Modeling Load-Transfer Behavior of H-Piles Using Direct Shear and Penetration Test Results

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Abstract
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Keywords
soil–structure interaction, H-piles, load-transfer, $t-z$ analysis, $t-z$ and $q-z$ curves, direct shear test, pile load-displacement curve

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ABSTRACT
The load transfer analysis (or t–z analysis) has long been used to predict the load-displacement response of axially loaded driven piles. However, the t–z curves along the pile length and q–z curve at the pile tip, required for the t–z analysis, are routinely obtained based on empirical correlations using field and/or laboratory soil tests. This study focuses on the use of a modified Direct Shear Laboratory Test (mDST) to directly quantify the t–z curves and the use of a penetration test, namely the Pile Tip Resistance test (PTR) to quantify the q–z curve, for partial-displacement piles. As part of this study, two instrumented steel H-piles driven in sandy soils were load tested and soil layers at the two sites were characterized using in situ and laboratory tests. A load transfer analysis was conducted utilizing the directly quantified t–z and q–z curves from the mDST and PTR, respectively, to calculate the response of the load tested piles. When compared to the measured load-displacement response and load distribution along pile length, the t–z analysis based on the mDST and PTR measurements showed very good agreement with the measured pile responses. Therefore, and despite the limited database availability at present, the proposed mDST-PTR based model is promising as it represents a simple and cost-effective means to accurately predict the load–displacement response of partial-displacement piles driven in cohesionless soils.

Keywords
soil–structure interaction, H-piles, load-transfer, t–z analysis, t–z, and q–z curves, direct shear test, pile load-displacement curve
Introduction

The typical design approach for axially loaded piles depends on calculating the factored capacity to satisfy the strength limit state requirements, while the serviceability limit of pile settlement is checked after satisfying the strength requirements (Misra and Roberts 2006). This may require several design iterations to satisfy both limit states, especially when the settlement governs the design. Hence, a design methodology integrating the load–displacement response of a pile, which can simultaneously incorporate both the strength and serviceability limits in the design process, will be more efficient compared to the currently used design approaches (Misra et al. 2007; Roberts et al. 2008). However, the analytical methods that can be used to predict the load–displacement response need to be simple for frequent use by design engineers, yet producing acceptable and reliable results.

Several analytical methods have been used in research and practice to characterize the behavior of axially loaded driven piles. The “t–z” method is a frequently used, simple approach for performing the load—transfer analysis of piles (Misra and Chen 2004; AbdelSalam et al. 2012). Although the t–z analysis is used to calculate the vertical load–displacement response at the pile head as well as the vertical load distribution along the pile length, the selection of the load–transfer curves, t–z and q–z curves that, respectively, describe the stress–displacement relationships along the soil–pile interface and at the pile tip, control the accuracy of the analysis (Misra and Chen 2004; Alawneh 2006). Using results from top-down or the O-cell bi-axial load tests on instrumented piles, the t–z and the q–z curves can be determined; however, these tests are relatively sophisticated and expensive. Due to the lack of simple and cost-effective tests that directly measure the t–z and q–z curves, these curves are routinely established using empirical correlations with soil properties obtained from in situ tests such as the Standard Penetration Test (SPT) and the Cone Penetration Test (CPT) (Roberts et al. 2008; AbdelSalam et al. 2012). The proposed laboratory test methods will avoid relying on such empirical methods.

This paper presents the results of top-down Static Load Tests (SLTs) conducted on two instrumented, partial-displacement, steel H-piles driven in cohesionless soils and focuses on the utilization of the load transfer analysis to accurately calculate the load–displacement response and load distribution behavior of the piles. To achieve this goal, a laboratory modified Direct Shear Test (mDST) was used to measure the t–z curves along the pile length, and a shear box was used as a penetration test, namely the Pile Tip Resistance (PTR) test, to measure the q–z curve representing the pile tip resistance. The directly measured t–z curves (using the mDST) and q–z curve (using the PTR) were then used as an input for the classical load transfer analysis. The load transfer analysis with directly measured t–z and q–z curves was validated using the results from the two SLTs conducted as part of this study. The development and use of the aforementioned laboratory tests in the load transfer analysis are intended to alleviate its dependency on empirical correlations to establish the t–z and q–z curves and reduce the associated uncertainties in modeling pile foundations.

Background

Analytical methods such as the boundary element and finite element approaches have been commonly used to solve sophisticated stress transfer problems in soil–pile interaction systems. Despite their complexity, the accuracy of these approaches when modeling axially loaded piles depends on their ability to adequately represent the behavior of the soil–pile interface and the soil response at the pile tip. Due to its simplicity, the load transfer method (t–z analysis) is widely used to calculate the response of axially loaded piles. The t–z method is a one-dimensional iterative technique that solves a set of differential equations, which represents the pile behavior using the finite difference approach (Misra and Chen 2004; Alawneh 2006). The concept of t–z analysis was first developed by Coyle and Reese (1966) and Suleiman and Coyle (1976) in cohesive and cohesionless soils, respectively. The analysis was later improved to account for the pile elastic deformation and to incorporate a hyperbolic response for the springs representing the soil–pile interaction (Reddy et al. 1998; Misra and Roberts 2006). The major limitation of the load transfer analysis, however, is the lack of simple tests to directly measure the t–z and q–z curves. Several empirical correlations have been reported and used to calculate the load–displacement and load distribution responses for pile foundations subjected to axial loading; a task that requires extensive work to determine the most appropriate empirical correlation for a specific soil and pile condition (Abu-Farsakh and Titi 2004; Omer and Delpak 2007).

