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Research Article

A Full-Scale Experimental Study of the Vertical Dynamic and Static Behavior of the Pier-Cap-Pile System

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The bearing capacity evaluation of bridge substructures is difficult as the static loading test (SLT) cannot be employed for the bridges in services. As a type of dynamic nondestructive test technique, the dynamic transient response method (TRM) could be employed to estimate the vertical bearing capacity when the relationship between static stiffness and dynamic stiffness is known. The TRM is usually employed to evaluate single piles. For the pier-cap-pile system, its applicability should be investigated. In the present study, a novel full-scale experimental study, including both TRM test and SLT, was performed on an abandoned bridge pier with grouped pile foundation. The test included three steps: firstly, testing the intact pier-cap-pile system; then, cutting off the pier and testing the cap-pile system; finally, cutting off the cap and testing the single pile. The TRM test was repeatedly performed in the above three steps, whereas the SLT was only performed on the cap-pile system. Based on the experimental results, the ratio of dynamic and static stiffness of the cap-pile system was obtained. The results show that (1) in the low-frequency range (between 10 and 30 Hz in this study), the dynamic stiffness of the whole system is approximately four times of that of a single pile; (2) the ratio of dynamic and static stiffness of the cap-pile system tested in the study is approximately 1.74, which was similar to other tested values of a single pile; (3) to evaluate the capacity of similar cap-pile system and with similar soil layer conditions by TRM, the value of K_d/K_s tested in the study can be used as a reference.

1. Introduction

With the large-scale construction of transportation infrastructures in China, a large number of bridges have been built and put into use. Subsequently, the demand for bridge maintenance, disease diagnosis, and reinforcement is increasing year by year. As train speed increases on the passenger railway line and the axle load increases on the freight railway line, the capacity of the bridge substructures should be re-estimated. Different from the superstructure of the bridge, the pier foundation is usually buried in the soil layers, so it is difficult to identify the location of the disease by conventional detection methods. In addition, it is especially difficult to estimate the bearing capacity of the pier foundation quantitatively for the bridges in service. Generally, the estimation of bridge foundation can be divided into static and dynamic methods. However, the static loading test (SLT) cannot be employed for the existing bridge foundation in service although it is widely used to evaluate the bearing capacity of pile and building foundations [1–3]. The dynamic loads can be applied on the horizontal and vertical directions. To evaluate the horizontal state of the bridge substructure, the impact vibration test method is widely used. Nishimura [4] reported an impact vibration test method to examining the bridge substructures. Zhan et al. [5] proposed the concept of soundness index to evaluate the damages of bridge foundations based on the dynamic test method. Zhan et al. [6, 7] also developed the method for evaluating railway bridge substructures. Kien [8] evaluated a bridge substructure condition in Vietnam by using the impact vibration test method. In Chinese standard *Code for Rating Existing Railway Bridges* [9], the horizontal condition of the pier can be evaluated by the dynamic method, based on the horizontal vibration amplitudes of the pier top and its natural frequencies. Nevertheless, there is no ripe experience to evaluate the vertical condition of the bridge piers and foundations. The vertical state of the bridge substructure can be estimated by the dynamic transient response method (TRM). The TRM was originally proposed in 1970s [10] and then used to evaluate the pile integrity [11, 12] and estimate the pile capacity [13]. TRM analyzes both the velocity and force signals in the frequency domain. The velocity spectrum $V(f)$ is divided by the force spectrum $F(f)$ to determine the mobility or mechanical admittance spectrum [14]. Figure 1 illustrates a theoretical mobility response tested by TRM. In the low-frequency range, the ideal mobility response is linear with frequency. Accordingly, the slope of the portion of the mobility plot usually below 50 Hz defines the compliance or flexibility of the area around the test point for a normalized force input. The inverse of the compliance is the dynamic stiffness K_d of the structural element at the test point and can be defined as [11, 15]:

$$K_d(f) = \frac{2\pi f}{|V(f)/F(f)|}, \quad (1)$$

where $V(f)$ and $F(f)$ are the velocity and force signals in the frequency domain. The value of K_d is sensitive to the stiffness of the pile under compression. When the frequency approaches to 0 ($f \longrightarrow 0$), the value of the dynamic stiffness approaches to the static stiffness ($K_d \longrightarrow K_s$). In practice, however, the frequency of the dynamic impulse cannot be 0 Hz. Therefore, a coefficient α should be introduced here to describe the ratio between the dynamic and static stiffness: $\alpha = K_d/K_s$ [16, 17]. Then, the pile service capacity Q can be estimated by the following equation [13]:

$$Q = \frac{K_d S_a}{\alpha}, \quad (2)$$

where S_a is a guideline value of the pile settlement. The coefficient α is more an empirical value rather than a theoretical one. It should be noted that based on the dynamic and static stiffness, only the bearing capacity in service can be estimated but the ultimate carrying capacity state.

Based on the dynamic stiffness index, Ma et al. [18] deduced and solved an analytical model of mobility response under vertical harmonic loads, in which two inhomogeneous cross sections can be considered to analyze the dynamic stiffness of the integrated and defective piles. Liu and Ma [19] analyzed the parameter sensitivity on single pile and found that a reasonable dynamic stiffness can be used as an alert value for pile capacity. Ma et al. [13] also measured the dynamic stiffness for 680 bridge piles and found that an obvious positive correlation exits between the dynamic stiffness and bearing capacity of intact piles. To consider the influence of pile cap in the method, Chu et al. [20] performed a laboratory test and a numerical model to investigate the dynamic stiffness of the cap-pile system. In addition, the dynamic stiffness index was also employed to detect the void behind the tunnel linings [21].

If the dynamic stiffness index and the method introduced above are introduced to evaluate the vertical condition of the pier-cap-pile system or cap-pile system, two additional

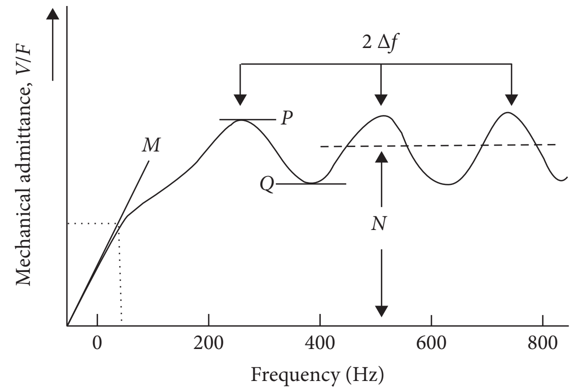


Figure 1: Theoretical mobility response spectrum tested by TRM [12].

problems should be addressed. Firstly, the value of the dynamic and static ratio should be determined. Secondly, the relation of the dynamic stiffness of the whole system and the single pile should be understood. In order to investigate the two problems above, in this paper, a full-scale experiment study was performed on a bridge pier-cap-pile system for the first time. Both dynamic and static stiffness were tested.

2. Test Outline

The test bridge pier was located outside the southeast gate of the Summer Palace in Beijing (Figure 2). These bridge piers were originally designed and constructed for a tram line. As the tram route has been adjusted, these bridge piers were abandoned. Therefore, it is a good opportunity to perform both the dynamic TRM test and static loading test on the pier-cap-pile system.

The height of test pier is 5.5 m above the ground surface. The dimension of the pier top is 2.4 m × 4.8 m. The bridge cap is supported by four friction piles. The length and diameter of each pile is 23 m and 1.2 m (Figure 3). The soil layers from the top to bottom are miscellaneous fill (L1), silty clay (L2), pebble (L3), silt (L4), gravel (L5), silty clay (L6), and pebble (L7). The detail soil layer information and their parameters are shown in Figure 4.

