diameter	D		L
Spring spacing	ΔL		L
Spring force	F	$F=p*\Delta L$	F
Soil pressure	P	$P=p/D$	F/L^2
Modulus of subgrade reaction	K	$K=p/y$	F/L^2
-Dependent on pile diameter	K_{dep}		F/L^2
-Independent of pile diameter	K_{ind}		F/L^2
Soil spring stiffness	K_s	$K_s=F/y,\ K_s=K*\Delta L$	F/L
Coefficient of subgrade reaction	k	$k=P/y,\ k=K/D$	F/L^3

modulus of subgrade reaction is independent of pile diameter.

Vesic (1961) provided a relationship between the modulus of subgrade reaction K, used in the Winkler spring problem and the material properties in the elastic continuum problem as

$$K=\frac{065E_s}{(1-\mu_s^2)}\left[\frac{E_sD^4}{E_pI_p}\right]^{1/12} \tag{1}$$

where E_s = soil modulus of elasticity; μ_s = Poisson's ratio of soil; D = pile diameter; and E_pI_p = flexural rigidity of pile. This expression also indicates that the modulus of subgrade reaction is independent of pile diameter because the moment of inertia of the pile I_p for a circular and square pile is proportional to the pile width to the fourth power (i.e., D^4). Thus, the diameter term in Eq. (1) disappears. Furthermore, according to the characteristics of the *p-y* curves for sand (Reese et al. 1974) and stiff clay under water table (Reese et al. 1975), the initial modulus of the subgrade reaction appears to be independent of pile diameter.

In contrast, Pender (1993) referred to the studies by Carter (1984) and Ling (1988), which concluded that the initial modulus of subgrade reaction is linearly proportional to pile diameter. This finding was made based upon analysis of published test results of full-scale lateral pile load tests using the hyperbolic soil model (Carter 1984). This simple soil model can be established using only three parameters, including the initial coefficient of subgrade reaction k_o, ultimate soil pressure P_{ult}, and nonlinearity index n, as presented in Fig. 1. The curve of hyperbolic soil model is given as

$$y=\frac{P}{k_o}\left[\frac{P_{\text{ult}}^n}{(P_{\text{ult}}^n-P^n)}\right] \tag{2}$$

where y = soil displacement at any point (L); P = soil pressure (F/L^2); n = index that controls the nonlinearity (1 for sand and 0.2 for clay); k_o = small strain coefficient of subgrade reaction (F/L^3); and P_{ult} = ultimate soil pressure (F/L^2). It should be noted that the dimensions of each variable are given in parentheses.

Although the initial modulus of subgrade reaction for sand *p-y* curves (Reese et al. 1974) is not a sensitive parameter in analyses of lateral pile response, especially in terms of maximum moment (Meyer and Reese 1979), it is important for load-deflection relations, particularly at small load levels. Furthermore, it is a major factor to control the nonlinear characteristic of the hyperbolic soil model to predict the pile response with the reasonable degree of accuracy (Carter 1984). Therefore, it is of great interest to study whether or not the initial modulus of subgrade reaction is independent of pile diameter.

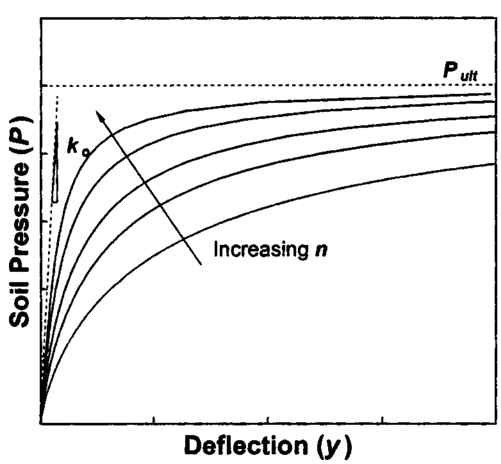

Fig. 1. Hyperbolic soil model (after Carter 1984)