To directly measure t–z curves in the field using a cost-effective approach, AbdelSalam et al. (2012) developed a modified Borehole Shear Test (mBST) that was successfully utilized in simulating the response of axially loaded piles installed in cohesive soils. Given that these piles were mainly friction piles, the mBST was primarily used to measure the t–z curves required to model the shaft resistance of piles. When compared to the measured load–displacement and load distribution responses from three axial load tests performed on instrumented steel H-piles driven in cohesive soil profiles, the t–z analyses conducted using the mBST data showed very good correlation (AbdelSalam et al. 2012). However, conducting the mBST in cohesionless soils is not easy due to the tendency for borehole collapse and the risk of losing the mBST shear head during testing. Furthermore, the mBST can only capture the t–z curves required for calculating the shaft resistance component.
and not the end bearing component. Hence, this approach is not considered acceptable for piles driven in cohesionless soils.

For the pile shaft resistance component in cohesionless soils, Reddy et al. (2000) conducted modifications to the conventional DST apparatus to measure the soil-pile shaft interface friction angle (\( \delta \)) in the laboratory. Different metal square plates with different surface roughness were fabricated and placed in the lower half of the direct shear box, while the upper half was filled with sand to measure the stress-displacement curves at the interface between the metal plates and the soil. Compared to soil–pile–slip tests as well as to monotonic tensile tests on model piles, Reddy et al. (2000) indicated that the DST can be successfully used to obtain values of \( \delta \) in order to estimate the shaft resistance of steel piles driven in sand. Moreover, Pando et al. (2002) performed a series of interface tests by placing representative pile specimens into the bottom half of the direct shear box to measure the interface friction angle between fiber reinforced polymer composite piles and sand. Pando et al. (2002) concluded that the shaft resistance calculated using the measured interface friction angle from the DST is comparable to the actual pile capacity. However, the DST has not been used to directly measure the t–z curves required in the load transfer analysis to calculate the load-displacement behavior at the pile head and the load distribution response along the pile length.

For the pile tip resistance component in cohesionless soils, several researchers have used small-scale laboratory models to simulate the load-penetration response at the pile tip. For example, Houlsby et al. (1988) studied the behavior of steel piles in cohesionless soils using a small-scale laboratory models to measure the load-penetration response at the pile tip and concluded that the soil stress influence zone is limited approximately to 1.5D below the model plate, where D is the model plate diameter or width. However, determining the minimum required dimensions of the laboratory model (or the calibrated chamber) with respect to the size of a pile tip is still disputed (Bowles 1988; Lee and Salgado 1999). On the other hand, the ASTM standard D1194 (2006) defines the minimum dimensions of the plate load test required to model the load-displacement behavior of rigid footings. Hence, the ASTM recommendations were utilized in this study to simulate the pile tip resistance behavior assuming that the pile tip behaves as a rigid footing.

According to Terzaghi and Peck (1967), the ultimate pile tip resistance can be computed following Terzaghi’s bearing capacity concepts for footings whilst ignoring the pile driving effects on the soil properties. In addition, Coyle and Castello (1981) indicated that the Hansen bearing capacity formulas of shallow foundations can be used for pile tip resistance calculations with an acceptable reliability. Consequently, several researchers such as O’Neill and Reese (1999) developed the bearing capacity factor (\( N_c \)) to calculate the tip resistance of deep foundations in cohesive soil. Likewise, to analytically calculate the soil stresses and the corresponding displacements under a pile tip resting on dense sand or sand–gravel deposits, it is common to simplify the problem assuming a fictitious rigid footing placed on top of the bearing stratum (Bowles 1988). As an example, Fellenius (1991) developed the tip resistance coefficient for deep foundations in cohesionless soils. From this discussion, it is clear that the behavior of a pile tip can be compared to the behavior of a footing with a long stem. Therefore, the possibility of using the ASTM D1194 guidelines to determine the minimum dimensions of a small-scale test to model the behavior of a full-scale pile tip is valid if the soil and confinement conditions at the depth of a pile tip can be simulated.

To convert the load-penetration response measured using small-scale laboratory tests to the response of a full-scale pile tip in case of cohesionless soils, Eqs 1 and 2 are adapted herein after Terzaghi and Peck (1967) and Bowles (1988).

\[
q_{\text{pile}} = q_{\text{model}} \left( \frac{A_{\text{pile}}}{A_{\text{model}}} \right)
\]

\[
w_{\text{pile}} = w_{\text{model}} \left( \frac{2A_{\text{pile}}}{A_{\text{pile}} + A_{\text{model}}} \right)^2
\]

where:

- \( q_{\text{pile}} \) and \( q_{\text{model}} \) = the resistance of the full-scale pile and the small-scale model plate,
- \( A_{\text{pile}} \) and \( A_{\text{model}} \) = the cross-sectional area of the full-scale pile and the small-scale model plate, respectively, and
- \( w_{\text{pile}} \) and \( w_{\text{model}} \) = the vertical displacement of the full-scale pile and the small-scale model plate, respectively.

Using Eqs 1 and 2, the \( q–z \) curve for a pile tip can be calculated using small-scale laboratory tests, provided that the boundary effects in the small-scale model are evaluated and minimized. However, it should be noted that the small-scale laboratory load–penetration curves developed in previous studies were not used in the load transfer analysis to calculate the pile load–displacement behavior, nor were they used to verify full-scale pile load tests.

