Testing and analysis of a laterally loaded bridge caisson foundation in gravel

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Abstract

This paper presents the results of an in situ lateral load test on a caisson-type foundation of the old Niu-Dou Bridge in Ilan County, Taiwan. The caisson was 12 m long and had circular cross-sections whose diameters were 5 m in the upper portion and 4 m in the lower portion. The test site was located on soil with a high gravel content. A site investigation, including laboratory and field tests, was carried out. A six-component Winkler-beam model was applied to simulate the caisson response in the lateral load test. To determine the nonlinear properties of the Winkler springs, a method based on large-scale geotechnical field testing was proposed. With this method, the soil springs could be properly set and the Winkler-beam model could reasonably capture the lateral behavior of the caisson foundation.

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1. Introduction

Caissons and piles are commonly used types of deep foundations in bridge engineering. They are deeply embedded in the soil to support the weight of the structure and to resist the lateral loads transmitted from the structure, e.g., lateral soil pressure, wind loads, earthquake loads, etc.

In designing a bridge foundation, the lateral response of the foundation is of considerable importance, because it often governs the final design. For obtaining the lateral response of deep foundations, many analysis methods have been developed, ranging from complex finite element models to simple beam-spring models. In engineering practice, the Winkler-beam model is a popular method for analyzing the behavior of piles under lateral loads (e.g., Hetenyi, 1946; Reese et al., 1974; Kramer and Heavey, 1988; Chiu and Chen, 2007). In this model, the piles are simulated by beam elements and the lateral soil reactions are simulated using independent horizontal spring elements (i.e., Winkler springs). Similar modeling concepts are also applied to caissons (e.g., Japan Road Association, 1990; Railway Technical Research Institute, 1997; Gerolymos and Gazetas, 2006a,b). Compared to piles, caissons generally have relatively large cross-sections, but shallow embedment, such that they behave like a rigid foundation and the soil within the embedded depth of the caissons is influenced by the caisson movement. Therefore, the modeling of a laterally loaded caisson requires more types of soil springs to simulate the different sources of soil resistance, including the normal stress and the shear stress along the perimeter of the caisson.
and the shear force on the base of the caisson, creating further difficulty in determining the soil spring properties. In addition, in situ lateral load tests on the caisson are essential for examining the applicability of the analysis models and the associated spring properties (Macklin and Chou, 1989; Yoshii et al., 1989).

In 2010, the 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 County, Taiwan. The test site was located on soil with a high gravel content. The tests included three cyclic pushover tests on three of its pier columns and a monotonic lateral load test on one of its caisson-type pier foundations. The aim of these tests was to investigate the seismic capacity of actual bridges and to examine the applicability of existing analysis methods.

In this paper, we focus on the results of the aforementioned lateral caisson load test. A six-component Winkler-beam model is used to analyze the response of the test caisson; the performance of the model is examined as well. To accurately determine the spring properties, a method based on large-scale geotechnical field tests is proposed.

2. Lateral caisson load test

The new Niu-Dou Bridge in Ilan County, Taiwan was opened to traffic in October 2010, and the old bridge was demolished 2 months later in December 2010. Before the old Niu-Dou Bridge was removed, NCREE conducted a series of in situ loading tests on it with the approval of the Directorate General of Highways, Ministry of Transportation and Communications of Taiwan. The old Niu-Dou Bridge, crossing the Nan-Yan River, was composed of two independent bridge structures, each supporting opposite directions of vehicular traffic flow, as shown in Fig. 1. Both bridge structures had seven spans with pre-stressed concrete I-girder-type decks. The bridge structure on the upstream side was built in 1961 and its pier columns were elliptically sectioned. The bridge structure on the downstream side was built in 1995 and its pier columns were circularly sectioned.

In the lateral caisson load test, caisson foundation P5FL at Pier #5 of the downstream bridge structure was monotonically pushed, as shown in Fig. 2. This caisson was 12 m long and originally had a 4-m-diameter circular section. After being hit by many typhoons, the diameter of its section was enlarged to 5 m from the caisson top to a depth of 2.8 m to prevent flood scouring.