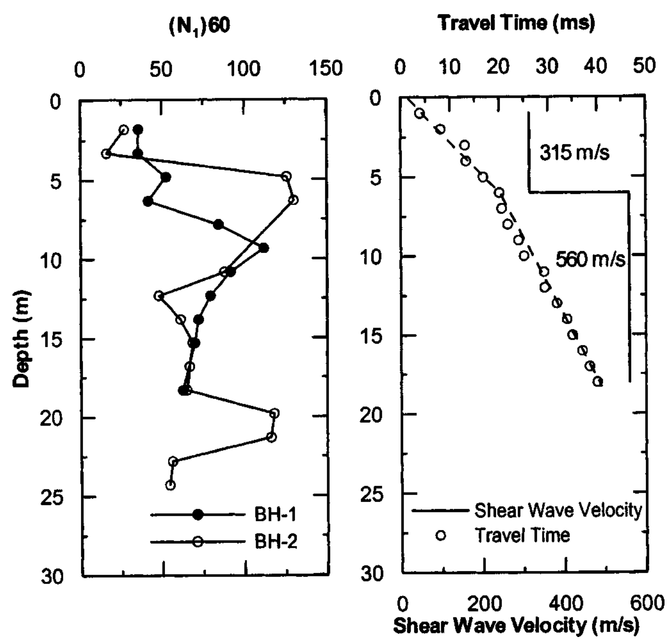

Fig. 2. Soil condition at test site including (a) corrected SPT N value, and (b) shear wave velocity profile