**Field Testing**

As part of this study, ten vertical SLTs were conducted at bridge sites on instrumented steel HP 254 mm (depth) by 63 kg/m (self-weight) (10 in. by 42 lbs/ft) piles—a partial-displacement pile type that is one of the most commonly used to support bridges in North America (AbdelSalam et al. 2010). However, only two out of these SLTs (named ISU9 and ISU10) were conducted on piles driven in cohesionless soil profiles (which is the focus of this paper). In the State of Iowa, test pile ISU9 was located in Des Moines County, and test pile ISU10 in Cedar County. For the ISU9 test site, the bridge was located south east of Huron close to the Mississippi River. For the ISU10 test site, the bridge was located south of Tipton at the intersection of the Interstate-80 and Iowa River. At both test sites, a 15.8 m (52 ft)
long test pile with 14.9 m (49 ft) embedded length was driven between two anchor piles. Both anchor piles were 18.3 m (60 ft) long with 16.4 m (54 ft) embedded length. The test piles were loaded using a 2000 kN (440 kips) hydraulic jack and the applied load was measured using a 1300 kN (290 kips) load cell. The “Quick Test” procedure outlined in the ASTM D1143 (2006) was used to load test both piles. Loads were applied in five percent increments of the estimated ultimate capacity of the test piles. For each increment, the load was kept constant for a duration of five minutes except for the first and last increments (duration was 10 min). After experiencing excessive vertical displacement at constant load, the piles were unloaded in five equal load decrements. In addition to using four 250 mm (10 in.) displacement transducers to measure the vertical displacement at the head of the test piles, the piles were also instrumented with strain gauges along the shaft and near the tip.

Figures 1 and 2 show the locations of strain gauges installed on each side of the web centerline of test piles ISU9 and ISU10, respectively. The locations of strain gauges presented in the figures were determined considering different soil layers at the two test sites and included at least two gauges near the tip of the piles to quantify the tip resistance. The soil profiles at the test sites were characterized using in situ SPT and CPT tests in addition to several laboratory tests including soil classification, Atterberg limits, and DST. Figures 1 and 2 also show the depths of soil samples taken to perform the mDST and PTR tests at the ISU9 and ISU10 test sites, respectively. At the ISU10 test site, a push-in pressure cell was used to monitor the lateral earth pressure before and after driving the test pile as well as during the SLT.

**SOIL CHARACTERIZATION**

The typical geological formation at the ISU9 test site consists of normally consolidated Alluvium deposits of clay, sand, and gravel deposits. As shown in Fig. 1, four different soil layers were identified during drilling, the first layer consisted of clay deposits extending to 4.8 m (15.7 ft) below the ground surface, followed by sand deposits down to 13.4 m (43.9 ft) underlain by a 2.6 m (8.5 ft) of granular material, and the bottom layer consisted of firm silty clay material. The groundwater table at the time of in situ testing was located at 5.2 m (17.1 ft) below the ground surface. Using laboratory tests conducted on soil samples collected at the ISU9 test site, the top soil layer was classified as low plasticity clay (CL) as per the Unified Soil Classification (USCS), the second layer was classified as well-graded sand (SW), the third as well-graded gravelly sand (SW), and the bottom layer as high plasticity clay (CH). In addition to the basic soil properties, Fig. 1 summarizes the measured CPT tip

**FIG. 1**

Soil classification, CPT tip resistance and shaft friction; corrected SPT and relative density; locations of DST, mDST, PTR, and SG; calculated lateral earth pressure for ISU9.
resistance \( (q_c) \) and shaft friction \( (f_s) \) conducted at the ISU9 test site. Also included in Fig. 1 are the SPT blow counts corrected for the effect of the soil overburden pressure and the corresponding relative density \( (D_r\%) \) estimated using the empirical correlations recommended by Terzaghi and Peck (1967). As can be noticed from the figure, the average value for \( q_c \) and \( f_s \) within the second sand layer equals to 13 300 and 70 kPa, respectively. Moreover, the average corrected SPT and \( D_r\% \) within the second layer is 11 and 38 %, respectively.

The soil profile at the ISU10 test site consists of three soil layers, as presented in Fig. 2, with sandy fill material extending to 4.6 m (15.1 ft) below the ground surface followed by 10.4 m (34.1 ft) layer of coarse sand with gravel and boulders that is underlain by gravelly sand. The groundwater table was located at 3.0 m (9.8 ft) below the ground surface at the time of in situ testing. Using laboratory tests conducted on the soil samples collected from the ISU10 test site during site investigation, the first soil layer was classified as well-graded sand (SW) as per the USCS, the second layer was classified as well-graded sand with seam of gravel and boulders (SW), and the bottom layer as well-graded gravelly sand (SW). As presented in Fig. 2, the gravel and boulders in the second layer forced the termination of the CPT to penetrate into the desired depth so as to avoid the risk of buckling the CPT rods. The figure also summarizes the basic soil properties, the measured \( q_c \) and \( f_s \) from the CPT results, the corrected SPT blow counts, and the \( D_r\% \) of different soil layers at the ISU10 test site. From Fig. 2, the average value for \( q_c \) and \( f_s \) within the second sand layer equals to 11 300 and 56 kPa, respectively. In addition, the average corrected SPT and \( D_r\% \) within the second layer is 37 and 70 %, respectively.

**MONITORING LATERAL EARTH PRESSURE**

In preparation to conduct the mDST at different normal stresses to measure the soil shear strength parameters and the \( t-z \) curves along the soil–pile interface, the range of effective lateral earth pressure acting along the test piles is required which accordingly depends on the angle of soil internal friction \( (\phi) \). The \( \phi \) was calculated based on both the SPT results as recommended by Peck et al. (1974), and CPT results as per Robertson and Campanella (1983). For the soil profiles at the ISU9 and ISU10 test sites, the average calculated value for \( \phi \) based on the SPT and CPT results was around 34 and 40 °, respectively (i.e., the difference is limited to around 15 % where the \( \phi \) values based on SPT are relatively conservative). Furthermore, the \( \phi \) values were measured in the laboratory using the conventional DST then compared with those calculated based on the SPT.
and CPT results, and it was found that the SPT is more close to the DST measurements. Hence, using the correlations based on the SPT to estimate \( \phi \) values is considered acceptable especially if the CPT data is incomplete for some soil layers.