Fig. 3 shows the test setup. The soil surface around the test caisson was leveled to 1 m below the top of the caisson.

![Fig. 1. Niu-Dou Bridge.](image1)

![Fig. 2. Lateral load test on caisson foundation P5FL.](image2)

![Fig. 3. Test setup.](image3)
Lateral loads were applied to the caisson using two 4.9-MN-capacity hydraulic jacks placed in series. The loads were applied at a depth of about 0.45 m below the top of the caisson. The reaction for the jacks was provided by a stiffened steel box resting against caisson foundation P5FR, as shown in Fig. 2, at Pier #5 of the upstream bridge structure. Caisson P5FR originally had a 4-m-diameter circular section, but the diameter of its upper portion was also enlarged to 10 m.

LVDTs were installed on reference beams to measure the displacements of both caissons P5FL and P5FR during loading, and a load cell was placed in line with the hydraulic jacks to measure the loads applied, as shown in Fig. 3.

To measure the tilt angle of the test caisson around the horizontal axis, which is orthogonal to the loading direction, during the loading process, two tilt meters were set on the sides of the caisson. A vertical hole with a length of 3.6 m was drilled into the original wall body of the caisson, and a shape acceleration array sensor (SAA-1) was embedded in it to measure the continuous displacement profiles of the caisson during loading. The shape acceleration array is composed of a series of micro-electromechanical accelerometers, using the array calculations to transform the acceleration records to displacement data.

To investigate the range in the supporting strata to the test caisson during loading, another shape acceleration array sensor (SAA-2) was set in a 15-m-long vertical PVC pipe, embedded in the soil 1 m in front of the caisson, to measure the profiles of the soil displacement over depth. The layout of these sensors is displayed in Fig. 4.

3. Soil conditions

3.1. Soil profile

An exploratory borehole (BH-1), extending to a depth of 20 m, was drilled within the area of the project. The elevation of the top of the borehole corresponded to about 1 m above the top of the test caisson. The results of the exploratory borehole and the results of past boreholes for
the bridge construction showed that the soil at the site, within a depth of 20 m, mainly consists of gravel and cobbles with some sand or silt. As shown in Fig. 5(a), 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 test. Laboratory tests on soil samples taken from the site, using a split-spoon sampler, indicated that the average water content, the average total unit weight, and the average void ratio of the soil were 15%, 21.3 kN/m³, and 0.66, respectively, and that the average specific gravity of the soil solids was 2.75. Due to the size of the split-spoon sampler, the split-spoon samples could not contain coarse material (large gravel and cobbles), and thus, the values for the water content, the unit weight, and the void ratio, determined from the above laboratory tests, were not fully representative of the field values of gravelly soil.

At borehole BH-1, a down-hole velocity investigation was also conducted. The travel times of the shear waves from the energy source to the receivers at different depths were measured. Fig. 5(b) shows the travel time-depth curve. The slope of the curve defines the shear wave velocity. Fig. 5(c) shows three distinct shear wave velocities \( V_s = 247 \text{ m/s} \) (0–3 m), 495 m/s (3–13 m), and 515 m/s (13–19.5 m). The value of the shear wave velocity is seen to roughly increase with the increase in depth, although the difference of the last two velocities is not obvious.

### 3.2. In situ geotechnical testing

Small-specimen laboratory tests may not be representative of gravelly soil due to the large grain size of gravel. Therefore, in situ large-scale geotechnical tests, including a field density test, a sieve analysis, vertical and horizontal plate loading tests, and direct shear tests, were performed to obtain more appropriate physical and mechanical properties of the soil. Four 1-m-depth test pits, whose bottom elevations corresponded to about 1 m below the top of the test caisson, were excavated for the geotechnical tests. The results are summarized as follows:

#### 3.2.1. Field density test and sieve analysis

The results of the field density test and the sieve analysis are summarized in Table 1. The results of the field density test indicate 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. As compared with the values from the split-spoon soil samples, the field value for the unit weight is larger, but the field values for the water content and the void ratio are smaller. The sieve analysis generated a grain-size distribution of the soil, as displayed in Fig. 6, indicating a gravel content of about 89%, an effective size \( D_{10} \) of 2 mm, a coefficient of uniformity \( C_u \) of 57.5, and a coefficient of curvature \( C_c \) of 1.5. According to the Unified Soil Classification System, the soil is classified as well-graded gravel (GW).