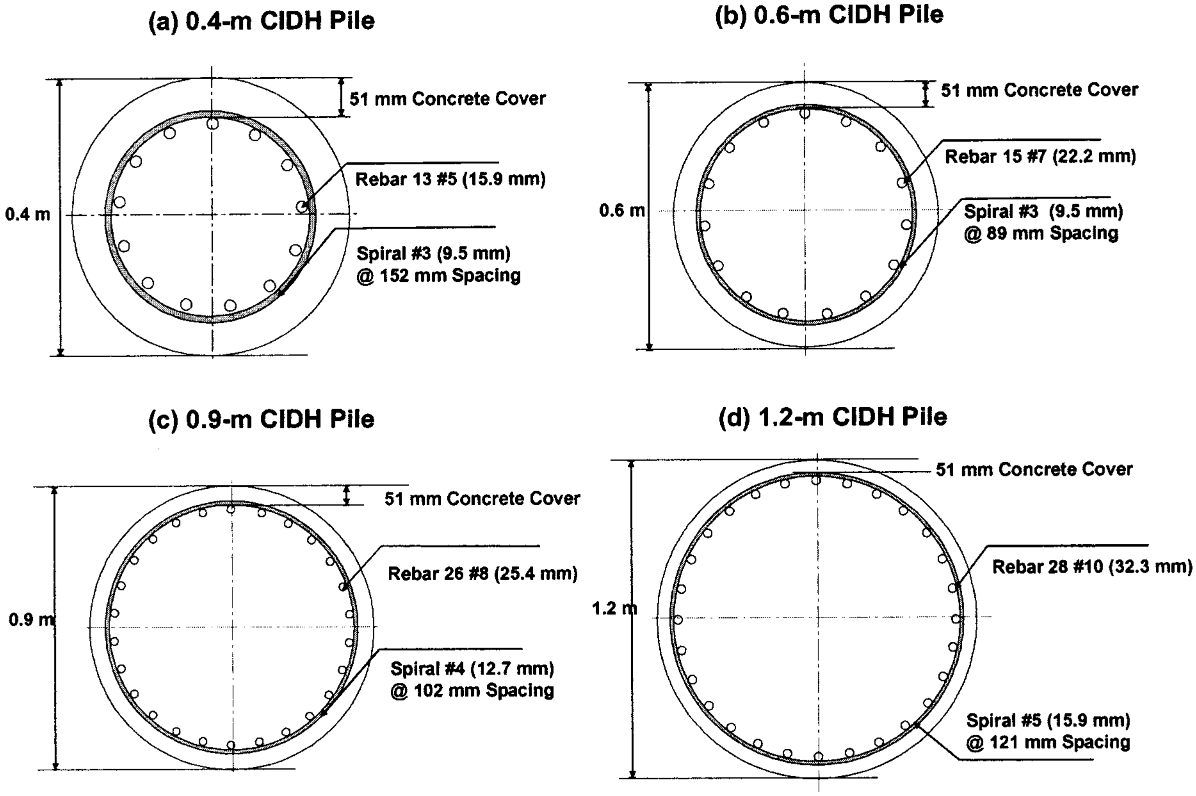

Fig. 3. Pile cross sections

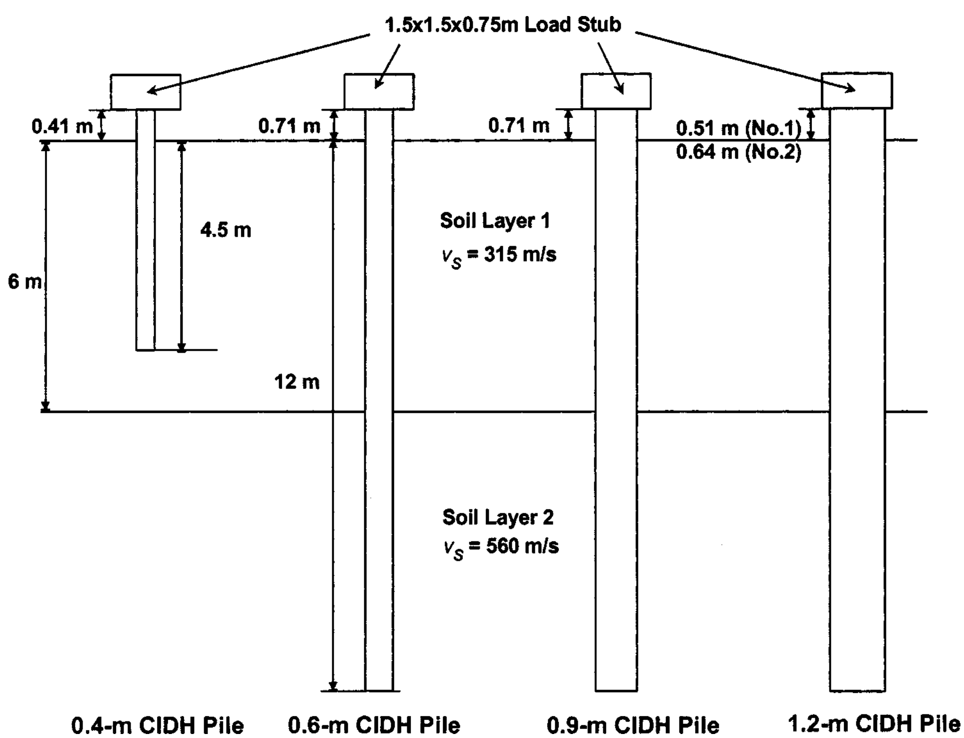

Fig. 4. Pile geometry

Table 2. Summary of Vibration Testing Program

Pile diameter (m)	Additional mass	Ambient Vibration Test		Impact Test	
		North–South	East–West	North–South	East–West
0.4	No	Y	Y	Y	Y
	Yes	Y	Y	Y	Y
0.6	No	Y	Y	Y	Y
0.9	Yes	Y	Y	Y	Y
1.2 (No. 1)	No	N	N	Y	Y
	Yes	N	N	Y	Y
1.2 (No. 2)	No	N	N	Y	Y
	Yes	N	N	Y	Y

Note: Y=Test, N=No Test

A vibration testing program was designed to study the relationship between pile diameter and modulus of subgrade reaction at small strain because it is difficult to evaluate from traditional lateral load tests. The test program consisted of small strain ambient vibration and impact tests to determine the natural frequencies of soil-pile systems. Numerical models were then developed for the soil-pile system based on the two different concepts of modulus of subgrade reaction (i.e., one dependent on pile diameter, the other independent of pile diameter) in order to compute the natural frequencies. The numerical model results were then compared to the experimental results to evaluate the pile diameter effect.

Site Description

The test site is located on the UCSD Campus and is underlain by the Eocene-aged Scripps Formation. According to available geologic literature (GEOCON 1986; Elliot 1988) the Scripps Formation, a marine sedimentary deposit, generally consists of light brown and gray, weakly cemented silty sand interbedded with sandy siltstone, with clay beds and seams. Very hard, cemented concretions occur frequently within this formation.

A subsurface exploration was conducted to obtain more detailed geotechnical information of the test site. Two boreholes were drilled to depths of 20 and 24 m. The groundwater table was not encountered during the soil investigation. According to the soil boring logs and laboratory test results, this site consists of light brown and gray to dark brown, medium dense to very dense weakly cemented clayey to silty sand. The automatic safety hammer was used to conduct the standard penetration test (SPT). The SPT N values were corrected based upon hammer type and release system, sampler configuration, short rod lengths, and overburden stresses (Seed et al. 1984, 1985; Liao and Whitman 1986). The corrected SPT N values, $(N_1)_{60}$ values, vary from 16 to approximately 50 for the first 6 m. Below this layer, the $(N_1)_{60}$ values exceed 50. The $(N_1)_{60}$ values profile is shown in Fig. 2(a). In addition, the shear-wave velocity profile was measured using the seismic down-hole technique. The travel-time curve together with calculated shear wave velocity is presented in Fig. 2(b). This type of stepped profile is common in weakly cemented sands (e.g., Ashford and Sitar 1994).

