**Validity of Dynamic Load Test (DLT) on H-piles**

Thai Nguyen, Ph.D., P.E.[1]

## ABSTRACT

The author evaluated H-pile load test data from three (3) sources: 1) Florida database of 641 H-piles from public bridges with Dynamic Load Test (DLT), 2) FHWA database of 23 H-piles having both DLT and Static Load Test (SLT), and 3) a private sector project with detailed DLT and SLT results. The need for this study originated from occasional very large divergence between DLT results and static analyses results of H-piles in several projects. Upon detailed evaluations it is concluded that the observed large divergence is not due to correlations discrepancy, but rather the invalidity of the DLT results for certain H-piles, especially for 1) long H-piles with penetration depth exceeding 30 m, the piles may behave as plugged in the static condition, but as unplugged during dynamic loading event; 2) short H-piles, the conventional lumped CASE damping factor in conventional range should not be relied on for these piles.

**Keywords:** H-pile, Dynamic Load Test, Plug, Unloading Skin Friction, Side Resistance.

## 1 INTRODUCTION

High strain dynamic pile load tests, commonly known as Dynamic Load Test (DLT), has been proven to correlate well with Static Load Test (SLT) bearing capacity of displacement type piles: Hussein and Rausche (1991) reported an average bias factor of $\lambda$= 0.99 where 80% of the piles are either displacement piles or small diameter drilled shafts; Goble et al. (1980) reported an average $\lambda$=1/0.96=1.04 for predominantly displacement closed end pipe piles. Likins and Rausche (2004) compiled 303 piles, most of them are displacement piles from 3 previous studies and reported an average $\lambda$=1/0.983=1.02. As such, the DLT practice on displacement piles have been carried out on a routine basis (Hussein and Goble, 2004; Grävare et al., 2014).

On the other hand, H-piles are also very popular in some parts of the world. For instance, in some parts of Florida (USA), due to the extreme geological spatial variations within short distances – even within a single pile cap, unexpected low resistance soils or pockets within limestone formations result in unforeseen pile splices. In these cases, steel piles have several advantages over concrete piles including the ability to bring pile sections of various lengths to the site, ease of transportation and handling, and quick welding for pile splicing to accommodate additional pile length (Kuhns et al., 2003). As such, DLT monitoring of H-piles have also been carried out on a routine basis, for example Hussein et al., 2009. For the purpose of this paper, the H-piles are categorized per penetration depths as:

i) Short piles: penetrations of less than 15 m;

ii) Long piles: penetrations of more than 30 m. This length is relative, as it is related to side resistances. Long piles typically have a pronounced dynamic unloading effect (to be discussed).

iii) Medium piles: in between the two ranges above, where DLT capacities can sometimes correlate relatively well with static bearing capacity

This paper evaluates the DLT data and suggests that DLT method is not a suitable tool to quantify the static pile capacities for either long or short H piles.

## 2 H-PILE DATABASES

The Florida data source used in this study includes 641 H-piles having Dynamic Load Test End of Initial Drive (DLT EOID) performed on each, as well as 78 records on setchecks and re-drives. Similarly, a Florida private project with both DLT and SLT results are discussed.

The FHWA data source was obtained from FHWA DFLTD database (Kalavar and Ealy, 2000) (https://www.fhwa.dot.gov/software/research/infrastructure/structures/bridges/dfltd/index.cfm). It includes 65 H-piles, of which 23 piles had both SLT and DLT.

The following sections will examine the pile driving hammers, pile plug conditions, and DLT results from the Florida and FHWA data sources.

The most popular H-pile shapes were found to be HP12x53, 14x73, 14x89, 14x117, and 18x135. Their true areas are from 100 $cm^2$ to 257 $cm^2$, with a median of 168 $cm^2$.

The Pile Factored Design Loads in the database ranges from 450 to 1420 kN. The Nominal Bearing Resistance (NBR) per Load and Resistance Factor Design (LRFD) method ranges from 600 to 2220 kN.

