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Analysis of In Situ Laterally Loaded Tests on Caisson Foundations

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ABSTRACT: In this study, three cases of in-situ laterally loaded tests of caisson foundations are simulated. The differences of these three cases are the application position of lateral loads and the scoured depth of the foundations. In the numerical model, a Winkler beam model is adopted. The Winkler beam model utilizes the beam element to model the caisson and six types of springs to simulate the horizontal subgrade reaction, the vertical shear stress on the front of the caisson, the horizontal shear stress and the vertical shear stress on the sides of the caisson, and the normal stress and the horizontal shear stress on the base of the caisson. The simulation results show that the model can properly capture the responses of the caisson foundations under lateral loading.
Keywords: Caisson foundations, *In situ* tests, Lateral loads, Winkler model.

1. INTRODUCTION

Caisson foundations are a commonly used foundation type of bridge structures. They generally have a large resistance due to a large cross section and a large embedded depth. Compared to vertical loading, lateral loading often governs final design due to a relatively low lateral capacity. In-situ load lateral tests on caisson foundations are very limited although the actual bearing behavior of foundations is essential to verify the appropriateness analysis models in design.

In 2010, National Center for Research on Earthquake Engineering (NCREE) of Taiwan conducted a series of in-situ loading tests on the old Niu-Dou Bridge in Ilan Country, Taiwan. The differences of these cases were the applied position of lateral loads and the scoured depth of the foundations. For these tests, Chiou et al. (2012) used a Winkler beam model to simulate the foundation load test. In their study, they proposed a method to determine the properties of Winkler springs for gravel ground. Chang et al. (2014) used nonlinear fiber elements for the nonlinear flexural behavior of columns to simulate the column load tests.

In order to investigate the foundation responses in the column load tests in more detail, this study conducts numerical simulation to analyze these tests.

2. OVERVIEW OF TESTS

The old Niu-Dou Bridge had two independent bridge structures, as shown in Figure 1. Both bridge structures had seven spans. In the test program, columns P2, P3, P4, and P5 of the left-side bridge were tested, as displayed in Figure 2. The columns were supported by caisson foundations. The foundations generally had a diameter of 4m and a length of 12m. The three column load tests were conducted on P2, P3, and P4 and the foundation load test on P5. The location of load application on P2, P3, and P4 were 9.54, 10.59, and 10.37 m, respectively, measured from the foundation top. The column sections had a diameter of 1.8 m. These columns had different foundation embedment. The foundation of column P2 was slightly exposed with a length of about 0.4 m. The P3 foundation was exposed with a length of 1.2 m. The P4 foundation had a larger exposed length of 4.0 m. The P5 foundation had an exposed length of about 1 m.

For the column load tests, cyclic displacement controlled load tests were performed on columns P3 and P4. A pseudo-dynamic test and a single-cycle pushover test were conducted orderly on column P2. A load-controlled monotonic lateral load test was carried out on the caisson foundation of column P5.

3. SOIL CONDITIONS

The soil at the site, within a depth of 20 m, mainly consists of gravels and cobbles with some sands or silts. The Standard Penetration Test (SPT) blow counts were generally larger than 50. The water level was about 3 m below the soil surface during the lateral load tests. The specific gravity of the soil solids was 2.75. The field density tests and the sieve analysis indicated that the moist unit weight, the water content, and the void ratio of the soil were 22.66 kN/m3, 10%, and 0.31, respectively. The grain-size distribution of the soil indicated the gravel content of about 89%, an effective size D10 of 2 mm, a coefficient of uniformity Cu of 57.5, and a coefficient of curvature Cc of 1.5. The soil is classified as well-graded gravel (GW). According to field direct shear test, the friction angle and the cohesion for the peak shear strength were 37° and 9.81 kN/m2, respectively, and the friction angle and the cohesion for the residual shear strength were 31.8° and 0 kN/m2, respectively.