Subsequently, to calculate the effective lateral earth pressure (\( \sigma' \)) acting along the pile length, the lateral earth pressure coefficients (\( K_o \)) were determined using the equations proposed by Jaky (1944) for normally consolidated soil (NC) and following the recommendations of AbdelSalam et al. (2012). Figures 1 and 2, respectively, represent the calculated \( \sigma' \) along the length of ISU9 and ISU10.

The H-pile driving effect on the lateral earth pressure and soil confinement was assessed using two push-in pressure cells (p.cells) at different depths below the ground surface. However, one p.cell was damaged during installation and the only remaining reliable records were acquired from a p.cell that was installed at the ISU10 test site. This p.cell was located at a distance of 200 mm (8.0 in.) from the pile flange and 3.1 m (10 ft) below the ground surface. The pressure cell data was continuously recorded before and during pile driving, during re-strikes, and during SLT. On average, the effective lateral earth pressure measured by the pressure cell at the time of SLT was 22 kPa (3.2 psi), which matches the theoretically calculated value of the lateral earth pressure for the soil using the correlations with SPT and CPT results (see Fig. 2). Therefore, the calculation of the \( K_o \) value based on SPT and CPT tests was insignificantly affected by driving the H-pile and is considered acceptable.

**PILE STATIC LOAD TESTS**

The measured total load–displacement response of ISU9 test pile is presented in Fig. 3 indicating that the pile response was approximately linear up to 521 kN (117.1 kips) and sustained a maximum applied load of 754 kN (169.5 kips). Based on Davison’s criterion (Davison 1972), the pile capacity was determined as 704 kN (158.3 kips). Moreover, the shaft load-displacement for ISU9 test pile was determined by integrating the strain gauge data using Simpson’s numerical integration rule. Then, the load–displacement curve at the pile tip was back-calculated by subtracting the shaft resistance component from the total response measured at the pile head as shown in Fig. 3. For ISU10 test pile, although it was instrumented with strain gauges along its length, the strain gauges did not function properly. This precluded an accurate separation of the shaft and tip resistances from the pile total capacity. Figure 3 also includes the load-displacement response measured at the pile head for ISU10, which produced a Davison capacity of 580 kN (130.4 kips).

Additionally, the load distribution along the length of ISU9 test pile was calculated using the strain gauge measurements recorded at the end of each load increment during the SLT. The load distribution for ISU9 is presented in Fig. 4, which led to the load transferred by shaft resistance of an average rate between 30 and 75 kN/m (2.1 and 5.1 kips/ft). Furthermore, the load transferred to the pile tip during the test ranged from 0 to 210 kN (0 to 47.2 kips), indicating a tip resistance of 28% at the maximum applied load.

**Direct Measurement of Load–Transfer Curves**

**MODIFIED DIRECT SHEAR TEST (MDST)**

The conventional DST was modified in order to directly quantify the \( t-z \) curves at the soil–pile interface for steel H-piles at ISU9 and ISU10 test sites. A square steel plate, having the same grade of the steel piles was fabricated to fit into the lower half of the DST mold with dimensions equal to 100 mm (3.94 in.) in width and 40 mm (1.57 in.) in height. As shown in Fig. 5, the steel plate was placed in the lower half of the shear box while the upper half was filled with appropriate soil material from the

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**FIG. 3** Measured total load-displacement responses for ISU9 and ISU10, and the separated shaft and tip resistance components for ISU9.

**FIG. 4** Load distribution along the pile length from the strain measured at different loads for ISU9.
field. As can be seen from Table 1, the soil samples used in the shear box had the same unit weight, moisture content, and relative density as the soil in the field. The soil samples collected below the ground water table were submerged in de-aired water during testing. The normal stress applied during the mDST was equal to the estimated horizontal stresses ($\sigma_h^0$) acting on the surface of the pile at the depth of the tested sample. The mDST was conducted following the ASTM D3080-98 (2006). During the test, the shear stress versus horizontal displacement was recorded, which represents the stress versus displacement of the soil–steel interface, or the $t$–$z$ curve at the soil–pile interface.

For the ISU9 test site, the conventional and modified DSTs were conducted on soil samples collected at depths of 2.6 m (8.5 ft), 7.0 m (23 ft), and 13.7 m (45 ft) below the ground surface, covering the main soil layers along the pile length. Figure 6 shows the Mohr–Coulomb failure envelops for the soil material and for the soil–pile interface obtained, respectively, from the DST and mDST for both test sites. As expected, the figure shows smaller values for adhesion ($a'$) and friction angle ($\phi')$ for the soil–pile interface when compared to the soil shear strength parameters (i.e., $c'$ and $\phi'$). For all soil layers at the ISU9 and ISU10 test sites, Table 1 summarizes the values of the water content, dry and total soil unit weights, relative density, normal stresses used during laboratory testing, soil shear strength parameters, and soil-pile interface properties. Table 1 also shows the $\delta' / \phi'$ ratio for different soil layers in both test sites, with an average value equal to 0.808, which is very close to the suggested value of 0.8 reported by Bozozuk (1972) for similar soil type.