### Table 1

<table>
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<th>Parameter</th>
<th>Value</th>
<th>Parameter</th>
<th>Value</th>
</tr>
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<td>Moist unit weight (kN/m³)</td>
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<td>Effective size ( D_{10} ) (mm)</td>
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<tr>
<td>Water content (%)</td>
<td>10</td>
<td>Coefficient of curvature ( C_c )</td>
<td>57.5</td>
</tr>
<tr>
<td>Void ratio</td>
<td>0.31</td>
<td>Coefficient of uniformity ( C_u )</td>
<td>1.5</td>
</tr>
<tr>
<td>Specific gravity of soil solids</td>
<td>2.75</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

#### 3.2.2. Plate loading tests

The typical size of a plate for plate loading tests is 30 cm. According to the grain-size distribution of the soil, this plate size did not seem to be large enough to cover most of the range in particle sizes. Therefore, a larger rectangular steel plate, 90 cm × 90 cm, was used for the plate loading tests. In addition, in order to further apply the results of the plate loading tests to analyze the lateral response of the test caisson, the use of this larger-sized plate also helps reduce the modification errors for the difference in scales between the plate and the caisson.

Table 2 lists the planned test loads and the actual maximum loads for the vertical and the horizontal plate loading tests. The pressure-displacement responses of the plate obtained from the vertical and the horizontal load tests are shown in Figs. 7 and 8, respectively. It is seen that the response curves are highly nonlinear. For the vertical plate loading test, the slope of the pressure-displacement curve starts gently, but steepens as the load is increased. This phenomenon could probably be explained by the fact that the soil particles are interlocked and the soil becomes more densely packed. For the horizontal plate loading test,
the uprising section of the pressure-displacement curve is gentler than that of the vertical plate loading test; when the plate pressure reaches about 372 kN/m², the subgrade reaction seems to reach its ultimate state (the soil had failed and could not sustain larger loading at this time).

3.2.3. Direct shear tests

Four soil specimens, 80 cm (L) × 80 cm (W) × 40 cm (H), were used to test the shear stress against the shear displacement under normal stress levels of 76.6, 153.3, 229.9, and 306.6 kN/m². The experimental shear stress–shear displacement responses of the specimens to the normal stress levels are displayed in Fig. 9. A plot of the shear stress against the normal stress was constructed based on the peak shear stress and the residual shear stress of each curve to yield the strength parameters of the soil. The frictional angle and the cohesion for the peak shear strength were 37° and 9.81 kN/m², respectively, and the frictional angle and the cohesion for the residual shear strength were 31.8° and 0 kN/m², respectively.

4. Results of lateral caisson load test

The loading procedure for the lateral caisson load test was load-controlled and consisted of five main loading–unloading cycles, as shown in Fig. 10. The target load levels for each cycle were 638, 1275, 3188, 3532, and 4120 kN. The actual loads applied were 638, 1285, 3296, 3590, and 4220 kN. The test results are summarized as follows:

4.1. Displacement of test caisson

Fig. 11 shows the relationship between the lateral load and the horizontal displacement at the position of the applied loads. The curve is highly nonlinear, although the displacement of the caisson was only 15.5 mm at the lateral load of 4220 kN.
4.2. Tilt of test caisson

Fig. 12 shows the relationship between the applied load and the average tilt angle by the tilt meters on the sides of the test caisson. At the maximum load of 4220 kN, the tilt angle was only about 0.4°. The tilt response of the test caisson is also highly nonlinear. Fig. 13 shows the displacement profiles of the upper portion of the caisson obtained from SAA-1. The differences between the SAA and the LVDT measurements at the position of the applied load are small, which shows that the precision of SAA is reliable. Furthermore, within the depth range of measurement, the caisson moved like a rigid body. However, the maximum tilt angle estimated from SAA-1 is close to about 0.1°, which is much smaller than that measured from the tilt meters, as shown in Fig. 12. A comparison with the numerical simulation, presented in the subsequent section, indicates that the tilt angles estimated from SAA are more consistent to the numerical results. The reason for the larger tilt measurements of the tilt meters may be that since the tilt meters were located on the enlarged portion of the caisson, the enlarged part had local deformation at the positions of the meters under several kilo-newtons of lateral loading so that their measurements contained the local tilt response in addition to the global one.