Figure 1 Niu-Dou Bridge (Chiou et al., 2012)

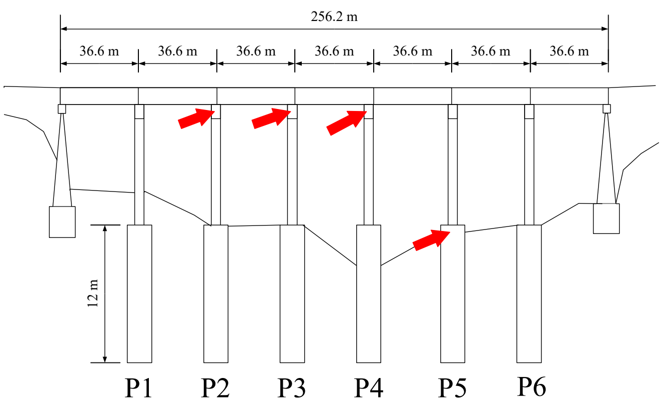

Figure 2 Test specimens of Niu-Dou Bridge

4. THE ANALYSIS MODEL AND SIMULATION

This section further analyzes the column tests through numerical simulation to clarify the behavior of the foundations in the tests in more detail.

4.1 Analysis model

Chiou et al. (2012) applied a Winkler beam model to simulate the lateral response of P5 foundation under loading. In this study, their model was used to simulate the foundation behavior in the column load tests. The Winkler-beam model adopted uses beam elements to simulate the caisson body and spring elements to simulate the soil reactions. To well simulate different soil reactions around the caisson, a six-component Winkler spring model, as shown in Figure 5, is adopted. Six types of springs are utilized to simulate the different components of the soil reactions acting on the caisson with an equivalent rectangular section of width B_e (perpendicular to the direction of lateral loading) and length L_e (parallel to the direction of lateral loading). Springs k_H and k_{SVB} represent the horizontal subgrade reactions and vertical shear stress levels on the front of the caisson, respectively; springs k_{SHL} and k_{SVL} represent the horizontal shear stress and the vertical shear stress on the sides of the caisson, respectively; springs k_V and k_S represent the normal shear stress and the horizontal shear stress on the base of the caisson, respectively. Since the test site was on gravelly soil, Chiou et al. (2012) proposed a method which modified the load-displacement responses from the plate loading tests and the direct shear tests to determine the load-deformation characteristics of the springs. The details for the method can be found in Chiou et al. (2012).

Beam elements are used to model the columns. In order to simulate nonlinear flexural behavior of the columns, the distributed plastic hinge model is applied. According to Chiou et al., (2009), the properties of plastic hinges are set based on the moment-curvature curves of the columns. Based on the above settings, the analysis models for the column load tests using SAP2000 program (Computer & Structures, 2002) are displayed in Figure 4.

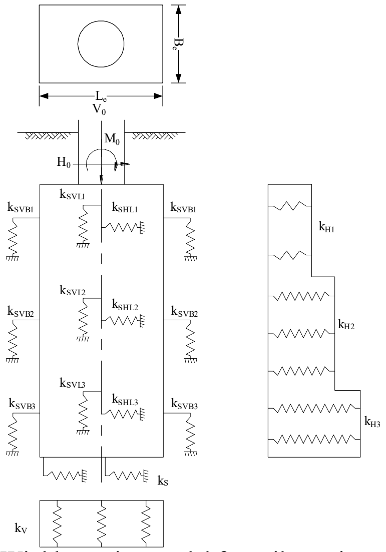

Figure 3 Winkler spring model for soil reactions on caisson [adapted from Japanese Specifications of Highway Bridges (JRA 2012)]

4.2 Analysis results of P2 column load test

In this test, the exposed depth of the foundation was about 0.4 m (3.3% caisson length), and the point of load application on the column was 9.54 m above the ground surface.

The results of analysis are shown in Figs. 5 and 6 for the lateral displacement of the column and the foundation, respectively. In Figure 5, the stiffness of the simulation curve is a little stiffer than that of the experimental one. This is because the column was slightly damaged due to the former pseudo-dynamic test on it. Figure 7 shows the comparison between the simulation curve and the pseudo-dynamic test: the initial portion of the simulated curve is very close to that of the pseudo-dynamic test. In Figure 6, the trend of the simulated foundation displacement is consistent to that of the measured displacement: the simulation curve is a little softer than that of the experimental one.