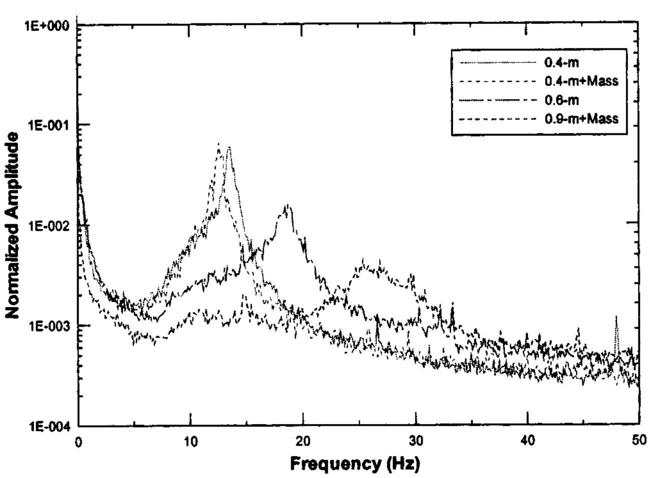

Fig. 5. Power spectrum for CIDH piles in E-W direction from ambient vibration tests

Pile Descriptions

Four different diameters of CIDH piles were installed at the UCSD test site ranging in diameter from 0.4 to 1.2 m. The 0.4-m CIDH pile was 4.5 m long and all others were 12 m long, though all acted as "long piles" (i.e., the piles were long enough that the lateral response was independent of depth). Longitudinal reinforcement of 2% and transverse reinforcement of 0.6% were used. Concrete cover of each pile was approximately 50 mm. The reinforcing steel configuration of individual piles is presented in Fig. 3. The geometry of each pile is shown in Fig. 4.

Testing Program and Testing Procedure

Both ambient and impact vibration testing were conducted. For both tests, several sensitive accelerometers were mounted on the load stub in the north-south (N-S) and east-west (E-W) directions to measure the vibration response of the pile during the test. A signal analyzer was used to acquire and process the data in both time and frequency domains.

For the ambient vibration test, vibrations from wind and other environmental factors were measured to obtain the natural frequencies of the soil-pile system. A power spectrum measurement was chosen to obtain the values of frequency components because the signal analyzer allowed averaging the results of many runs, thus smoothing out the signal. In this case, a total of 100 runs were used for each test. The ambient vibration tests were performed on all piles except the 1.2-m diameter pile where the magnitude of vibration was approximately noise level and the peak representing the natural frequency of the system could not be observed. Additional information on natural frequency was also obtained by testing the pile with an additional mass of ap-

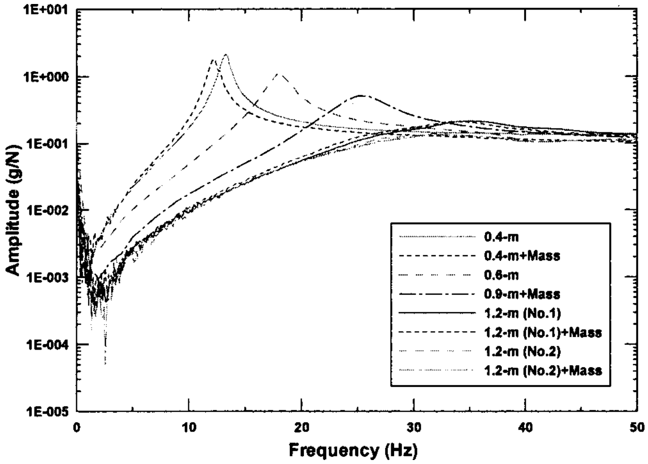

Fig. 6. Frequency response function for CIDH piles in E-W direction from impact vibration tests

proximately 220 kg mounted on the load stub so as to decrease the natural frequency of the system.

Frequency response measurement was used for the impact vibration test, which shows the ratio of the measured output to the input stimulus. A modal hammer with a load cell and rubber tip was struck on the load stub to generate an initial velocity to the pile, and the response under a free vibration of the pile was recorded using accelerometers. In this case, the input stimulus is the time history of forces between hammer and load stub over the strike duration, and the output is the acceleration of the load stub. A summary of vibration testing program is given in Table 2.

