FULL-SCALE EXPERIMENTAL INVESTIGATION ON DYNAMIC P-Y CURVE FOR STEEL PILE MICROPILES

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\textbf{Key words:} Micropile, dynamic p-y backbone curve, static p-y curve, pile-soil interaction, shake-table test

\textbf{Abstract:} Full-scale shake-table tests on micropile-soil (dry sand) interaction were carried out to study the dynamic p-y curve for use in analyzing a new type of jointless bridge where the relationship between soil lateral resistance and deflection in micro piles can be depicted. A set of dynamic p-y curves was developed based on shake table test results. Then, a comparison between the dynamic p-y backbone curves and the static curves suggested by API and Reese was performed. Finally, simplified FEM pile-soil interaction models that incorporate the dynamic p-y backbone curve and also the static p-y curves was calibrated against the test results. Results indicate that: 1) ultimate soil resistance predicted by the dynamic p-y backbone curve is significantly greater than that calculated using the static p-y curves; 2) peak moment and deflection values calculated using the proposed dynamic p-y backbone curve approximately agree with the test results. Traditional p-y curves cannot accurately depict the dynamic response of micropiles, as there are significant differences between the dynamic and static responses. Thus, the dynamic p-y backbone curve proposed in this paper is recommended when considering the dynamic interaction of steel pipe micropiles with soil. Also it gives a reference for dealing with similar dynamic structure-soil interaction problems.

1 INTRODUCTION

Jointless bridges can be classified as integral abutment jointless bridge (IAJB), or semi-integral abutment jointless bridge (SIAJB){\textsuperscript{[1]}}. In comparison with IAJB, SIAJB has a somewhat reduced level of seismic performance{\textsuperscript{[2]}}. To improve SIAJB’s seismic performance, some researchers have proposed a new type of SIAJB in which a row of micropiles, which diameter is usually less than 300cm, are embedded underneath the sleeper beams of approach slabs{\textsuperscript{[3-4]}} (Fig.1). It is anticipated that some earthquake energy can be dissipated by the micropile-soil interaction system, thus effectively improving the structural integrity and seismic performance of SIAJB. In this paper, dynamic testing and numerical analyses were performed to study the micropile-soil interaction by focusing on its dynamic p-y curve.
Fig. 1 SIAJB with micropiles under sleeper beams

The Winkler Foundation Beam Model [WFBM] shown in Fig 2, is generally adopted to simplify the computation of pile-soil interaction response[5] where the pile is discretized into a series of beam elements. Horizontal displacement and rotation are applied at lumped masses located at the joints between those elements, and the soil surrounding the pile is replaced by a series of discretized spring-damper elements in which the spring stiffness is determined by p-y curves such as NCHRP [6], JRA [7], API-RP2A [8] and Reese [9]. However, there are two limitations associated with all the above-mentioned p-y curves. First, most are only suitable for sliding prevention piles or those with a diameter greater than 1 m [10]. Sun[11] and Fan[12] concluded, based on theoretical analysis and quasi-static test, that the prevailing p-y curves were not suitable for micropiles in static action, and thus proposed a static p-y curve applicable to micropiles. Those p-y curves were generally derived from only static or quasi static testing. Dou and Byrne[13],Ting et al.[14] and Choi et al. [15] concluded that static p-y curves are not suitable for analyzing dynamic pile-soil interaction. Choi et al. [15] derived a dynamic p-y curve from elasticity and plasticity, which provides a theoretical dynamic p-y curve research analysis without experimental verification.

Based on full-scale shake-table tests of micropile-soil interaction and Kondner’s hyperbolic function [10][16], this paper proposes a dynamic p-y backbone curve suitable for micropiles, and offers calculation method for considering pile-soil interaction.

Fig. 2 Dynamic Winkler Foundation Model[5]
2 ESTABLISHING DYNAMIC P-Y BACKBONE CURVE

There are two steps involved in establishing dynamic p-y backbone curves: 1) obtaining the properties of soil by a soil test; 2) establishing a relationship between soil resistance to pile movement and the deflection of pile shaft by performing a shake-table test. And then, a dynamic p-y backbone curve can be established by referring to Kondner’s hyperbolic function [10].