The cast in place bored piles were designed as frictional piles. Based on the empirical formula suggested by TB10093 [22], its vertical bearing capacity $[P]$ can be estimated by the following equation:

$$[P] = 0.5 \times U \times \Sigma f_i l_i + m_0 A[\sigma], \quad (3)$$

where U is the circumference of the pile, with the value of 3.768 m; A is the area of pile bottom, with the value of 1.1304 m^2; l_i and f_i are the depth and ultimate friction force for each soil layers with the code suggested value in Figure 4; m_0 is a reduction coefficient of the supporting force on the pile bottom, with a suggested empirical value of 0.3 in the calculation; and $[\sigma]$ is the capacity of the ground around the pile bottom with the value of 1092.8 kPa. Accordingly, the designed allowable capacity for each single pile can be calculated as 6180 kN.

The test included three steps (Figure 5): Step 1, testing the whole pier-cap-pile system under only dynamic loads; Step 2, testing the cap-pile system under both dynamic and static loads; and Step 3, testing the single pile under only

Figure 2: The test bridge pier-cap-pile system.

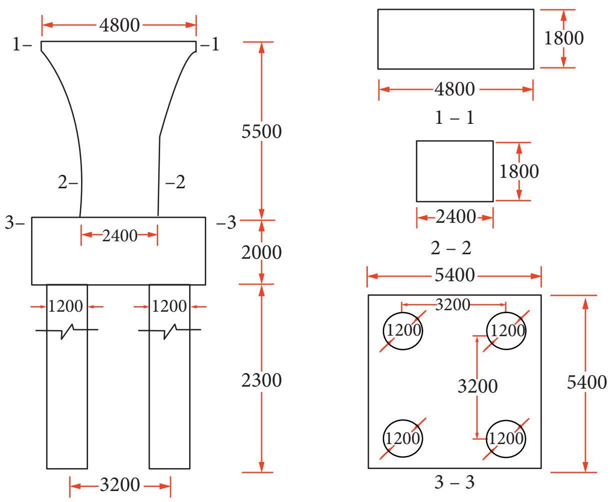


Figure 3: Dimension of the tested pier-cap-pile system (unit: mm).

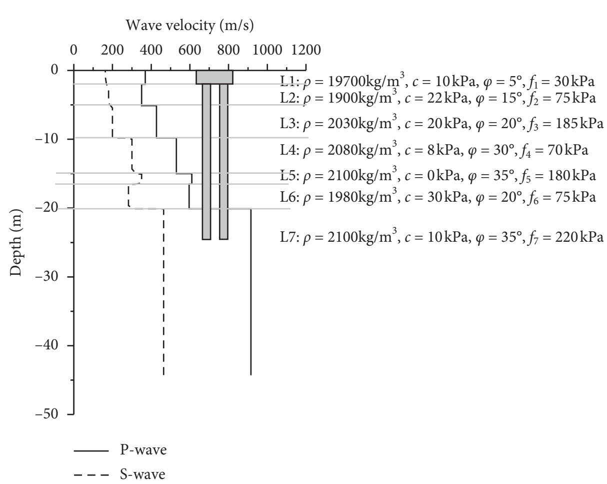


Figure 4: Soil layers and their parameters around the test bridge foundation.

dynamic loads. The dynamic test employed TRM and the static test employed SLT. In the TRM test, velocity sensors were symmetrically arranged, as illustrated in Figure 5.

3. Dynamic Test by TRM

3.1. Test Pier-Cap-Pile System. In the TRM test, a drop weight setup with a total weight of 200 kg was developed

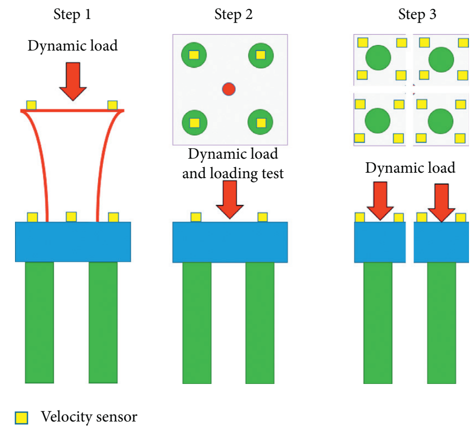


Figure 5: Three test stages.