Figure 7 presents the measured $t$–$z$ curves for the soil–pile interface obtained using the mDST for ISU9 test pile at normal stresses corresponding to the estimated lateral earth pressure in the field (see normal stresses range in Table 1). As can be seen in Fig. 7, the $t$–$z$ curves measured from mDST provided a good match compared to those back-calculated from the strain gauge readings obtained from the SLT data of ISU9—especially when comparing the maximum shear stress for the top and bottom soil layers, where the difference did not exceed 5 %. The figure also shows that the mDST-measured $t$–$z$ curves of the soil–pile interface corresponded to the estimated lateral earth pressure in the field, with an average value equal to 0.808, which is very close to the suggested value of 0.8 reported by Bozozuk (1972) for similar soil type.

![Figure 5](image)

**TABLE 1** Soil and soil–pile interface parameters measured using in situ tests.

<table>
<thead>
<tr>
<th>Test ID</th>
<th>Depth (m)</th>
<th>$w_%$</th>
<th>$\gamma_{dry}$ kN/m$^3$</th>
<th>$\gamma_{bulk}$ kN/m$^3$</th>
<th>D$_2/%$</th>
<th>$P_n$ (kPa)</th>
<th>$c'$ (kPa)</th>
<th>$\phi'$ Deg.</th>
<th>$a'$ (kPa)</th>
<th>$\delta'$ Deg.</th>
<th>$\delta' / \phi'$</th>
<th>$E^s$ (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>ISU9</td>
<td>2.6</td>
<td>24</td>
<td>18.8</td>
<td>23.4</td>
<td>163$^a$</td>
<td>10</td>
<td>20$^e$</td>
<td>30</td>
<td>n/a</td>
<td>n/a</td>
<td>9.7</td>
<td>14.2</td>
</tr>
<tr>
<td></td>
<td>7.0</td>
<td>23</td>
<td>18.5</td>
<td>22.9</td>
<td>38$^a$</td>
<td>60</td>
<td>100$^e$</td>
<td>128</td>
<td>0</td>
<td>37</td>
<td>1.6</td>
<td>25.5</td>
</tr>
<tr>
<td></td>
<td>13.7</td>
<td>18</td>
<td>21.4</td>
<td>25.2</td>
<td>68$^a$</td>
<td>108</td>
<td>206$^e$</td>
<td>295</td>
<td>0</td>
<td>39</td>
<td>0.69</td>
<td>28.7</td>
</tr>
<tr>
<td>ISU10</td>
<td>2.1</td>
<td>15</td>
<td>17.6</td>
<td>20.2</td>
<td>40$^a$</td>
<td>25$^e$</td>
<td>40</td>
<td>60$^e$</td>
<td>3.6</td>
<td>37</td>
<td>28.7</td>
<td>0.76</td>
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<tr>
<td></td>
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<td>20.3</td>
<td>24.0</td>
<td>70$^a$</td>
<td>60$^e$</td>
<td>120</td>
<td>170</td>
<td>0</td>
<td>31.5</td>
<td>1.06</td>
<td>24.0</td>
</tr>
<tr>
<td></td>
<td>13.7</td>
<td>7</td>
<td>22.2</td>
<td>23.7</td>
<td>76$^a$</td>
<td>120$^e$</td>
<td>220</td>
<td>310</td>
<td>0.2</td>
<td>41</td>
<td>33.5</td>
<td>0.81</td>
</tr>
</tbody>
</table>

$^a$Calculated based on SPT data after Terzaghi and Peck (1967).
$^b$Normal stress applied on soil during conducting the DST and mDST tests.
$^c$Represents the normal stress corresponding to measured lateral earth pressure and selected for TZ-mDST analyses.
$^d$Value of the $Su$ (kPa) for clay calculated based on CPT results using correlations from Schmertmann et al. (1978), assuming $N_k = 15$.
$^e$After Kulhawy and Mayne (1990) using $M = 8.25 (q^r - \sigma_{vo})$.  

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interface are nonlinear and that the ultimate stress and stiffness increase as a function of depth. For ISU10 test pile, the DST and mDST tests were conducted following the same procedures used for ISU9 at different normal stresses as summarized in Table 1. For the ISU10 test pile, similar observation to those observed for ISU9 were noticed (complete information can be seen in the report by Ng et al. 2011), which validates the reliability of the $t$–$z$ curves measured using the mDST test.

In the Load Transfer Analysis section of this paper, the $t$–$z$ curves measured utilizing the mDST were used to quantify the shaft load–displacement and shaft load distribution responses for ISU9 and ISU10 test piles. The $q$–$z$ curve required to include the tip resistance component was obtained as detailed below.

**FIG. 6** Mohr–Coulomb failure envelope for the soil and the soil–pile interface obtained using DST and mDST at various depths for ISU9 and ISU10.

**FIG. 7** Comparison between the $t$–$z$ curves from mDST and strain gauges (SG) for ISU9.

**PILE TIP RESISTANCE TEST**

As previously discussed and for practical purposes, the load-penetration response (i.e., $q$–$z$ curve) of a pile tip installed in sand deposits can be modeled using a rigid footing, which was validated using a finite element analysis that is described in the next section. Based on this concept, a penetration test called the PTR is introduced herein, which can measure the $q$–$z$ curves in the laboratory. The PTR mainly consists of a steel box that contains the soil extracted from below the pile tip, a small-scale steel plate to represent the rigid footing, a loading frame to apply vertical load on the steel plate, and two dial gauges to measure the vertical load and displacement on top of the steel plate (see Fig. 7).