Two hammer types were used: i) OED or Closed End Diesel (CED) hammers, ii) External Combustion Hydraulic (ECH) hammers with heavier ram weights and much less stroke heights than the diesel hammers. The ram size ranges from small (D8-42 with 770 kg ram)

[1] Chief Geotechnical Engineer, H2R Corp, Pinellas Park, Florida, USA

to very large, i.e., oversize hammers (D30-42 with 3000 kg ram).

With H-piles, the concern of exceeding yield stress is not as serious as with concrete driven piles. Therefore, a number of large hammers were accepted on several H-pile projects, resulting in less than the specified 36 blows per 0.3 m (blows per 0.3 m equal to blows per foot - bpf) for acceptance at End of Initial Drive (EOID) per Florida Department of Transportation (FDOT) Specification (FDOT, 2021). Large hammers more likely cause the piles to behave unplugged during driving – as discussed in detailed in Section 3 below – which will affect the validity of the DLT method on H-piles.

## 3 H-PILE PLUG CONDITIONS

This section presents a review of the H-pile plug conditions in different soil types followed by the authors' examination of the DLT results due to different state of plugs during driving versus static conditions.

### H-pile Plug Condition in Clay

Tomlinson (1994) reported a case study on the site of the Hartlepools Nuclear Power Station. The data indicates that the H-piles were plugged solidly with clay.

Similarly, Hannigan et al. (2006) recommends the box configuration in cohesive soils. However, in stiff clay or stiff glacial till, Tomlinson (1994) suggested that in the upper part of the pile, the shaft resistance would occur only on outer flange surfaces. This opinion is also cited in Hannigan et al. (2006).

### H-pile Plug Condition in Sand

Tomlinson (1994) reported several case studies where minimal or no plugging had occurred over the full depth of the pile during driving into sands.

However, in static load conditions, this is not the case. Coyle and Ungaro (1991), based on 14 static load tests of H-piles installed predominantly in sandy soils, recommend using the half plug configuration for both the toe end bearing and the shaft resistance.

### H-pile Plug Condition in Rock

The plug condition in rock is typically not discussed. The reason is that the toe capacity for H-piles driven to bed rock is usually governed by the pile's structural strength, thus the ultimate end bearing is limited by 0.9* $f_y$ * $A_{true}$, where $f_y$ is the steel yield strength – typically 250 MPa or 345 MPa as A36 or A50 are the most popular grades for H-piles. For a typical true area of $A_{true}$ = 168 $cm^2$ the end bearing limit is 3760 to 5220 kN, providing more structural capacity than most NBR values that the Design Engineers specify. Furthermore, end bearing piles generally have minimal side resistance, which leads to minimal dynamic unloading and therefore may not subject to most of the discussion in this paper.

### H-pile Plug Condition during Driving

Hannigan et al. (2006) cited Holloway and Beddard (1995) in reporting that the hammer blow size (impact force and energy) influenced the dynamic response of the soil plug. With a large hammer blow, the plug will "slip" under the dynamic event whereas under a smaller hammer blow, the pile encounters a toe resistance typically of a plugged condition – which may better resemble the true condition under static condition.

For diesel hammers, the starting blow for a Beginning of Restrike (BOR) has very low transferred energy (EMX), which is approximately 40 to 60% the EMX of the next blow. We call this Blow #0 where the calculated stroke height is displayed by the DLT software as 0 (due to lack of time interval to calculate the stroke height).

For other pile types (e.g., concrete piles), this blow may be ignored or deleted as it may have been the blow where the diesel hammer had stalled. It is also the blow that does not mobilize the pile capacity due to its lower hammer force compared to the next immediate blow, i.e., Blow #1 – where the hammer force (FMX) and the maximum Case resistance (RMX) are usually the highest.

However, in several H-pile projects, the authors observed that despite having a lower EMX, Blow #0 recorded higher RMX than Blow #1. The most plausible reason for this is that the pile is behaving as a plugged pile on Blow #0.