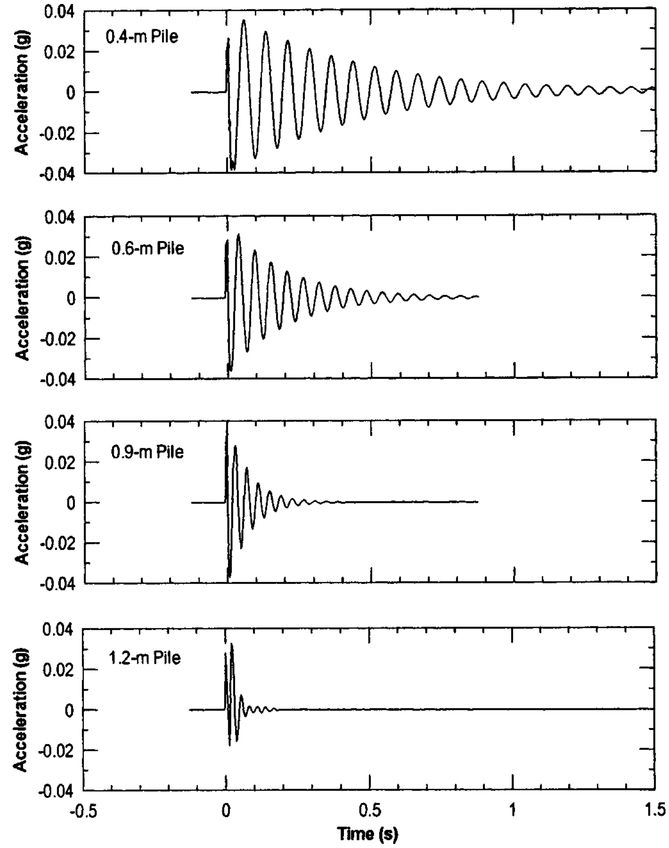

Fig. 7. Acceleration versus time for each CIDH pile in E-W direction from impact vibration tests

Experimental Test Results

The tests results obtained from both N-S and E-W directions are essentially the same; therefore, only the results for E-W direction are presented in this paper. Fig. 5 presents an example of the power spectrum obtained from the ambient vibration tests. The frequency response function of each pile obtained from the impact tests is shown in Fig. 6, which corresponds to the ratio of the pile acceleration to the force applied using a modal hammer. The peak of each curve represents the natural frequency of each system. As expected, the results indicate that the natural frequency of the system increases with increasing its stiffness. The results from both types of vibration testing are in good agreement. However, the impact test seems to be more reliable than the ambient vibration test, especially for very stiff piles because the amplitude of excitation is much higher than the noise level. A summary of the natural frequencies of the system from both types of vibration tests for all cases is presented in Table 3.

Fig. 7 shows a sample of acceleration amplitude-time curves of each pile obtained from the impact tests. The damping ratio of each soil-pile system was estimated by using two different methods: The logarithmic decrement method and the half-power bandwidth method. The damping ratio varies with pile diameter from approximately 3% for the 0.4-m pile to about 25% for the 1.2-m pile as summarized in Table 4. The results indicate that motion in the larger piles was damped out comparatively faster than in the smaller piles. This was due to the fact that the damping of the system contributed by the soil is mainly from the radiation damp-

Table 3. Summary of Natural Frequency of Soil-Pile System from Vibration Testing

		NATURAL FREQUENCY (Hz)			
		Ambient Vibration Test		Impact Test	
Pile diameter	Additional mass	North–South	East–West	North–South	East–West
0.4	No	13.9	13.6	13.7	13.4
	Yes	12.9	12.6	12.7	12.3
0.6	No	18.8	18.8	18.2	18.0
0.9	Yes	—	25.4	26.3	25.4
1.2 (No. 1)	No	—	—	34.5	34.5
	Yes	—	—	33.4	33.6
1.2 (No. 2)	No	—	—	33.0	31.8
	Yes	—	—	32.3	31.8

Table 4. Summary of Estimated Damping Ratio Using Logarithmic Decrement and Half-Power Bandwidth Methods (from Impact Test)

		DAMPING RATIO (%)			
		Logarithmic Decrement		Half-Power Bandwidth	
Pile diameter (m)	Additional mass	North–South	East–West	North–South	East–West
0.4	No	2.8	2.8	3.0	2.6
	Yes	2.8	3.2	3.3	3.0
0.6	No	4.5	4.6	5.6	4.9
0.9	Yes	9.9	8.6	10.8	9.1
1.2 (No. 1)	No	23.2	21.4	29.9	22.5
	Yes	24.1	21.9	29.6	22.3
1.2 (No. 2)	No	—	24.7	—	—
	Yes	—	24.9	—	—

ing which is a function of the contact area between soil and pile as well as excitation frequency (Dobry and Gazetas 1985).

Analysis of Natural Frequency

In order to verify the influence of pile diameter on initial modulus of subgrade reaction, a numerical model of the soil-pile system was developed as presented in Fig. 8. The pile was modeled by using a series of beam elements. The mass distributed throughout the pile element was idealized as a concentrated mass at the nodal points. The flexural rigidity of the pile E_pI_p was computed based upon the uncracked concrete section. Though minor cracking due to shrinkage may be present, these will have an insignificant effect on the stiffness of the pile (Hsu 1993). A summary of pile properties is presented in Table 5. Soil around the pile was modeled by using a series of linear Winkler springs evenly spaced at 0.15 m along the pile length. Two types of soil springs were considered in this study. One was developed based on Terzaghi's (1955) conclusion in which the modulus of the subgrade reaction is independent of the pile diameter (i.e., K_{ind}), and the other one is developed based on Carter (1984) and Ling's (1988) conclusions in which the modulus of the subgrade reaction is linearly dependent on the pile diameter (i.e., K_{dep}).