to impact the center of pier top. A sketch of the setup is illustrated in Figure 6. The drop weight setup was firstly lifted by crane onto the pier top (Figure 7). A typical time history of the tested force is shown in Figure 8. The time interval of the impact load was set as 1.041×10^{-4} s. The maximum amplitude of the impact load was 2.13×10^{5} N, and the duration of the impulse is approximately 17.5 ms. The sampling time interval of the velocity response was set as 3.16×10^{-4} s. To calculate the vibration admittance, the varied-time-base technique [23] was employed, which can deal with the different sampling time intervals between force and response signals. The velocity sensors were installed on both the pier top and the cap top, and then based on equation (1), two types of dynamic stiffness can be calculated: in the first type of dynamic stiffness, both of the input force and output response were at the pier top, whereas in the second type of dynamic stiffness, the input force was at the pier top and the output response was at the cap top. The impact tests were repeated at least five times and the final dynamic stiffness was averaged by all calculated results.

In this test step, the impact load was applied on the pier top, and the velocity sensors were installed on both of the pier top and the cap top. Figure 9 illustrates the tested dynamic stiffness on the pier top and the cap top, which were calculated by the velocity spectra on the pier top/cap top and the force spectrum by Equation (1). The values of dynamic stiffness plotted in bold lines were averaged by five samples also plotted in the figure. It can be observed that the values between 10 and 20 Hz remain stable. As illustrated in Figure 1, the ideal mobility response is linear with frequency in a very low-frequency range. Then, the ideal dynamic stiffness should be a constant in low frequencies. However, due to the limited low-frequency impact energy provided by the equipment used in this study, the dynamic stiffness below 5 Hz was very small. Due to the vibration attenuation through the pier, the vibration responses on the cap top were smaller than those on the pier top when the impact force acts on the pier top. Then, according to Equation (1), the low-frequency dynamic stiffness on the cap top should be larger than that on the pier top, which can be observed in Figure 9.

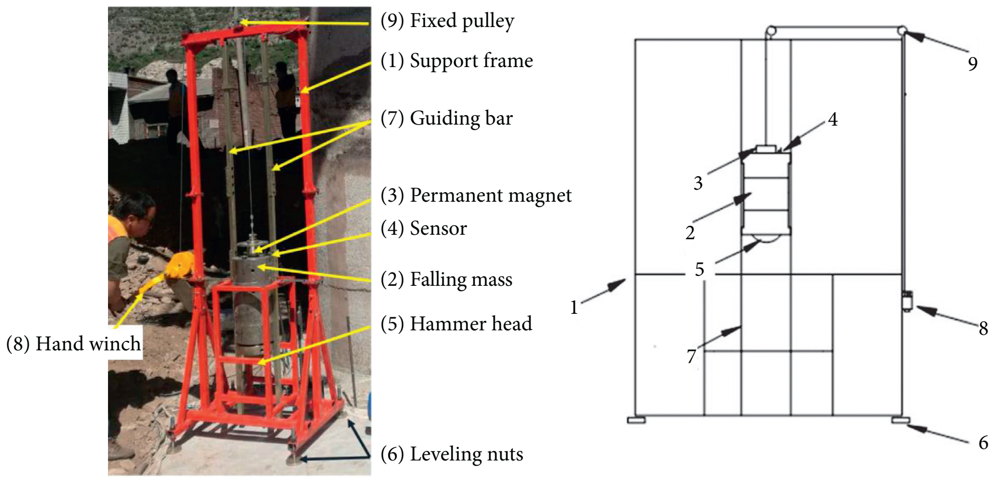


Figure 6: Sketch of the drop weight setup.

(a) (b)

Figure 7: The drop weight setup was (a) lifted by crane and (b) onto the pier top.