The observed behavior suggests that during initial drive, the toe area behaves unplugged most likely due to pile acceleration and effects of soil inertial forces. However, for the initial blow of the restrike - after a sufficient period has elapsed - a plugged toe due to setup (freeze) of the material along the sides of the pile (i.e., increase in side-resistance) develops. Of interest was the ease with which that condition was lost almost immediately after Blow #0. As the driving continued, the material in proximity to the toe area is suspected to erode as the piles returned to their unplugged behavior.

One such example is the Hypoluxo project site (Figure 1 and 2). The above behavior can be seen by examining the Upward Wave (Wave Up or WU) and Downward Wave (WD) forces shown in Figure 1. In the case of Blow #0 little tension reflection exists (blue dashed WU line at 2L/c), while Blow #1 (solid red WU line at 2L/c) shows large tension reflection (i.e., small toe resistance), comparable to the EOID blow in Figure 1.

As such, when the Consultant plotted all the DLT RMX resistances versus elevations (Figure 2), the resistances on BOR blow #0 were much higher than the resistances on all other blows, despite having similar side resistances among the blows (based on the similarities of the WU magnitudes within the side resistance zones) for all blows as shown in Figure 1.

The above discussion regarding H-pile plug condition supports the opinions of Hannigan et al. (2006) and Holloway and Beddard (1995) that soil plug depends on hammer dynamic responses, as well as the blow sequence (EOID or BOR). Therefore, the DLT estimated capacity may – unfortunately – not be representative of the true static capacity.

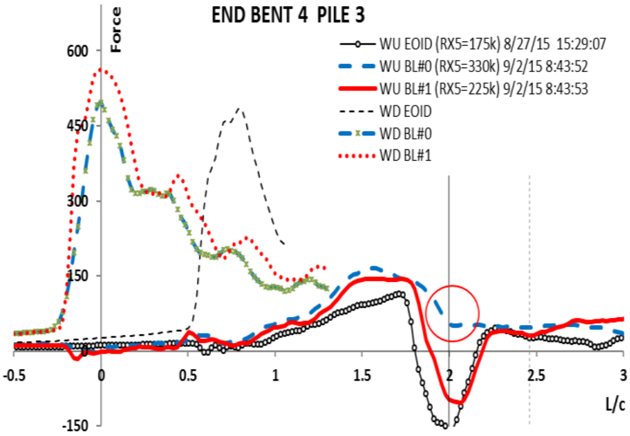
## 4 DLT SIDE RESISTANCE UNLOADING

During driving of long piles, by the time the upward wave returns from the pile tip to the DLT sensors at the pile top, the pile has already experienced unloading. That is, the upper portion of the pile has undergone elastic release after initial impact, prior to the return of the stress wave from the pile tip. This response is evident when instrumentation attached near the pile head records negative velocity prior to 2L/c time. In those instances, the Maximum Case Method RMX capacity can significantly under-predict the true static capacity. The deeper and the more the unload resistance, the more the Case Method under-predicts the true capacity.

Severe Side Resistance Unloading (not to be confused with negative skin friction or down-drag) typically happens to friction piles with more than 30 m of penetration. Severe unloading tends to happen frequently to H-piles due to:

i) Small pile cross-sectional areas result in a tendency for the pile to cut through soils and weak rocks, thus, very deep pile penetrations;
ii) Increased elastic shortening due to reduced stiffness (EA/L: A is small and L is large) for long piles resulting from compression loading at the pile top, while the pile toe may move very little. During the elastic release ($t < 2L/c$), the top section of the pile will experience side resistance unloading.

Because of the described unloading phenomenon, as a long pile is driven deeper, the DLT total side resistance does not increase very much (positive side resistance at lower depths is being cancelled by negative side resistance at upper depths). This is illustrated in Figure 3 for Eller Drive project where the piles are three time longer and have significantly more side resistance than those at the Hypoluxo site. At Eller project site, the DLT resistances estimated using the RMX approach show very little increase in pile capacity with depth and is significantly less than estimated values based on SPT Soil Borings TB7L, 8L, and 9R using FB-Deep software.




Note: The distance from WD peak to WU valley on Blows #0 and #1 is longer due to longer pile length after splicing.