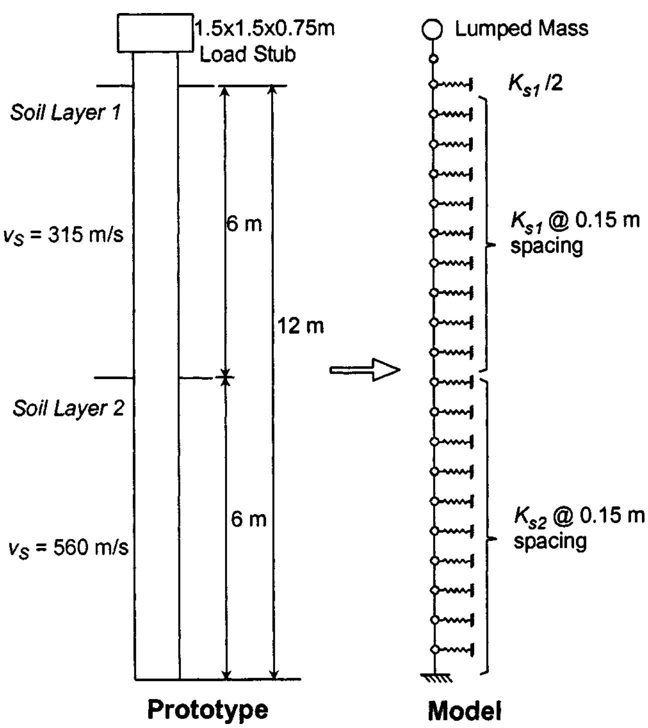

Fig. 8. Numerical soil-pile system model

The soil spring stiffness can be estimated from shear-wave velocity and equation modified from the Vesic's Equation [Eq. (1)]. The solution obtained from Eq. (1) is taken from the beam on elastic foundation case. Bowles (1988) suggested a modification on Eq. (1) in that the modulus of subgrade reaction K for the lateral loaded pile case should be doubled since the pile has soil contact with both sides. However, in reality, soil does not have contact all around the pile when the pile is subjected to lateral loading, but the friction developed at both sides of the pile can increase the overall soil resistance. The average value from lower-bound [Eq. (1)] and upper-bound solutions suggested by Bowles seems to be reasonable for analysis of laterally loaded pile. This is in agreement with what was proposed by Carter and Ling who found that the closest agreement in predicting the pile deflection was obtained by using a factor of 1.0 as

$$K=\frac{1.0E_s}{(1-\mu_s^2)}\left[\frac{E_sD^4}{E_pI_p}\right]^{1/12} \tag{3}$$

To account for the effect of pile diameter on initial modulus of subgrade reaction, Carter and Ling suggested a linear relationship between the modulus of subgrade reaction and the pile diameter, and then K based on Ling's concept can be expressed as

$$K=\frac{1.0E_sD}{(1-\mu_s^2)D_{\mathrm{ref}}}\left[\frac{E_sD^4}{E_pI_p}\right]^{1/12} \tag{4}$$

where $D_{\mathrm{ref}}=1.0$ m.

The soil elastic modulus E_s can be determined by

$$E_s=2\rho V_s^2(1+\mu_s) \tag{5}$$

where ρ = soil density; and V_s = shear-wave velocity.

From the above expressions, the initial horizontal modulus of subgrade reaction can be calculated. K_{ind} can be determined by using Eq. (3) and K_{dep} can be obtained by using Eq. (4). The soil spring stiffness then can be computed by multiplying the modulus of subgrade reaction with the soil spring spacing. A summary of soil spring stiffnesses based on diameter-dependent and diameter-independent modulus of subgrade reaction is given in Table 5.

Based on the numerical model of the soil-pile system, the mass matrix and system stiffness matrix can be simply formulated. The natural frequency of the system can then be calculated by using a modal analysis. In this study, *RUAUMOKO* (Carr 1998), a structural analysis program for inelastic dynamic analysis, was utilized

Table 5. Summary of Pile and Soil Properties Used in Natural Frequency Computation

Pile diameter (m)	Concrete strength (MPa)	Modulus of Elasticity		Pile flexural rigidity, E_pI_p (MN-m^2)	SOIL SPRING STIFFNESS			
					Based on K_{ind} Concept		Based on K_{dep} Concept	
		Steel (MPa)	Concrete (MPa)		1st layer	2nd layer	1st layer	2nd layer
0.4	30.3	$2.0E+05$	26,070	40	83	288	33	115
0.6	41.4	$2.0E+05$	30,443	238	83	288	50	173
0.9	40.7	$2.0E+05$	30,188	1,217	83	288	75	259
1.2	32.4	$2.0E+05$	26,944	3,530	83	288	100	346

to perform a modal analysis to predict the damped natural frequency and mode shape of the soil-pile system.