4.3. Soil displacement profile

Fig. 14 shows the profiles of the soil displacement against the depth measured from SAA-2 during the loading. The soil displacement was found to be less than the caisson displacement. The majority of the soil deformation seemed to occur within 4 m of the soil surface. It can also be observed that as the lateral load reached about 1912 kN, an obvious turning point formed in the profile at a depth of 1 m. The occurrence of this turning point is possibly due to a soil wedge forming and its boundary passing through this point. The upper soil mass, displacing in both horizontal and vertical directions, had a different displacement pattern from the lower one.

5. Simulations

5.1. Analysis model

The lateral caisson load test was simulated using the structural analysis program SAP2000 (Computer & Structures, Inc, 2002). A numerical model was constructed, as shown in Fig. 15(a). The numerical model included the test caisson and the pier column above it. The pier column was modeled as beam-column elements. A vertical load of 2863 kN was applied to the top of the column to simulate the self-weight of the superstructure above it. The Winkler-beam model, in which the caisson body is modeled by beam elements and the soil reactions are simulated by spring elements, was applied to simulate the test caisson-soil system. The six-component Winkler spring model, proposed by the Japanese Specifications of Highway Bridges (Japan Road Association, 1990), was adopted to simulate the soil reactions on the test caisson. This model is also included in the Specifications for
Foundations of Building Structures of Taiwan (Ministry of Interior of Taiwan, 2001). As shown in Fig. 15(b), this model uses six types of springs 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 $k_{H1}$ and $k_{SVB}$ represent the horizontal subgrade reaction and vertical shear stress 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.

According to the Japanese Specifications of Highway Bridges, for the test caisson, the width and the length, $B_e$ and $L_e$, of its equivalent section are calculated as

$$B_e = L_e = 0.8D$$

where $D$ is the caisson diameter.

5.2. Determination of the spring properties

In the Japanese Specifications of Highway Bridges, the load-displacement responses of the springs are assumed to be elasoplastic, and the SPT-N value (Standard Penetration Test blow count) of the stratum can be used to estimate the stiffness and the ultimate resistance of the springs. Since the test site was on gravelly soil, it was not appropriate to use the SPT-N value to determine the properties of the springs for the test caisson. Therefore, this study proposes a method which modifies the load-displacement responses from the plate loading tests and the direct shear tests to determine the load-deformation characteristics of the springs. It is described below.

5.2.1. $k_H$

This type of spring was used to simulate the horizontal subgrade reaction from the soil in front of the test caisson. The load-displacement response from the aforementioned horizontal plate loading test was adopted to determine the property of $k_H$ through some modifications. Since the size of the test caisson was different from that of the plate used in the plate loading test, and since the soil deformation modulus changed with depth, the effects of foundation size and depth were considered to modify the load-displacement response obtained from the plate loading test.

The modification is shown in Fig. 16. Firstly, for the convenience of subsequent modifications, the plate pressure-displacement response is transformed to a curve of soil
horizontal reaction \( p \) (in terms of force per unit length) and displacement \( y \) by multiplying the pressure values by the plate width. This curve is regarded as a base \( p-y \) relationship, which corresponds to a foundation size of 0.9 m at a depth of 0.5 m. The base curve is further simplified as a simple nonlinear curve with an ultimate reaction \( p_u \). Fig. 17 shows the simplified \( p-y \) response of the plate. The secant subgrade reaction modulus \( E_s \) (i.e., the secant slope of the curve) is expressed to be a function of displacement \( y \) as follows:

\[
E_s(y) = 30877 \left( \frac{y}{0.009} \right)^{-0.32} \text{[kN/m}^2\text{]} \tag{2}
\]

and the \( p_u \) value is 335 kN/m. Then, two modification factors, \( F_s \) and \( F_r \), are introduced to modify the secant subgrade reaction modulus and the ultimate reaction, respectively, for considering the effects of foundation size and depth. The procedures for determining the modification factors are introduced below.