2.1 Computation method

P-y curve is composed of soil resistance along the pile length and relative lateral displacement between the pile and soil. Soil resistance (p) is first calculated using the measured strains on pile shaft with a five-point finite difference algorithm[17]. Then, the shaft lateral deflection (\(y_p\)) is calculated using moment-curvature method [18]. The displacement of soil (\(y_s\)) is calculated by integrating the measured soil acceleration in frequency domain using the quadratic integral method[19]. Finally, the proposed p-y curve can be obtained from soil resistance p on pile and the relative deflection y (\(= y_p - y_s\)).

2.2 Establishment of dynamic p-y backbone curve

The relationship between soil resistance around the pile and relative displacement obtained from test can be represented using an approach proposed by Kondner as follows [10]:

\[
p = \frac{y}{k_{ini} + \frac{y}{p_u}}
\]

(1)

where \(k_{ini}\) is the initial modulus of subgrade; \(p_u\) is the ultimate soil resistance; \(y\) is the relative displacement and \(p\) is the soil resistance. The initial modulus of subgrade \(K_{ini}\), subgrade modulus \(K\) and lateral self-weight stress \(\delta\) can be calculated using the following equations [10]:

\[
K = A P_u (\frac{\delta}{p_u})^{0.5}
\]

(2)

\[
\frac{P}{0.01D} = \frac{1}{k_{ini} + \frac{0.01D}{p_u}} = K (N/mm^2)
\]

(3)

where \(\delta = K_0 \gamma_c \cdot \gamma_c = \frac{\gamma}{1-\mu}; \gamma_c\) is the vertical stress due to self-weight; \(K_0\) is a coefficient of lateral soil resistance; \(\gamma\) is subsoil specific weight and \(\mu\) is Poisson ratio.[10]

The ultimate soil resistance \(p_u\) can be calculated as follows [10]:

\[
\frac{P}{D} = BK_p \gamma_c^n
\]

(4)

where \(K_p\) is the coefficient of passive soil pressure; \(\varphi\) is the angle of internal friction; and \(B\) is a constant determine by curve fitting.
3 SHAKE-TABLE TESTING

The earthquake simulation system includes a shake table, with its maximum longitudinal and transverse acceleration of 1.5g and 1.2g, respectively. The operating frequency ranges from 0.1Hz to 50 Hz and the maximum displacement is \( \pm 250 \) mm. The testing arrangement is shown in Figure 3. The size of soil container was \( \times \times \times \) 2.1m with a 10 cm thick polyethylene foam at two ends to reduce the boundary effects to negligible level [20].

![Fig. 3 Test setup on a large shake-table](image)

3.1 Steel micropile

Earthquake energy could be dissipated through the interaction between micropile and soil in this proposed concept for semi-integral abutment jointless bridges. The dissipated energy is achieved through a steel pipe micropile with an appropriate diameter. A pile with a diameter of 6cm has the maximum energy dissipation in comparison with other two piles with their diameters of 8cm and 10cm [20]. Thus, 6cm was selected as the diameter of this test pile. All other pile parameters are shown in Table 1.

<table>
<thead>
<tr>
<th>Material</th>
<th>Outer diameter(cm)</th>
<th>Inner diameter (cm)</th>
<th>Modulus of elasticity (MPa)</th>
<th>Density (kg/m(^3))</th>
<th>Poisson ratio-(\nu)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Steel</td>
<td>6</td>
<td>5.4</td>
<td>(21\times10^4)</td>
<td>7850</td>
<td>0.3</td>
</tr>
</tbody>
</table>

3.2 Primary soil parameters

The soil surrounding the micropile was a dry medium sand. Its basic properties are shown in Table 2.
<table>
<thead>
<tr>
<th>Material</th>
<th>Density (kg/m³)</th>
<th>Relative density (%)</th>
<th>Friction angle (°)</th>
<th>Void ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dry sand</td>
<td>1601</td>
<td>43</td>
<td>31</td>
<td>0.663</td>
</tr>
</tbody>
</table>

The grain size distribution curve obtained from sieve analysis is shown in Fig. 4. The steep slope of the grain size distribution indicates an approximately homogeneous grain size. Moreover, d60 is 0.57mm, d10 is 0.18mm, and the non-uniformity coefficient Cu is large than 3.17 and smaller than 5. These further indicate that a large proportion of the sand is of one particle size.