The computed natural frequencies based on the two different concepts on initial modulus of subgrade reaction are given in Table 6. The comparison between experimental and computational results was made by plotting the ratio of computed to measured natural frequency against the pile diameter as shown in Fig. 9. It is clearly seen that the results obtained from Terzaghi's concept (i.e., K_{ind}) give a much better agreement on natural frequency prediction over the range of diameter considered. From this, it is inferred that the initial modulus of subgrade reaction for weakly cemented sand appears to be independent of pile diameter. The computed natural frequency of the system based on K_{dep} appears to be significantly underestimated at diameters less than 1 m and slightly overestimated beyond that diameter. Since the Terzaghi's approach is consistent with the test results, the comparison between the two concepts can be extrapolated over a wider range of pile diameters by performing a parametric study. The results are shown by the dotted line in Fig. 9, and confirm these trends above and below the 1-m diameter.

Analysis of Damping

Gazetas (1991) proposed closed-form expressions to estimate the static stiffnesses and damping coefficients (i.e., K_{HH}, K_{MM}, K_{HM}, ξ_{HH}, ξ_{MM}, and ξ_{HM}) for flexible piles in a constant stiffness soil profile. With these equations, the horizontal static stiffness K_h and rotational static stiffness K_θ, of the pile can be estimated by using the following equations:

$$K_h=\frac{K_{HH}K_{MM}-K_{HM}^2}{K_{MM}-K_{HM}M/H} \tag{6}$$

$$K_\theta=\frac{K_{HH}K_{MM}-K_{HM}^2}{K_{HH}-K_{HM}H/M} \tag{7}$$

where K_{HH}, K_{MM}, and K_{HM} = static lateral, static rocking, and static swaying-rocking cross stiffnesses of the pile; H = horizontal force at pile head; and M = moment at pile head. The various components of the pile head impedances $\sigma_{\alpha\beta}$ can be determined as

$$\sigma_{\alpha\beta}=K_{\alpha\beta}(k_{\alpha\beta}+2\xi_{\alpha\beta}i) \tag{8}$$

where $\alpha\beta$ refers to various components (i.e., *HH*, *MM*, and *HM*); $K_{\alpha\beta}$ = static pile head stiffness; $k_{\alpha\beta}$ = dynamic stiffness coefficient, which is approximately equal to one (Gazetas 1991); and $\xi_{\alpha\beta}$ = damping coefficient.

The horizontal and rotational pile head impedances (i.e., σ_h and σ_θ) can be determined in the same fashion as Eqs. (6) and (7) by replacing $K_{\alpha\beta}$ terms with $\sigma_{\alpha\beta}$ terms. The σ_h and σ_θ are in complex form similar to Eq. (8). The horizontal and rotational dampings (ξ_h and ξ_θ) can then be simply calculated as the ratio between the imaginary part and two times of the real part. Wolf (1985) proposed the equation to estimate the equivalent damping of a single-degree-of-freedom (SDOF) system with the pile foundation as

$$\xi=\frac{\xi_{\mathrm{st}}+\xi_h\dfrac{k_{\mathrm{st}}}{K_h}+\xi_\theta\dfrac{k_{\mathrm{st}}h^2}{K_\theta}}{1+\dfrac{k_{\mathrm{st}}}{K_h}+\dfrac{k_{\mathrm{st}}h^2}{K_\theta}} \tag{9}$$

where ξ_{st} = damping for the structure; k_{st} = stiffness of the structure; and h = height of the SDOF structure.

The damping ratio of each pile was calculated using the above expressions and then compared with the measured one as presented in Fig. 10. The results show a good agreement between predicted and measured damping, though the measured values are somewhat higher than the computed values, particularly at high frequencies. This might be due to two possible reasons: (1) the analytical solutions used in this analysis were derived based upon single constant soil modulus, while the soil at the test site consists