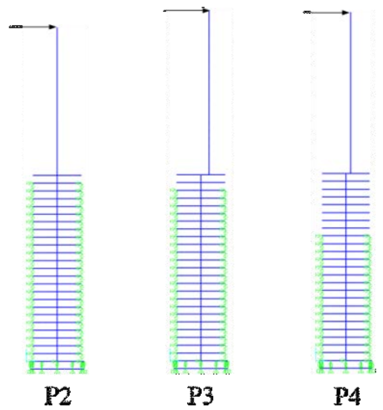

Figure 4 Analysis models for the column load tests

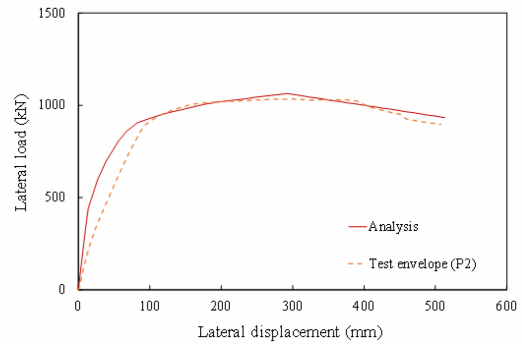

Figure 5 Load-displacement curve at the top of pier of P2

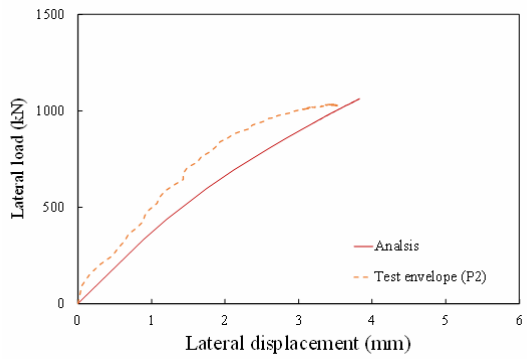

Figure 6 Load-displacement curve at the top of foundation of P2

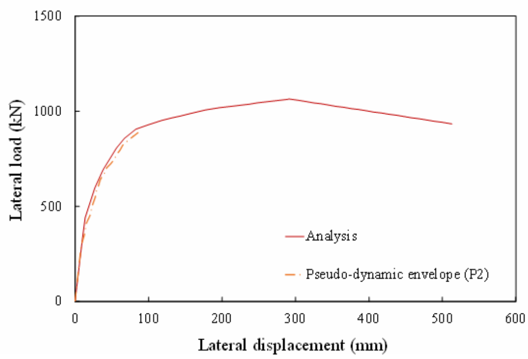

Figure 7 Load-displacement curve at the top of pier of P2

4.3 Analysis of P3 column load test

In P3 test, the point of load application on the column was 10.59 m above the ground surface. The foundation embedment condition was similar to P5 foundation (exposed length of 1.2 m (10% caisson length)). Therefore, the foundation analysis model used for this test is close to that for P5 test.

Figure 8 displays the analysis result of the lateral displacement of the column. Compared with the experimental curve, they are in good agreement. Figure 9 only displays the simulated load-foundation displacement curve because of some measurement errors on foundation displacement.

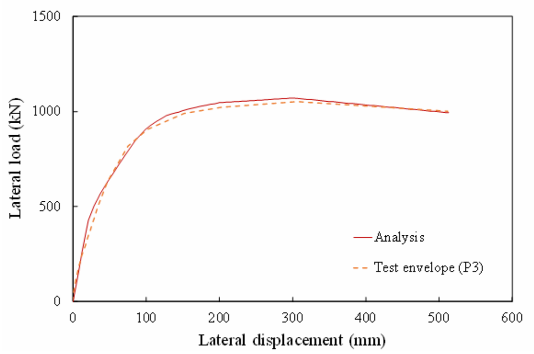

Figure 8 Load-displacement curve at the top of pier of P3

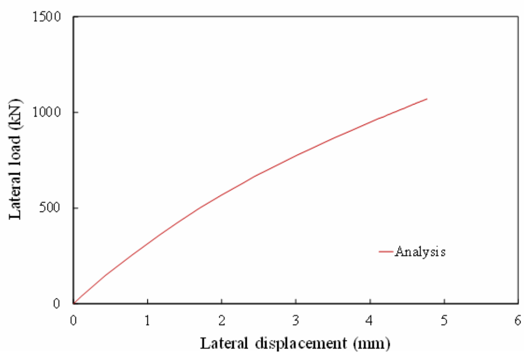

Figure 9 Load-displacement curve at the top of foundation of P3

4.4 Analysis of P4 column load test

In P4 test, the exposed depth of the caisson was 4 m (1/3 length of caisson). The point of load application on the column was 10.37 m above the foundation top.

The analysis results of P4 test for the lateral displacement of the column and the foundation are shown in Figs. 10 and 11, respectively. In Figure 10, the simulation curve is close to the experimental one. In Figure 11, the trend of the simulated foundation displacement is consistent to that of the measured.