Fig. 4 Gradation of sand used in testing

3.3 Instrumentation and load setup

Two piles (marked as Pile 1 and pile 2) are shown in Figure 5. However, only pile 1 is related to this paper, therefore, the instrumentation plan discussed below relates to Pile 1 only. The end effect and pile group effect can be ignored in this test because the distance between pile 1 and the soil container wall is larger than 6D (D is the diameter of pile 1), and the distance between pile 1 and pile 2 is larger than 10D [20]. Accelerometers shown in Fig. 5 were marked as A1 ~ A9.

(a) Front elevation  (b) Side elevation  (c) Ground plan

Fig. 5 Accelerometer layout
There were 38 strain gages installed on the pile as shown in Fig. 6. It can be seen that every 10 cm from bottom to cap at two sides of steel pile is arranged with a symmetrical arrangement of A1–A19 and B1–B19 gauges. Attachment of many strain gages surrounding the pile would affect the experimental results. Therefore, drilled holes in the lower part of pile at certain an interval allows strain gage wires to go through the inside of the pile, while keeping the upper (more stressed) parts without drilled holes (Fig. 7).
Sinusoidal acceleration waveforms were imposed at the top of the shake-table as shown in Fig.8. The use of a series of sinusoidal input waves (with different amplitudes and frequencies in different tests) was chosen in lieu of a complex earthquake excitation, which would make the data analysis very complex[21]. The loading cases are shown in table 3. The head mass at the top of the pile (placed over the disk shown in Fig. 7) was 60kg corresponding to the tributary weight of the link slab. Two other mass options (120 kg and 180 kg) were also tested. Therefore, there are three mass as 60kg, 120kg and 180kg had been chosen to analyzing the impact of head mass to seismic performance of micropile, and above depicted that the stress states and loading conditions are the same full-scale.

![Fig.8 One of the input acceleration for shake-table test (with amplitude and frequency of 0.2g and 2Hz, respectively)](image)

<table>
<thead>
<tr>
<th>Load case</th>
<th>Frequency (Hz)</th>
<th>Mass (kg)</th>
<th>Peak acceleration (g)</th>
<th>Waveform</th>
</tr>
</thead>
<tbody>
<tr>
<td>Case 1</td>
<td>1, 2, 5, 8, 9</td>
<td>60</td>
<td>0.05</td>
<td>Sinusoidal wave</td>
</tr>
<tr>
<td></td>
<td>10, 15, 20, 25</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Case 2</td>
<td>5, 8, 15</td>
<td>60, 120, 180</td>
<td>0.05</td>
<td>Sinusoidal wave</td>
</tr>
<tr>
<td>Case 3</td>
<td>5, 8, 15</td>
<td>120</td>
<td>0.05, 0.1, 0.2, 0.3, 0.4</td>
<td>Sinusoidal wave</td>
</tr>
</tbody>
</table>
3.4 End conditions

To eliminate the effects of end condition (container walls), 10-cm-thick polyethylene foam (Fig.5) was glued to the inside of the container wall in the longitudinal direction. Additional experiments were conducted to investigate the effects of end conditions. A comparison of soil accelerations between the longitudinal direction (where there was polyethylene foam) and transverse direction (where there was no polyethylene foam) was performed. In this comparison, the amplification factor for acceleration in the longitudinal (Fig.9) is approximately 1, and is ranging from 2 to 3 in Y direction. Amplification factor is defined as the ratio of soil acceleration at 60 cm from the container wall to the soil acceleration at the container wall[21]. Results show that end condition effects can be ignored in the direction where the foam is applied.