Table 6. Summary of Measured and Computed Natural Frequency

Pile diameter (m)	Additional mass	NATURAL FREQUENCY (Hz)			Ratio of Computed to Measured Natural Frequency	
		Computed				
		K_{dep}	K_{ind}	Measured	K_{dep}	K_{ind}
0.4	No	11.3	13.6	13.6	0.84	1.01
	Yes	10.9	13.0	12.5	0.87	1.04
0.6	No	16.3	18.2	18.1	0.90	1.01
0.9	Yes	25.5	26.2	25.9	0.99	1.01
1.2 (No. 1)	No	36.1	34.0	34.5	1.05	0.99
	Yes	35.2	33.2	33.5	1.05	0.99
1.2 (No. 2)	No	33.9	32.0	32.4	1.05	0.99
	Yes	33.1	31.3	32.1	1.03	0.98

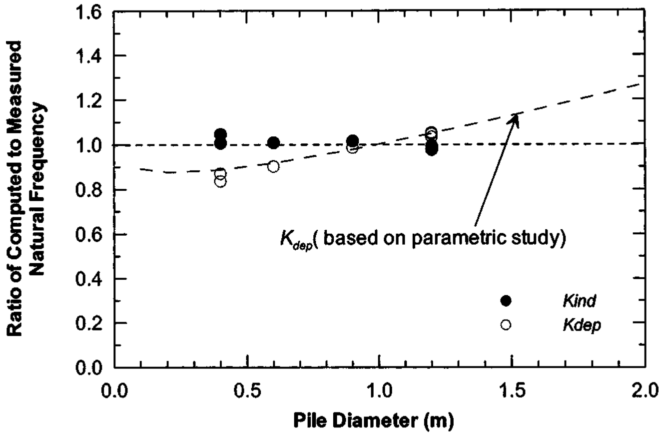

Fig. 9. Ratio of computed to measured natural frequency versus pile diameter

of two constant soil modulus layers system and (2) the damping ratio determined based on logarithmic decrement method at high frequency might have significant errors due to the limited number of peaks during free vibration. The computed damping ratios based on analytical solutions were lower than that measured damping ratios indicating that Gazetas's damping expressions are conservative for this site and test.

Conclusions

A series of vibration tests on different diameters of CIDH piles were performed to determine the natural frequencies and damping ratios of the soil-pile systems. A soil-pile system numerical model was developed to evaluate whether or not the pile diameter has an effect on the initial modulus of subgrade reaction. Two different concepts of initial modulus of subgrade reaction were implemented to model the soil resistance: (1) the initial modulus of subgrade reaction is independent of pile diameter and (2) the initial modulus of subgrade reaction is linearly proportional to pile diameter. A modal analysis was performed to compute the natural frequency of the systems. The results from experiment and analysis were then compared and the following conclusions can be drawn:

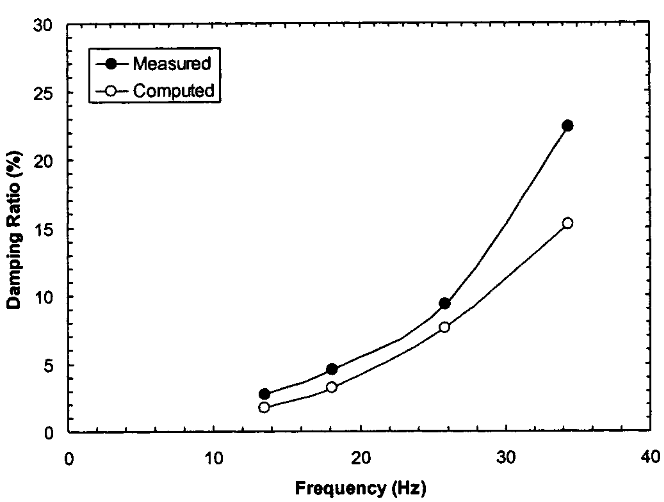

Fig. 10. Damping ratio versus frequency

1. The measured natural frequencies of a soil-pile system obtained from ambient vibration and impact tests were in good agreement. However, the ambient vibration test might not be suitable for piles of very high stiffness because the level of vibration is similar to the noise level;
2. The computed natural frequencies based on Terzaghi's concept were in good agreement with the measured natural frequencies obtained from vibration tests. Therefore, the initial modulus of subgrade reaction is apparently independent of pile diameter for the piles and soil tested;
3. The damping ratio varies with pile diameters from 3% for 0.4-m pile to 25% for 1.2-m pile. The damping ratio obtained from logarithmic decrement and half-power bandwidth methods agreed well. The damping ratio increases with increasing pile diameter. This was due to the fact that the damping of soil mainly came from the radiation damping, which increased with increasing contact area between the pile and soil as well as with the excitation frequency; and
4. The analytical solutions proposed by Gazetas (1991) gave a reasonable prediction on the damping ratio over the diameter range considered. The predicted damping ratios were slightly lower than the measured ones indicating that the damping ratios estimated using Gazetas expressions are conservative at least for this site and test.

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