**Figure 1. WU, WD – Hypoluxo Site in Florida**

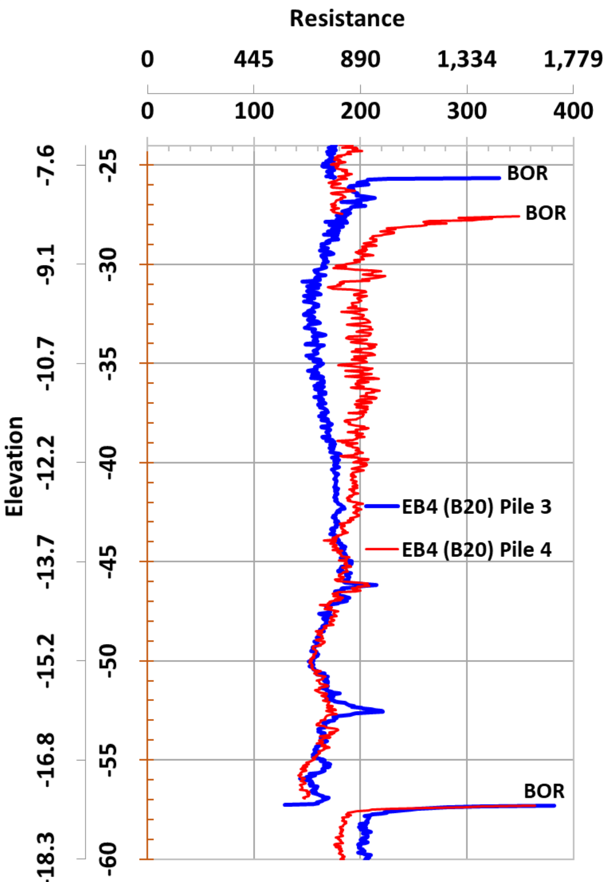



**Figure 2. DLT Resistance vs. Elevation – Hypoluxo**

In Figure 4.a, Blow #1 with pile tip elevation at -35 m exhibits a significant amount of unloading: by the time the wave reflects back from the segments near the pile toe, the pile is already in the unloading phase, i.e., the pile top velocities are well into the negative zone prior to the 2L/c time. Consequently, due to unloading the pile integrity was incorrectly evaluated as low (integrity indicator BTA<80%) for Blow #1. However, on subsequent blows, the side resistance setup had been eroded, pile velocity did not move much into the negative zone, unloading effect had been diminished, and the DLT integrity BTA returned to 100%, as shown in Figure 4.b.