### 5.2.1.1. Modification factor \( F_s \)

According to previous studies (Terzaghi, 1955; Scott, 1981; Scott and Juinmarongrit, 2003; Chiu and Chen, 2005), the subgrade reaction modulus of a foundation-soil system (note: the unit is force per unit area) is nearly constant with respect to the foundation size. Therefore, stiffness modification factor \( F_s \), for the influence of foundation size, was set to one in this study. According to the velocity profile of the shear wave at the test site, the dynamic shear moduli for depth ranges 0–1 m and 1–11 m of the soil surface around the test caisson were 141 MN/m² and 564 MN/m², respectively, representing the effect of depth on the soil modulus. Stiffness modification factor \( F_s \), considering the depth effect, was determined based on the ratio of the shear modulus of each depth to that of a depth of 0.5 m; \( F_s = 1 \) for the depth of 0–1 m and \( F_s = 4 \) for the depth of 1–11 m.

### 5.2.1.2. Modification factor \( F_r \)

The passive wedge model (Reese et al., 2006), shown in Fig. 18, was applied to estimate strength modification factor \( F_r \). The effects of foundation size and depth can be directly included in the model. In this model, it is assumed that the soil is a \( c-\phi \) soil and the front soil ultimate resistance is provided by the soil wedge developing in front of the foundation. Firstly, the limit equilibrium method is applied to compute ultimate lateral force \( R_l \) as follows (Chiou and Chen, 2006):

\[
R_l = (W + C_n \cos \theta + 2C_l \cos \theta + 2F_1 \tan \phi \cos \theta) \cot(\theta - \phi)
+ C_n \sin \theta + 2C_l \cos \eta \sin \theta + 2F_1 (\tan \phi \sin \theta \cos \eta - \sin \eta) \tag{3}
\]

In the above equation,

\[
W = \gamma z^2 \tan \theta \left( \frac{D}{2} + \frac{z}{3} \tan \theta \tan \eta \right) \text{ (weight of wedge)} \tag{4}
\]

\[
C_n = c z \sec \theta (D + z \tan \theta \tan \eta) \text{ (cohesive force on base of wedge)} \tag{5}
\]

\[
C_l = \frac{c}{2} z^2 \tan \theta \sec \eta \text{ (cohesive force on side of wedge)} \tag{6}
\]

\[
F_1 = \frac{c}{6} K_0 \tan \theta \sec \eta \text{ (normal force on side of wedge)} \tag{7}
\]

where \( \gamma \) is the effective unit weight of the soil, \( c \) and \( \phi \) are the cohesion and the frictional angle of the soil, respectively, \( D \) is the diameter of the caisson, \( z \) is the depth of the wedge, \( \theta \) and \( \eta \) are the slope of the wedge and the expansion angle of the wedge at the soil surface, respectively, in which \( \theta = 45^\circ + \phi/2 \) and \( \eta = \phi/2 \) based on the suggestions in Gabr and Borden (1990), and \( K_0 \) is the coefficient of lateral earth pressure at rest \((=1-\sin \phi)\).

When the soil is purely cohesive or cohesionless, Eq. (3) can be further expressed explicitly (e.g., Reese et al., 2006).