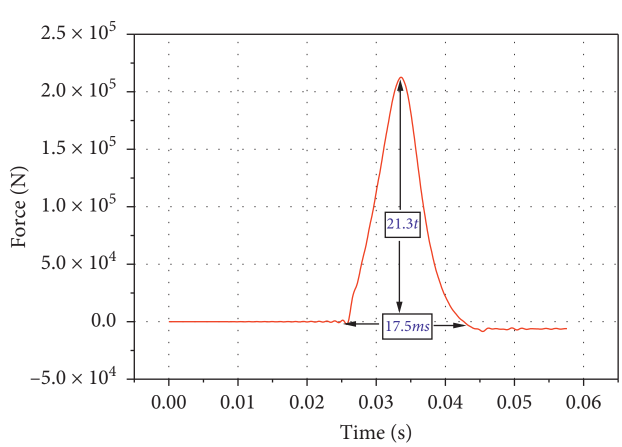


Figure 8: Typical time history of tested force.

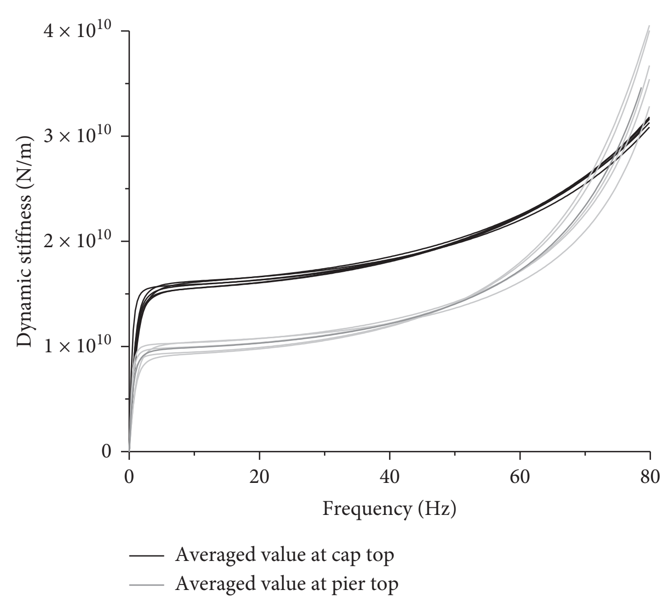


Figure 9: Dynamic stiffness on the pier top and the cap top.

3.2. Test Cap-Pile System and Single Pile. After finishing the test of Step 1, the bridge pier was cut off. Then the dynamic test of Step 2 was performed on the cap-pile system (Figure 10). Finally, the cap was also cut off, and then the

Figure 10: Impact on the cap-pile system by the drop weight setup when the pier was cut off.

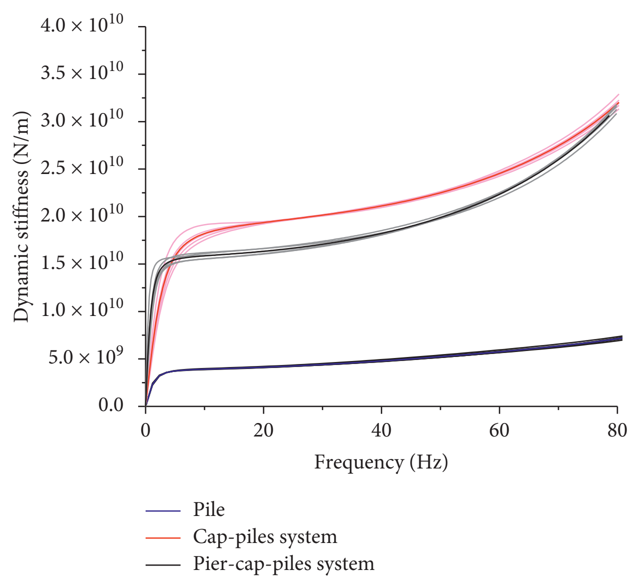
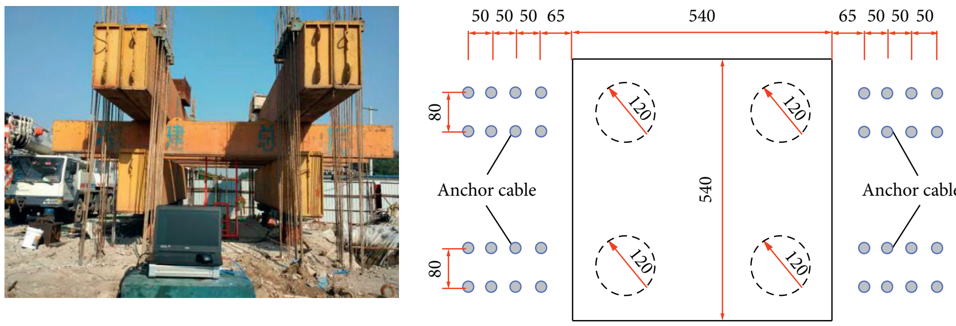


Figure 11: Comparison of the dynamic stiffness of three-step test results.