![Graphical representation of end conditions](image-url)

**Fig.9 Impact factor of acceleration at two directions**
3.5 Dynamic p-y backbone curve

The p-y curves obtained based on the above-mentioned test and using the algorithm described earlier are shown in Figs.10–12. Peak points of p-y curves were extracted at different depths along pile shaft and then plotted as shown in Fig. 13. Considering the scatter in peak points, two curves, an upper and a lower curve, are selected to envelope the data as shown in Fig. 16. backbone

(a) Changing acceleration
(0.1g, 0.2g, 0.3 g, 0.4g)

(b) Changing frequency
(5Hz, 8 Hz, 15 Hz)

(c) Changing mass (60 kg, 120kg, 180kg)

Fig. 10 Experimental dynamic p-y curves at a depth of 30 cm
Yizhou Zhuang et al. FULL-SCALE EXPERIMENTAL INVESTIGATION ON DYNAMIC P-Y CURVE FOR STEEL PILE MICROPILES

Fig. 11 Experimental dynamic p-y curves at a depth of 50 cm

(a) Changing acceleration

(0.1g, 0.2g, 0.3 g, 0.4g)

(b) Changing frequency

(5Hz, 8 Hz, 15 Hz)

(c) Changing mass (60 kg, 120kg, 180kg)
Fig. 12 Experimental dynamic p-y curves at a depth of 70 cm
Fig. 13 Peaks of dynamic p-y curves at different burial depths
3.5.1 Calculation of initial modulus of subgrade

The modulus of subgrade k was acquired based on soil resistance p and formulation (3). Because of scattering of soil resistance p in this test, two sets of data were selected to sufficiently reflect the relation between p and k. One set represents the maximum soil resistance and the other represents the minimum. Self-weight stresses \( \delta \) at different depths were acquired. Then, a regression analysis was used to determine the parameter \( A \) used in Equation 2. The \( A \) parameters were determined to be equal to 330.85 and 170.84 for the upper bound backbone and lower bound curves, respectively. Therefore, k can be defined as follows:

For the upper bound p-y backbone curve:

\[
K = 330.85 P_a \left( \frac{\delta}{P_a} \right)^{0.5}
\]  \hspace{1cm} (5)

For the lower bound p-y backbone curve:

\[
K = 170.84 P_a \left( \frac{\delta}{P_a} \right)^{0.5}
\]  \hspace{1cm} (6)

The initial modulus of subgrade can be obtained using:

\[
K_{ini} = \frac{K P_u}{P_u - K \frac{D}{100}}
\]  \hspace{1cm} (7)

Fig. 14. Comparison of test data with predicted equation (Constant A)
3.5.2 Calculation of ultimate soil resistance

The ultimate soil resistance $P_u$ and the coefficient of passive soil resistance $K_p$ were obtained by combining soil resistance $p$ and deflection $y$ data from test with Equations (1) and (7). Then, parameters $B$ and $n$ in Equation (4) can be obtained by curve fitting using regression analyses (Fig. 15). Thus, $B$ and $n$ were determined to be 7.74 and 1.14, respectively for the upper bound $p$-$y$ backbone curve. The corresponding $B$ and $n$ values for the lower bound $p$-$y$ curve were 5.52 and 1.20, respectively. The ultimate soil resistance $P_u$ can be obtained using Equations 8 and 9.

Upper bound $p$-$y$ backbone curve:

$$\frac{P_u}{D} = 7.74K_p\gamma z^{1.08}$$

(8)

Lower bound $p$-$y$ backbone curve:

$$\frac{P_u}{D} = 3.95K_p\gamma z^{1.25}$$

(9)

Fig. 15 Comparison of test data with predicted equation (Constant B)

Fig. 16 shows the upper and lower bound $p$-$y$ backbone curves. Most peak points are contained between the two boundary curves, thus indicating that the backbone developed equations are in agreement with the test results.
3.6 Comparison between static and dynamic p-y curves