For the soil medium below the pile tip to be efficiently represented in the PTR test, the square shaped DST box was used after changing its dimensions to follow the ASTM D1194 standards. The ASTM standards indicate that the size of the steel plate used to represent the rigid footing in a soil box should be equal to or less than $1/4$ the box width. Therefore, assuming an influence depth not less than two times the size of the steel plate, the minimum depth below the largest possible steel plate in the DST box should not be less than 50 mm. However, the preliminary internal dimensions of the square shaped DST box were 100 mm (3.94 in.) and 40 mm (1.57 in.) in width and height, respectively. For that purpose, a 20 mm (0.79 in.) spacer steel plate was added on top of the shear box and therefore the total height was increased to 60 mm (2.36 in.).

For the small-scale steel plate represents the rigid footing in the PTR test, and given that the size of this steel plate relative to the dimensions of the PTR soil box may affect the test results,
two steel plates with different sizes were fabricated and tested. The width of the first steel plate, P1, was 25 mm (1 in.) and the width of the second plate, P2, was 17.5 mm (0.7 in.). The ratio of the width of the soil box to the width of the model plate for P1 and P2 was 4.0 and 5.7, respectively. The ratio of the box depth and the width of the model plate for P1 and P2 was 2.4 and 3.4, respectively. These width and depth ratios were designed to provide an influence zone below the plate tip ranging from 2D to 4D, where D is the steel plate size. Figure 8 represents the fabricated steel plate P2 and the modified DST box dimensions that were used in the laboratory test to measure the q–z curve under the pile tip for ISU9 and ISU10.

During testing, the steel plates were embedded 10 mm (0.40 in.) into the compacted soil inside the PTR box. For the PTR test representing ISU9 test pile, the soil sample collected at a depth of 15 m (49.2 ft) (i.e., the depth of the pile tip) was compacted inside the box to match the estimated Dr of 68 %. Note that the pile driving and its effect on the soil relative density, lateral earth pressure, and soil confinement were all previously considered herein based on pressure-cells reading, SPT, and CPT results. For the PTR test representing ISU10 test pile, the sample collected at the pile tip was compacted to achieve a Dr% of 76 %. Following the testing procedures provided by Yasufuku and Hyde (1995), the loads were applied in incremental steps with each step providing one-twentieth of the estimated maximum stress. For each loading increment, the corresponding vertical displacement was measured until the rate of settlement was less than 0.01 mm/min (0.0004 in./min).

Normalized curves can allow for basic comparison between tests performed with different dimensions and material properties (Gavin and Lehane 2007). Hence, having similar normalized load-penetration curves for ISU9 and ISU10 should indicate minimal boundaries effect from the soil box and model plate on the measured response of the soil (Cerato et al. 2006; Loukidis 2006). Consequently, the PTR test results collected for ISU9 and ISU10 using P1 and P2 were normalized and presented in Fig. 9 as a relation between Δ/D and σn/σv (where Δ = the measured vertical displacement of the steel plate;
D = the steel plate size or width; $\sigma_n$ = the applied normal stress on the steel plate during the PTR test; and $\sigma_v$ = the overburden vertical stress at the pile tip in the field. It was observed that the normalized relation between $\Delta/D$ and $\sigma_n/\sigma_v$ acquired using P1 for ISU9 and ISU10 show a difference of approximately 18%. Ideally, there should not be a noticeable difference between ISU9 and ISU10 since the responses are normalized and the soil type, depth, and overburden stresses are similar at both test sites, which indicates the possibility of the PTR box boundaries influencing the results when using the larger steel plate (P1). When using the smaller plate, P2, the difference between the normalized curves obtained for ISU9 and ISU10 was reduced to less than 4%. Therefore, the PTR box boundaries have insignificant effect on the model plate behavior using P2, which was also validated using a finite element model described in the next section. The PTR results obtained using P2 were used to calculate the $q$–$z$ curves required for the $t$–$z$ analysis to characterize the behavior of ISU9 and ISU10.

As shown in Fig. 10, the measured load–displacement curves for the model plate (P2) were converted to load–penetration curves ($q$–$z$ curves) for the full-size piles at ISU9 and ISU10 using Eqs 1 and 2. It can be seen from the figure that the maximum normal bearing forces specified from the $q$–$z$ curves for ISU9 and ISU10 were about 68.5 kN (15.4 kips) and 62.8 kN (14.1 kips), respectively, which correspond to approximate vertical displacements of about 5.0 mm (0.19 in.) and 7.0 mm (0.27 in.).

### PTR VALIDATION USING FINITE ELEMENT

To validate the use of the PTR test to simulate the pile tip behavior (i.e., $q$–$z$ curve), a finite element (FE) analysis representing the full-scale test piles ISU9 and ISU10 was conducted. Initially, the accuracy of the FE analysis was validated by comparing its results with load distribution along the pile and load-displacement curves from the SLT measurements. Then the load–penetration at the pile tip (or $q$–$z$ curve) calculated from the FE analysis was compared with that measured using the PTR laboratory test.

An axisymmetric model based on the Mohr–Coulomb constitutive relation for the soil material was utilized in the FE analysis using computer program PLAXIS 7.2 (Brinkgreve and Broere 2004). The constitutive parameters were established for the soil material based on the values provided in Table 1. Additionally, the steel H-piles (i.e., test piles ISU9 and ISU10) were represented in the axisymmetric model assuming that the cross-section is circular. However, the model pile diameter deemed to provide the same surface and cross-sectional areas of the actual pile along the shaft and at the tip, respectively. The test piles were modeled as a non-porous linear–elastic material with elastic modulus equal to 200 GPa ($2.9 \times 10^4$ ksi) and Poisson’s ratio equal to 0.2. For the soil–pile interface along each soil layer in the FE models, the values of the strength reduction factor for interfaces ($R_{inter}$) were assumed based on the recommendations provided by Brinkgreve and Broere (2004). For the FE model that represents ISU9 (i.e., the FE-ISU9 model), Fig. 11(a) shows the boundary conditions, the soil–pile interface elements used, and the deformed mesh. Figure 11(b) provides a distribution of the total displacement corresponding to a prescribed vertical displacement of around 10% of the full-scale pile width, and Fig. 11(c) demonstrates the stress distribution along and below the pile tip.