Table 1: Values of dynamic stiffness at different frequencies.

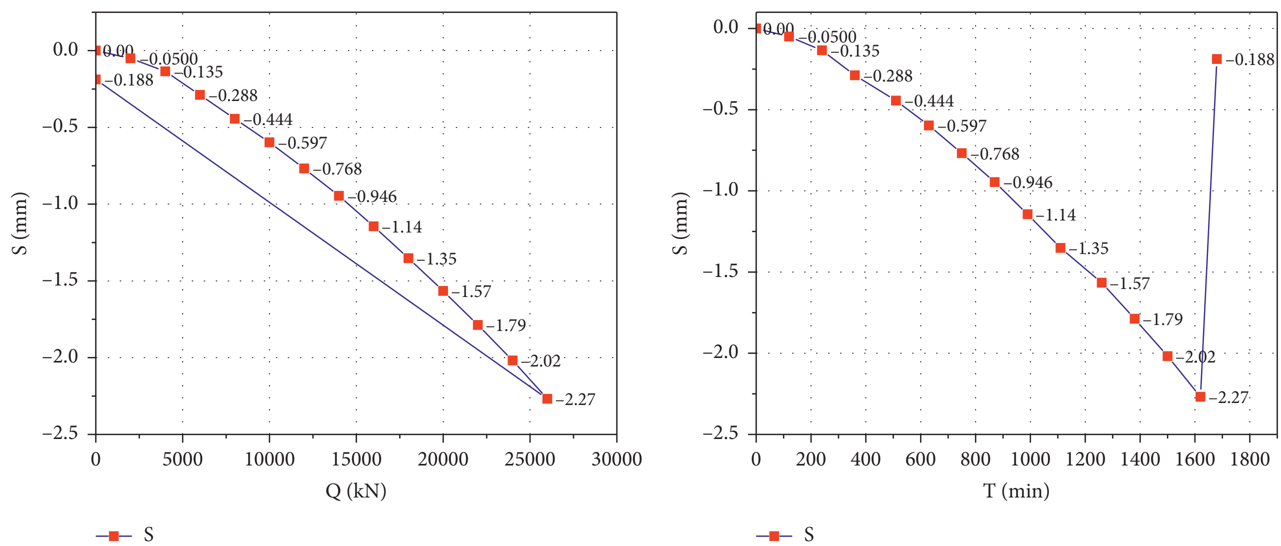
Frequency (Hz)	Dynamic stiffness ($\times 10^{10}$ N/m)			Ratio to the value of single pile		
	Pier-cap-pile system	Cap-pile system	Single pile	Pier-cap-pile system	Cap-pile system	Single pile
10	1.631	1.821	0.466	3.499	3.907	1
15	1.660	1.896	0.491	3.382	3.863	1
20	1.690	1.940	0.522	3.241	3.720	1
25	1.727	1.977	0.560	3.086	3.532	1
30	1.770	2.012	0.603	2.935	3.337	1