Fig. 17 shows a comparison between static p-y curves suggested by API and Reese for sand, and the dynamic p-y curve developed in this study. The shapes of these four curves are similar to each other. The soil resistance shows an initial high slope followed by a reduced slope at higher displacements. The ultimate soil resistance from the static p-y curves is far less than the resistance from the dynamic p-y backbone curves, due to two reasons. One reason is related to the small size of the micropile, where deflection is mainly due to flexure (when diameter is less than 0.3 m). The deflection of common size piles with diameter larger than 1 m is due to both flexure and shear in which the latter will constitute a higher proportion out of the total deflection with larger diameter. The other reason is related to the dynamic nature of the loading. The pile loaded dynamically is subjected to two actions, one is the inertial action caused by mass on pile head, and the other is the kinematic action caused by the moving soil. The latter action is negligible under static loading. In a conclusion, the performance of micropile under the action of dynamic loading cannot be determined using the prevailing static p-y curves alone.
4 COMPARISON OF ANALYSIS BETWEEN FEM ANALYSIS AND TEST RESULTS

4.1 FEM model

Pile shaft and soil springs were simulated using ‘Beam23’ and ‘Combin39’ elements, respectively, within the ANSYS finite element program. ‘Combin39’ is an element with nonlinear generalized force-deflection (F-D) response. Head-mass was simulated using the ‘Mass21’ element, and the toe of the pile was fixed at the base. One end of each soil spring element (‘Combin39’) was connected to the pile (beam element), and the other was connected to a node where all six degrees of freedom were constrained. A damper element (‘combin14’) was used to include the damping effects of the soil. In addition, properties of sand (shown in Table 2) were simulated using a D-P material that is suitable for modeling sandy soils [22-23].
4.2 Parameters

4.2.1 Damping

Horizontal damping can be determined using a method proposed by Lysmer where viscous damping is used to simulate the process of wave energy dissipation in semi-infinite space [24].

\[ C_{el} = 2D\rho_l (v_{pl} + v_{s1}) \]  
\[ C_{el} = 2D\left[ \rho_l (v_{pl} + v_{s1}) + \rho_{i+1} (v_{p,i+1} + v_{s,i+1}) \right] (i = 2, 3 \ldots, n) \]

Where:  
D is the diameter of pile;  
\( v_{pl} \) is the longitudinal wave velocity;  
\( \mu \) is the Poisson’s ratio;  
\( v_{s1} \) is the Shear wave velocity.

4.2.2 Stiffness

The response of soil is nonlinear with a variable stiffness. This response be described by the appropriate p-y curve. Four different p-y curves, including an upper dynamic p-y backbone curve, a lower dynamic p-y backbone curve, and two static p-y curves (API and Reese) were used in the FEM model.

4.3 Comparison of analysis results

Figs. 18 and 19 show comparisons of peak deflection and peak moment using different p-y curves under 8 Hz and 0.05g sinusoidal excitation. Maximum displacement and maximum moment determined using p-y curves suggested by API and Reese were both greater than those obtained from the test by approximately 20%. However, corresponding results using the upper dynamic p-y backbone curve is within 10% of the shake-table results. The preceding analyses indicate that soil resistance obtained from the dynamic p-y backbone curve is greater than that from the commonly-used static curves under the same displacement. In other words, the pile displacement obtained from the upper dynamic p-y backbone curve is less than that from the static curves under the same soil pressure. The less deflection a pile has, the less internal forces are developed in the pile. Therefore, pile moments obtained from the dynamic backbone curve are lower compared to those from the common static p-y curves. Therefore, the proposed upper dynamic backbone curve is suggested when soil condition is a medium-dense sand.
5 CONCLUSION

Based on the results of shake-table tests and using Kondner’s functions as well as the basic concept of p-y backbone curve, the following conclusions can be made:

(1) The general shape of p-y curves obtained from dynamic test is consistent with the corresponding static curves, but their discrepancy in value is obvious. Ultimate resistance of soil in a dynamically-loaded pile is significantly greater when compared against soil resistance calculated using the common static p-y curves suggested by API or Reese.

(2) A simplified model of pile-soil interaction was established by proposing a dynamic p-y backbone curve for a steel pipe micropile. Results indicate that the peak moment and deflection of micropiles calculated using the proposed dynamic p-y backbone curve (upper curve) are close to those from the shake-table tests. However, results obtained using the common static p-y curves are substantially different from the test results and cannot reflect the response of micropile under dynamic action. Therefore, the upper dynamic p-y curve proposed in this paper is suggested for micropile-soil dynamic interaction when soil conditions conform to medium-dense sand.

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