When comparing the calculated load–displacement responses at the pile head from the FE-ISU9 model with that measured from the SLT results, it was found that the difference is limited to about 8.5% at the plastic portion of the curve (see Fig. 12(a)). For the FE model that represents ISU10 (i.e., the FE-ISU10 model), Fig. 12(b) also shows a difference limited to only 2.5% between the calculated and the measured load–displacement responses at the plastic portion. Additionally, the load distribution response along the pile length from the FE model was compared with that calculated from the strain gauges measurements, and the difference was insignificant. Given the acceptable accuracy of the FE model for ISU9 and ISU10, its results were used to validate the PTR measurements. Consequently, the $q$–$z$ curve calculated using the FE model was compared with the that measured using the PTR test. As shown in Fig. 12(c) and 12(d) for test piles ISU9 and ISU10, respectively, there is a reasonable agreement between the calculated and measured $q$–$z$ curves.

To investigate the effect of the box boundaries on the measured $q$–$z$ curve, another FE analysis simulating the PTR test (the FE–PTR model) was conducted. In the FE–PTR model, the steel plate P2 was modeled as a non-porous linear–elastic material and its behavior was manually adjusted to match the measured response in the actual PTR test. The results of the FE–PTR model showed that the maximum vertical stresses
induced in the soil were concentrated directly below the steel plate and degraded to less than 10% of the applied load toward the base bottom boundary of the modeled soil box. The maximum horizontal shear stress was concentrated at the vertical soil–plate interface, and then degraded toward the vertical side boundary of the shear box to less than 1% of the maximum shear. Moreover, the vertical soil displacement under the plate did not exceed 1.5 D, where D is the steel plate size, which is less than the minimum required depth of the soil box suggested by Bowles (1988) and Houlsby et al. (1988). Based on the FE analysis and the laboratory measurements, it was confirmed that the boundaries of the used soil box in the PTR test for P2 are minimally affecting the vertical load-displacement response of the tested model.

### Load-Transfer Analysis

In this section, the \( t-z \) and the \( q-z \) curves measured utilizing the mDST and the PTR tests were used to model the shaft and tip resistance components of the test piles ISU9 and ISU10 in the load-transfer analysis (\( t-z \) model) using TZPILE v.2.0 (Reese et al. 2005). This software allows users to manually input...
the load-transfer data. Hence, the measured \( t-z \) curves from the mDST results were inserted in the model to represent the soil–pile interface stiffness within each soil layer, including cohesive and cohesionless layers. Also, the measured \( q-z \) curve from the PTR test was inserted under the model pile to represent the tip resistance component. The load–displacement response as well as the load distribution along the pile length for both test piles were calculated using the \( t-z \) model and then compared with field results from the SLTs.

**MODEL DESCRIPTION**

To simulate ISU9 test pile in the \( t-z \) model, given that the pile is HP 254 by 63 (HP 10 by 42) with 14.9 m (49 ft) embedded length, the pile was divided into 50 segments and was represented by a series of elastic springs. The stiffness of these elastic springs was assumed constant and taken as \( AE = 3.596 \times 10^8 \) kN (0.808 \( \times 10^8 \) kips), where \( E \) and \( A \) are the pile elastic modulus and the cross-sectional area, respectively. For the three soil layers surrounding ISU9 pile shaft, the soil–pile interface of each layer was represented using non-linear springs (\( t-z \) curves). In addition, a non-linear spring characterizing the soil behavior at the pile tip (\( q-z \) curve) was also used. The three measured \( t-z \) curves from the mDST results (previously shown in Fig. 7) were manually inputted into the TZPILE software to represent the soil-pile interface for each soil layer (including the upper clay layer with \( t-z \) curve at 2.6 m). For the tip resistance component, the PTR-measured \( q-z \) curve (previously shown in Fig. 10) was used in the model.

Similarly, ISU10 test pile (pile with same type, size, and embedment length of ISU9) was represented in the \( t-z \) model as shown in Fig. 13, which also provides additional details for the used \( t-z \) model. For the two main soil layers surrounding ISU10, the \( t-z \) curves measured from the mDST and the \( q-z \) curve measured from the PTR test are shown in Fig. 13. Since the second soil layer (Layer 2) has a relatively large thickness, it was represented in the model by two different \( t-z \) curves for more accuracy.

For both \( t-z \) models representing ISU9 and ISU10, load increments similar to those applied during the SLTs were applied at the modeled pile head. Finally, according to Seo et al. (2009), the possibility of soil plugging between the flanges was ignored in the \( t-z \) model since most of the shaft and the tip of the test H-piles were embedded in sandy soil profiles.

**LOAD–DISPLACEMENT CURVES**

For ISU9 test pile, the response of the full-scale test was evaluated using two different \( t-z \) analyses. Initially, the analysis was conducted using only the \( t-z \) curves measured from the mDST (i.e., ignoring the tip resistance component), which was called TZ-S-mDST. This analysis was conducted to compare the predicted response with the shaft load–displacement relationship calculated using the strain gauge data. The analysis was then repeated using both the \( t-z \) and \( q-z \) curves (TZ-T-mDST) to compare the predicted response with the measured total load–displacement relationship (note that a prescribed maximum vertical displacement of around 10 % of the full-scale pile width was defined in the model). Figure 14 shows the calculated shaft and total load–displacement relationships compared with the measured responses. As seen in the figure, the calculated shaft load-displacement curve using the TZ-S-mDST model matched the measured response along the initial portion of the curve (within the initial loading stage) with difference limited to 3.3 %. When the effect of the tip resistance, using \( q-z \) curves developed from the PTR test, was included in the analysis (i.e., using model TZ-T-mDST), the calculated total load–displacement relationship provided a good match of the
measured total response along the initial and plastic portions (plastic portion represents the at failure loading stage) of the curve and resulted in a difference of 4.8 % in the capacity based on Davison’s criterion (see Table 2).