From the above simulations, it is seen the model adopted can reasonably capture the response of the behavior of the columns and their foundations in the tests.

With the analysis results, Figure 12 compares the lateral load-displacement curves of columns P2, P3, and P4. As observed in the test results, the overall curves are close. Their stiffnesses are a little different, but their strengths are close. However, as shown in Figure 13, their foundations have different responses. It can be seen in the figure that at the same lateral load P4 foundation has the largest lateral displacement while P2 and P3 foundations have close lateral displacements. P4 foundation has the lowest lateral stiffness and strength because of the largest exposed length of foundation. As those curves are compared with that of P5 foundation, the lateral stiffness is the largest because of a pure horizontal load on the top of the caisson and a smaller exposed length.

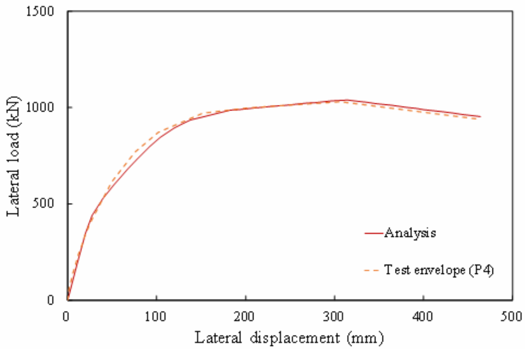

Figure 10 Load-disp. curve at the top of pier of P4

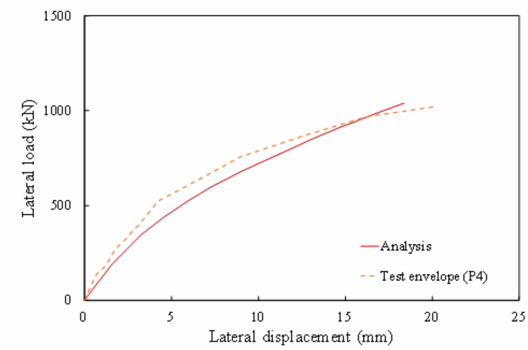

Figure 11 Load-displacement curve at the top of foundation of P4

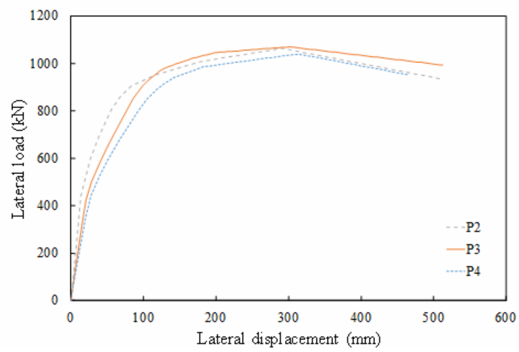

Figure 12 Simulated load-displacement curves of columns P2, P3, and P4

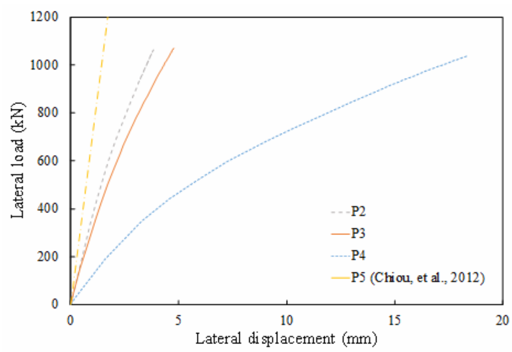

Figure 13 Simulated load-foundation displacement curves of P2, P3, and P4

5. CONTRIBUTIONS OF SOIL REACTIONS

In addition to the horizontal soil reaction in front of foundation k_H, the shear reactions around the caisson (k_{SVB}, k_{SHL} and k_{SVL}) and the bottom resistance also contribute lateral resistance (k_S and k_V). Figure 14 (a) and (b) compares the horizontal resistance contribution of P2 and P4. The resistance of the caisson to the horizontal loading mainly comes from the horizontal resistance of the soil in front of the caisson. The horizontal side shear resistance of the caisson contributes part of the resistance; however, its contribution reaches a maximum at a small displacement. The shear resistance on the base of the caisson acts in the reverse direction to the lateral loading due to the different directions of movement between the upper and the lower parts of the caisson. It is noted that for P4 foundation, with large exposed length, the lateral resistance in front of the caisson even exceeds the total applied lateral load to cancel out the reverse base shear.