From the horizontal plate loading test, the ultimate horizontal load was 302 kN. To fit this ultimate resistance with the wedge model, the parameters of shear strength \( c \) and \( \phi \) from the direct shear tests were slightly modified to be 24.52 kN/m² and 37°, respectively, using Eq. (3). By differentiating \( R_l \) with respect to \( z \), the variation in ultimate subgrade reaction \( p_u \) with depth could be calculated. It is expressed as

\[
p_u(z) = \frac{dR_l}{dz} = \left( \frac{dW}{dz} + \frac{dC_n}{dz} \cos \theta + 2 \frac{dC_l}{dz} \cos \theta \right)
+ 2 \frac{dF_1}{dz} \tan \phi \cos \theta \cot(\theta - \phi) + \frac{dC_l}{dz} \sin \theta
+ 2 \frac{dC_n}{dz} \cos \eta \sin \theta + 2 \frac{dF_1}{dz} (\tan \phi \sin \theta \cos \eta - \sin \eta) \tag{8}
\]

Let \( \sigma_r \) represent the effective vertical stress to replace the term \( \gamma z \) for deriving the following general expressions for \( dW/dz, dC_n/dz, dC_l/dz, \) and \( dF_1/dz \):

\[
\frac{dW}{dz} = \sigma_r \tan \theta (D + z \tan \theta \tan \eta) \tag{9}
\]

\[
\frac{dC_n}{dz} = c \sec \theta (D + 2z \tan \theta \sec \eta) \tag{10}
\]

\[
\frac{dC_l}{dz} = c \tan \theta \sec \eta \tag{11}
\]
With Eq. (8) and the parameters in Table 3, the modification factor for the ultimate horizontal reaction at any depth $z$ was computed by dividing the corresponding ultimate reaction by that obtained from the horizontal plate loading test (i.e., 335 kN/m).

According to modification factors $F_s$ and $F_r$, determined for different depths (as displayed in Fig. 19), the modified $p$–$y$ curves for the test caisson were then obtained. For example, the modified $p$–$y$ curves at depths of 1 m, 2 m, and 4 m are shown in Fig. 20.

### 5.2.2. $k_{SVB}, k_{SHL}, k_{SVL}$

In the absence of data on the shear characteristics of the interface between the test caisson and the soil, this study tentatively adopted the shear stress–shear displacement responses obtained from the direct shear tests to roughly estimate the spring properties of $k_{SVB}$, $k_{SHL}$, and $k_{SVL}$, assuming that the shear behavior of the caisson–soil interface was similar to that of the soil in the direct shear tests. Firstly, a normalized shear stress–shear displacement relationship, with respect to the peak shear strength, was constructed based on Fig. 9. With the normalized relationship constructed, the force–deformation properties of springs $k_{SVB}$, $k_{SHL}$, and $k_{SVL}$ at different depths of the test caisson were then determined based on the associated peak shear strength. The shear strength for springs $k_{SVL}$ and $k_{SHL}$ was estimated based on the lateral earth pressure at rest ($K_0$ condition) on the caisson shaft. As for spring $k_{SVB}$, the lateral pressure may be increased with the process of the lateral movement, especially for the shallow soil layer, and thus, strictly speaking, spring $k_{SVB}$ would be coupled with spring $k_H$. It is not an easy task to simulate this coupling effect with the current Winkler spring model. Thus, based on the Japanese Specifications of Highway Bridges, in this study, spring $k_{SVB}$ was regarded as an uncoupled spring and its property was set to be the same as that of springs $k_{SVL}$ and $k_{SHL}$.

### 5.2.3. $k_V$

The load-displacement response from the vertical plate loading test was modified for determining $k_V$. Similar to the above procedure, the load-displacement relationship was firstly expressed in terms of displacement-dependent secant moduli. Since the subgrade reaction continued to increase with the plate displacement, without an ultimate point, only the subgrade modulus was modified.
Considering the soil modulus difference due to depth, stiffness modification factor \( F_s \), determined previously, was adopted to modify the secant slope of the load-displacement curve for the property of \( k_V \).

5.2.4. \( k_S \)

With the same assumption applied to \( k_{SVB} \), \( k_{SHL} \), and \( k_{SVL} \), the normalized shear stress–shear displacement curve constructed from the direct shear tests was used to determine the property of \( k_S \) based on the peak shear strength on the base of the test caisson. In this study, the peak shear strength on the caisson base was estimated based on the submerged weight of the caisson.