dynamic test of Step 3 was performed on the single pile. A comparison of dynamic stiffness is illustrated in Figure 11. The values of dynamic stiffness plotted in bold lines were averaged by five samples also plotted in the figure. Between 10 and 80 Hz, the values of dynamic stiffness of the cap-pile system were the largest and the values of the single pile were the smallest. Table 1 lists the values of dynamic stiffness at some low frequencies. The ratios of the value of single pile were also calculated. If the designed length, diameter, and soil condition are the same for the four piles, they can evenly share the upper loads from the cap. Then, the capacity of cap-pile system is approximately four times of it of each single pile. If the cap-pile system is regarded as a rigid system, they have the same settlement. Theoretically, the static stiffness of the cap-pile system is also four times that of each single pile. In Table 1, one can found that when the frequency becomes lower, the dynamic stiffness ratio of the cap-pile system and the single pile is approaching to 4 when the frequency f is approaching 0 Hz. As the frequency approaches to 0 ($f \longrightarrow 0$), the value of the dynamic stiffness approaches to the static stiffness ($K_d \longrightarrow K_s$), and the test result was consistent with the theoretical result. That is, the



(a) (b)

Figure 12: The loading test equipment (a) and the arrangement of anchor bolts around the cap (unit: cm) (b).



(a) (b)

Figure 13: The Q-S (a) and T-S (b) curves obtained by SLT.

Table 2: Test results of SLT.

Step	T (min)	Q (kN)	S (mm)
0 (initial)	0	0	0
1	120	2000	0.05
2	240	4000	0.13
3	360	6000	0.29
4	510	8000	0.44
5	630	10000	0.6
6	750	12000	0.77
7	870	14000	0.95
8	990	16000	1.14
9	1110	18000	1.35
10	1260	20000	1.57
11	1380	22000	1.79
12	1500	24000	2.02
13	1620	26000	2.27
14 (unloading)	1680	0	0.19

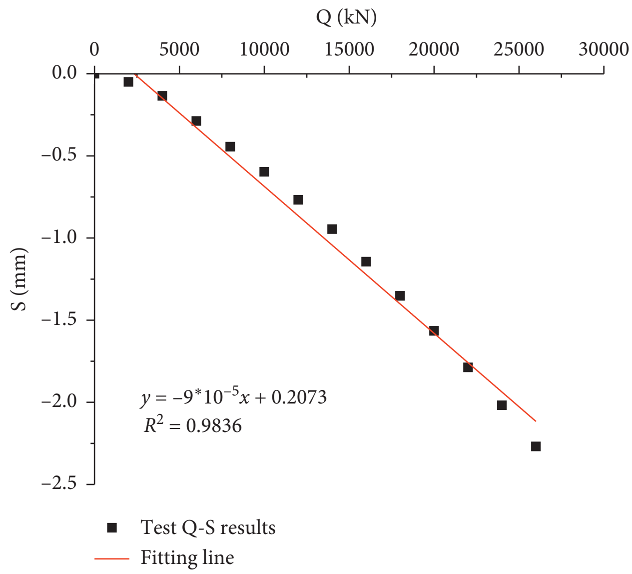


Figure 14: Linear fitting of static stiffness for the cap-pile system.

Table 3: K_d/K_s of the cap-pile system or single pile at different frequencies.

Tested cap-pile system or single pile		Averaged K_d between 10 and 30 Hz (N/m)	K_s (N/m)	Averaged K_d/K_s
Cap-pile system	Tested in the study	1.93×10^{10}	1.11×10^{10}	1.74
	Pile #1 tested in [19]	3.66×10^{8}	1.41×10^{8}	2.60
	Pile #2 tested in [19]	3.49×10^{8}	1.61×10^{8}	2.17
Single pile	Pile #1 tested in [24]	2.74×10^{9}	1.25×10^{9}	2.19
	Pile #2 tested in [24]	1.97×10^{9}	9.90×10^{9}	1.99