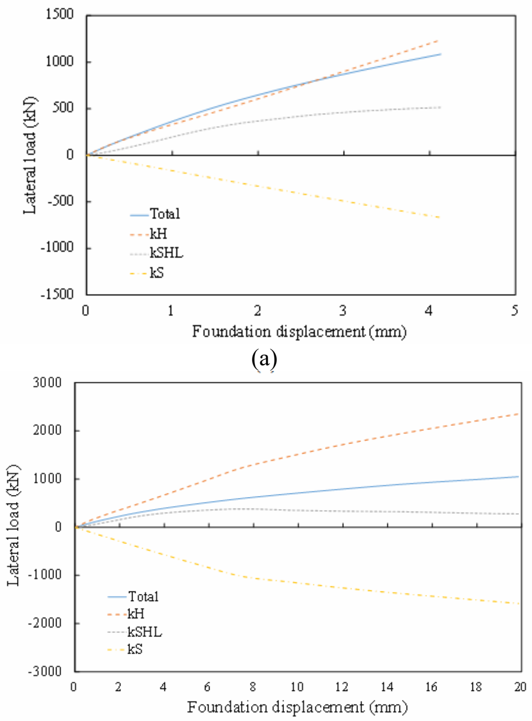

Figure 14 Soil reaction contributions to horizontal loading at different lateral displacements: (a) P2, (b) P4

For the moment resistance, with respect to the rotation center, Figure 15 (a) and (b) compares the moment resistance contributions of P2 and P4 foundations, respectively. All the soil reactions around the caisson contribute to resist the moment loading. The soil reactions from the front of caisson provide the maximum moment resistance, about 50-60% for both P2 and P4 foundations. The horizontal shear resistance along the caisson shaft k_{SHL} provides the secondary largest resistance, about 25-35% and 10-22% of total applied overturning moment for P2 and P4 foundations, respectively. The k_{SHL} on P4 has a smaller contribution due to a lesser embedded length. The moment resistances from k_S are about 12% and 16% for P2 and P4 foundations, respectively. The moment resistance from k_V are about 4% and 12% for P2 and P4 foundations, respectively. With increasing exposed length, the base resistance provides more moment resistance. The vertical shear reaction ($k_{SVB}+k_{SVL}$) also provides about 8-10% moment resistance for both P2 and P4 foundations although it does not directly provide resistance to horizontal loading.

6. CONCLUSIONS

In this study, the six-component foundation model is used for the foundation behavior and the distributed plastic hinge model for the nonlinear flexural pier behavior. The behavior of the foundations and pier in the column load tests with different degrees of foundation exposure are reasonably captured.

The lateral responses of the foundations are influenced by the foundation embedment and the location of load application. P4 foundation has the lowest lateral stiffness and strength because of the largest exposed length of foundation. The lateral stiffness of P5 foundation is the largest because of a pure horizontal load on the top of the caisson and a smaller exposed length.

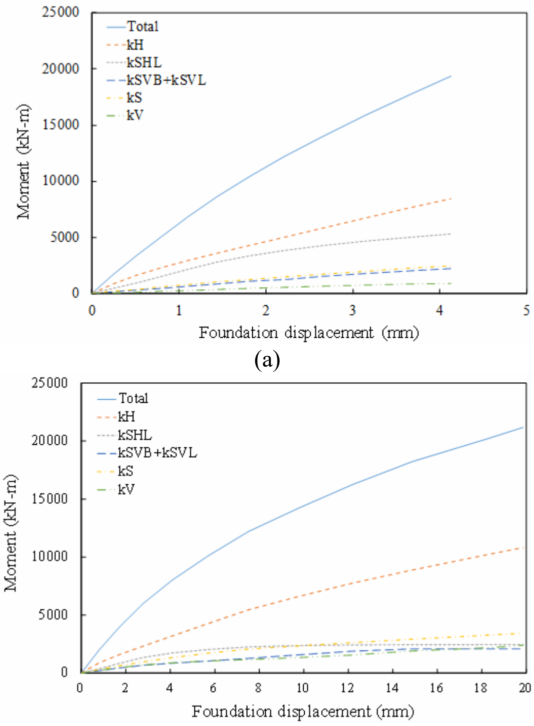

Figure 15 Soil reaction contributions to moment at different lateral displacements: (a) P2, (b) P4

7. REFERENCES

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