In addition to comparing the load–displacement response, Fig. 15 presents the calculated load distribution along the pile length at different applied loads of 754 kN (170 kips), 567 kN (127.5 kips), and 307 kN (69 kips) compared to the measured pile response from strain gauges (SG) readings. The 754 kN represents the maximum applied load in the field, which corresponded to the pile behavior at failure loading stage; the 307 kN represented the pile behavior within the initial loading stage and the 567 kN represented the transition between the initial and failure loading stages. At failure stage, it can be seen from Fig. 15 that the TZ-T-mDST is reasonably close to the measured load distribution along the pile shaft as well as a very good agreement with the pile tip resistance, when compared with the loads obtained from SG readings. Hence, the TZ-T-mDST model accurately captured the load-displacement response and load distribution along the pile length at the failure stage for ISU9 test pile. However, at initial and transitional loading stages of 307 and 567 kN, it is worth noting that a relatively larger difference was observed at the pile tip. This difference could be attributed to the fact that the soil around the pile tip is being sheared with constant volume, whereas the PTR was conducted under constant normal stress. Thus, the response of dilative or contractive volume change during shear may have slightly affected the measured shearing resistance (after AbdelSalam et al. 2012). The greater constraint of soil movements in the case of the soil around the pile tip may cause relative, but acceptable, differences in the resistance compared to the PTR.

For ISU10 test pile, the total load–displacement response was calculated using the t–z model based on the t–z and q–z curves obtained from the mDST and PTR (see Fig. 16). As shown in the figure, the slope of the initial portion of the calculated load–displacement response is about 12 % stiffer than the measured response, while the capacity estimated using the Davison’s criterion from the TZ-T-mDST analysis was almost equal to the pile capacity estimated from the measured response (see Table 2). Therefore, the TZ-T-mDST analysis resulted in a reasonable accuracy compared with the measured load–displacement response for ISU10. Based on comparisons with measured responses for ISU9 and ISU10 test piles, it is evident that the load transfer analysis, based on the laboratory

<table>
<thead>
<tr>
<th>TABLE 2</th>
<th>Summary of the t-z model findings compared to the pile load test results.</th>
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<tbody>
<tr>
<td>Test ID</td>
<td>Model</td>
</tr>
<tr>
<td>ISU9</td>
<td>SLT</td>
</tr>
<tr>
<td></td>
<td>TZ-S-mDST</td>
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<td></td>
<td>TZ-T-mDST</td>
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<tr>
<td>ISU10</td>
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<td>TZ-T-mDST</td>
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*Difference between the load-displacement curves adapted from the t-z analyses and the pile measured response during SLT.
measured $t-z$ and $q-z$ curves, respectively, using the mDST and PTR tests, provided an effective means of predicting the response of vertically loaded steel piles in cohesionless soils.

Summary and Conclusions

A laboratory mDST and PTR test were proposed to improve the $t-z$ analysis by measuring the load transfer curves. The mDST and PTR provide a measurement for the $t-z$ and $q-z$ curves, respectively, which are required for the $t-z$ analysis and representing the shaft and tip resistances of piles. As part of this study, two static load tests were conducted on instrumented, partial-displacement, steel H-piles (ISU9 and ISU10 test piles) driven in cohesionless soil profiles. The soil investigation program for both test sites included in situ tests such as SPT, CPT, and push-in-pressure-cells, as well as laboratory tests such as soil classification and DST. The $t-z$ analysis based on the mDST and PTR measurements provided the shaft load–displacement response (TZ-S-mDST) as well as the total load–displacement response (TZ-T-mDST) and the results were compared to the field measured responses. A summary of the major findings is presented below.

- The laboratory measured $t-z$ curves obtained using the mDST were compared to those measured in the field using the strain gauges readings, and it was found that the difference at the maximum shear stress did not exceed 5%.
- The PTR test was conducted to measure the $q-z$ curves using different sizes of small-scale model plates, and after normalizing the load-penetration curves for ISU9 and ISU10, it was found that using the model plate P2 minimized the boundaries effects. In addition, an FE model was utilized to validate the $q-z$ curves and to assess effect of the PTR box boundaries, and it was confirmed that the PTR test results are acceptable.
- For ISU9 test pile, the predicted load–displacement relationship for the shaft using the TZ-S-mDST model matched the measured response.
- When adding the tip resistance component to the ISU9 analysis (i.e., TZ-T-mDST), the model provided a good match of the measured load distribution as a function of depth as well as the total load–displacement curve, with only 3.3% softer response along the elastic portion of the curve and 4.8% lower capacity at the plastic portion.
- The TZ-T-mDST model also led to an acceptable prediction of the measured total load–displacement response for ISU10 with difference limited to about 12% along the elastic portion of the curve and about 1.5% at the plastic portion.
- Based on overall response predictions for the two test piles, the load transfer analysis, with $t-z$ and $q-z$ curves measured using the mDST and PTR tests, respectively, has proven to provide satisfactory modeling approach which offers a simple and cost-effective procedure to predict the response of partial-displacement steel H-piles driven in cohesionless soils; however, it is recommended to conduct a further study on this model when a larger database is available and compare it with existing models.

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