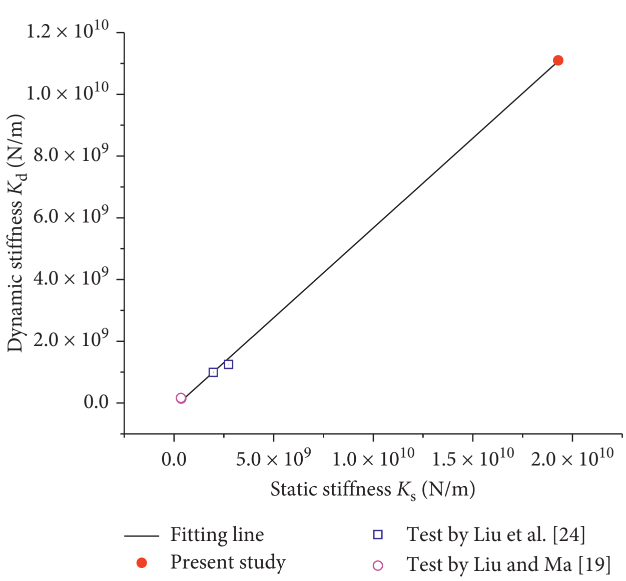


Figure 15: Comparison of dynamic and static stiffness.

value of dynamic stiffness in the low-frequency range can reflect the capacity of the whole system to some extent.

4. SLT on the Cap-Pile System

After cutting off the bridge pier, SLT on the cap-pile system was performed in Step 2. The test followed the slow maintenance loading method, with an actual maximum loading of 2600 t and total 13 loading steps. In order to provide loading reaction force, the anchor bolts were installed around the cap and the steel beams were constructed. The static load was applied by the high pressure oil pump and jack hydraulic system. The loading test equipment and the arrangement of anchor bolts are illustrated in Figure 12.

The SLT lasted 28 hours, including 13 loading steps and one unloading step. The tested *Q-S* (load-settlement) and *T-S* (time-settlement) curves are illustrated in Figure 13. The details of the test data are listed in Table 2. It can be observed from the results that with the increase of the loading step, the settlement kept increasing. When the force reached 2600 t, the maximum settlement reached 2.27 mm and residual settlement was only 0.19 mm (8.3%) after fully unloading. In addition, there was no obvious inflexion or sudden change was observed in the *Q-S* curve. Accordingly, it can be concluded that during the whole loading test, the cap-pile-soil system was in the elastic state.

5. Ratio of K_d/K_s and Discussion

By linear fitting of the *Q-S* curve (Figure 14), the static stiffness can be calculated from the slope of the linear fitting as 1.11×10^{10} N/m. As from Table 1, the average dynamic stiffness for the cap-pile system between 10 and 30 Hz was 1.93×10^{10} N/m, and the averaged value of K_d/K_s was 1.74, as listed in Table 3.

The ratio of K_d/K_s discussed in Table 3 was tested for the cap-pile system. Then, it is interesting to compare it with the tested K_d/K_s for single piles. Table 3 and Figure 15 also compare the results of four single piles tested by the authors. In the research by Liu and Ma [19], both SLT and TRM test were performed on two piles with a diameter of 0.6 m and length of 11.2 m. In the research by Liu et al. [24], both SLT and TRM test were performed on two piles with a diameter of 1.0 m and lengths of 24.7 m and 28.6 m, respectively. The static stiffness was obtained from the initial linear slope of the tested *Q-S* curve, while the dynamic stiffness was averaged by the K_d between 10 and 30 Hz. It can be observed that all the three groups of test have the similar linear relationship, and the ratios of K_d/K_s range between 1.74 and 1.76. Accordingly, the TRM and the value of K_d could also be employed to estimate the capacity of the cap-pile system by Equation (2).

6. Conclusions

In the present study, a dynamic test by TRM was performed on the pier-cap-pile system, cap-pile system, and single piles. Then, SLT was also performed on the cap-pile system. The following conclusions can be drawn.

(1) In the low-frequency range (between 10 and 30 Hz in this study), the dynamic stiffness ratio of the cap-pile system and the pile was approaching to 4. As the capacity of cap-pile system should be four times that of each single pile, the dynamic stiffness can reflect the foundation capacity.

(2) The averaged ratio of K_d/K_s tested in the study is approximately 1.74 for the cap-pile system, which was similar to value of single piles.

(3) To evaluate the capacity of similar cap-pile system and with similar soil layer conditions by TRM, the value of K_d/K_s tested in the study can be used as a reference.

Data Availability

The data used to support this study are available upon request to the corresponding author.

Conflicts of Interest

The authors declare no conflicts of interest.

Acknowledgments

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