5.3. Analysis results

Fig. 21 shows the load-displacement response of the test caisson at the point of the applied loads computed by SAP2000. Comparing this to the experimental curve, there were a few discrepancies, namely, that at load levels lower than about 3100 kN, the computed displacement was slightly larger than the measured displacement, and at higher load levels, the computed displacement was slightly smaller than the measured displacement. Despite these discrepancies, the agreement is fairly good. The tilt angles of the analysis were compared with those of the tilt meters and SAA. The computed tilt angles were more consistent with those estimated from the SAA measurements, as shown in Fig. 22.

According to the geometry of the test caisson, whose embedded depth to width ratio is 2.75, the caisson is generally classified as a rigid foundation (Gerolymos and Gazetas, 2006a). However, this is a crude classification, because it ignores the influence of soil stiffness. Fig. 23 shows the lateral displacement of the caisson shaft at the maximum load. The caisson moved half-rigidly accompanied with some rotation, and its depth of zero-deflection was located at a depth of about 8.5 m below the soil surface (about 77% of the embedded depth of the caisson). The movement of the upper portion of the caisson is consistent with that observed from the SAA-1 measurements.

The total lateral resistance is provided by three parts of soil reactions: \( k_H \), \( k_{SHL} \), and \( k_S \). Since the directions of \( k_{SVB} \), \( k_{SVL} \), and \( k_V \) are vertical, their contribution to the lateral resistance is null. Fig. 24 shows the variations in lateral resistance contributed by \( k_H \), \( k_{SHL} \), and \( k_S \) during the loading process. It can be clearly observed that the soil resistance in front of the caisson (\( k_H \)) is the main source of...
lateral resistance. The side shear force \( k_{SHL} \) provides some lateral resistance and it fully develops at a displacement of about 5.5 mm. After this displacement, the side resistance seems to gradually decay. This is because the caisson moves in opposite directions above and below the depth of zero deflection; thus, the directions of the corresponding side shear forces are opposite. The shear force on the base of the caisson \( k_S \) is negative because of its reverse movement with respect to the upper part of the caisson.

Figs. 25(a) and (b) plot the distributions of the horizontal subgrade reaction on the front of the caisson and the horizontal shear stress on the side of the caisson, respectively. In Fig. 25(a), it can be seen that the horizontal subgrade reactions on the front of the caisson at depths of 0–1 m are smaller because of smaller subgrade reaction moduli. Also, the subgrade reactions at all depths have not reached the ultimate reactions at the maximum load. It can be seen in Fig. 25(b) that the side shear stress levels at shallow depths quickly reach their respective ultimate limit states at a low level of loading, and as the loading increases, the range at which the stress reaches its ultimate limit state increases. At the maximum lateral load, the depth at which the ultimate stress is reached is around 7 m, about 64% of the embedded depth of the caisson.

Conclusions

The in situ load test on caisson foundation P5FL, of the old Niu-Dou Bridge in Taiwan, provides an opportunity to investigate the existing method of caisson foundation analysis. Several conclusions from this study can be drawn, as follows:

1. The test caisson, embedded in gravelly soil, seemed to provide large resistance to lateral loading. When the lateral load was about 4220 kN, the top of the caisson displaced only 15.5 mm. However, even though the lateral displacement was small, the load-displacement response exhibited high nonlinearity.

2. The Winkler-beam model, using six components of Winkler springs, can be applied to simulate the lateral behavior of the test caisson. The parameters of the springs can be determined by modifying the response curves of the plate loading tests and the direct shear tests considering the influence of the foundation size and depth.

3. According to the results of the SAA measurements and the numerical analysis, the movement of the test caisson during loading was semi-rigid. Based on the deflection shape of the caisson from the numerical analysis, the point of zero-deflection was located at a depth of about 8.5 m, which is about 77% of the embedded depth of the caisson.

4. The lateral resistance of the test caisson to the lateral loading mainly comes from the horizontal resistance of the soil in front of the caisson. The side shear resistance of the caisson contributes part of the resistance; however, its contribution reaches a maximum at a small displacement (about 5.5 mm). 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.
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