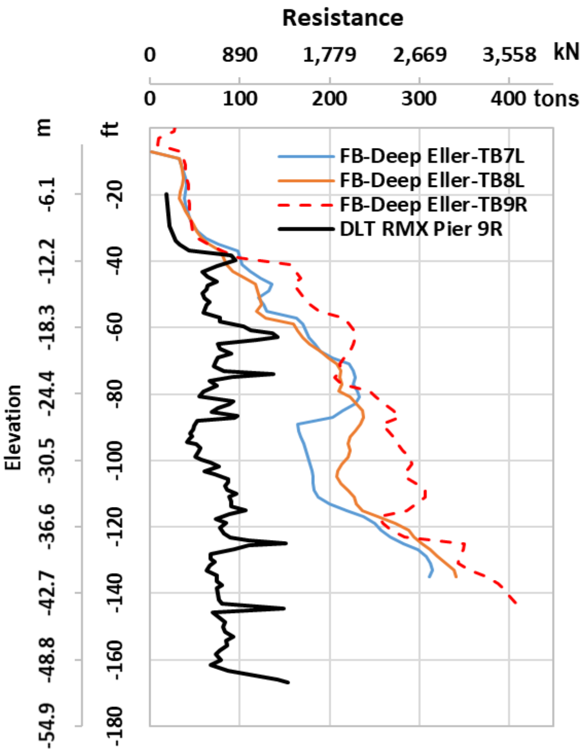



**Figure 3. Eller Drive - DLT vs. Predicted Resistances**

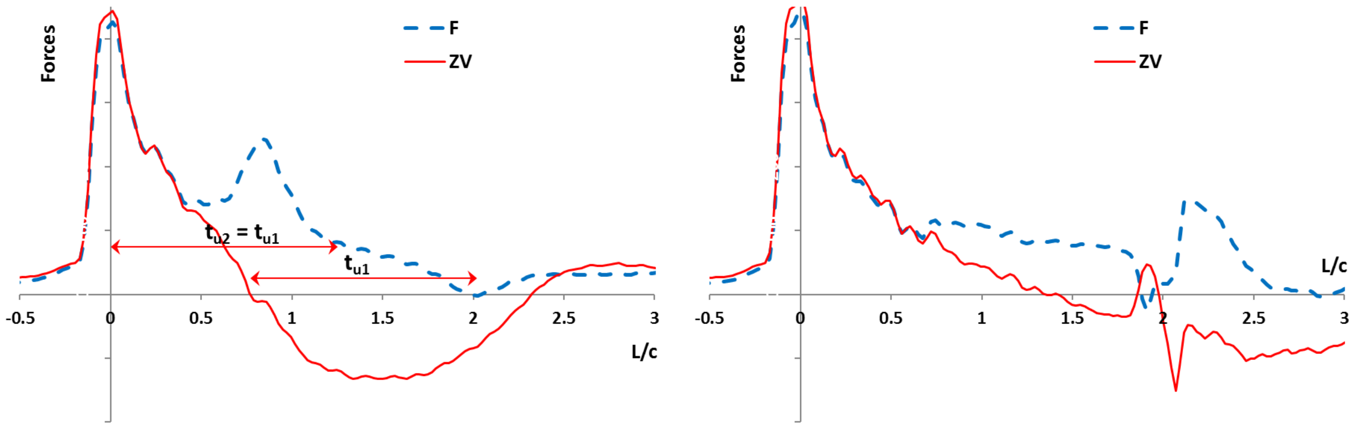
Figure 5.a indicates that the hammer input forces (FMX) were all about the same for 4 blows: EOID, BOR Blows #1, #5, and #10. Evident from Figure 5.b, it can be seen the pile side resistance had a very substantial freeze (setup gain) at BOR Blow #1 (WU – blue dashed line). However, that substantial amount of freeze was completely lost by blow #10. In fact, most of the freeze had disappeared by blow #5.

The DLT RMX pile capacity is around 1290 kN for both EOID and BOR (analyzed at pile tip elevation of -35 m. This is a very large diversion from FB-Deep estimates of 2220 kN as shown in Figure 3 despite the built-in conservatism within the FB-Deep program.

Estimated, i.e., extrapolations (dashed lines) of the WU as in Figure 5.b by extending the WU slopes indicates that a large portion of the side resistance had been lost.

Signal Matching Analysis with a match quality of 1.24 performed by the authors for BL#1 indicates 1700 kN for side resistance and 1840 kN for total pile capacity and a proposed lumped Case damping $J_C$ of 0.15. It's noted that the Signal Matching Analysis also indicates the pile segments near the toe moved only 1.5 mm, which is significantly less than the movement needed for the full ultimate end bearing typically to be mobilized.

In summary, it may not be appropriate to use the DLT RMX method to estimate static resistance, especially when pile penetration is deeper than 30 m and when pile plugging shapes may differ between driving and static conditions.




a) Blow #1 b) Blow #10

**Figure 4. Eller Drive – Signals on RSTK Pier 8R Pile 44**

Even if we assume the same plugging condition, unloading Case method (RSU) or Signal Matching Analysis may not be sufficient to prove the pile capacities per certain strict specifications:

i) Some specifications stipulate that the required pile resistances remain during the dynamic load test event for at least 5 blows. However, impact driving may quickly destroy most of the side resistance setup (freeze) gain and plugging associated with H-pile resistances.

ii) Unloading may be too severe and the resulting lumped Case damping $J_C$ or $J_{CU}$ from Signal Matching Analysis on Blow #1 can be as low as 0.1. Extremely low $J_C$ or $J_{CU}$ values generally trigger a skepticism to the dynamic load test result.

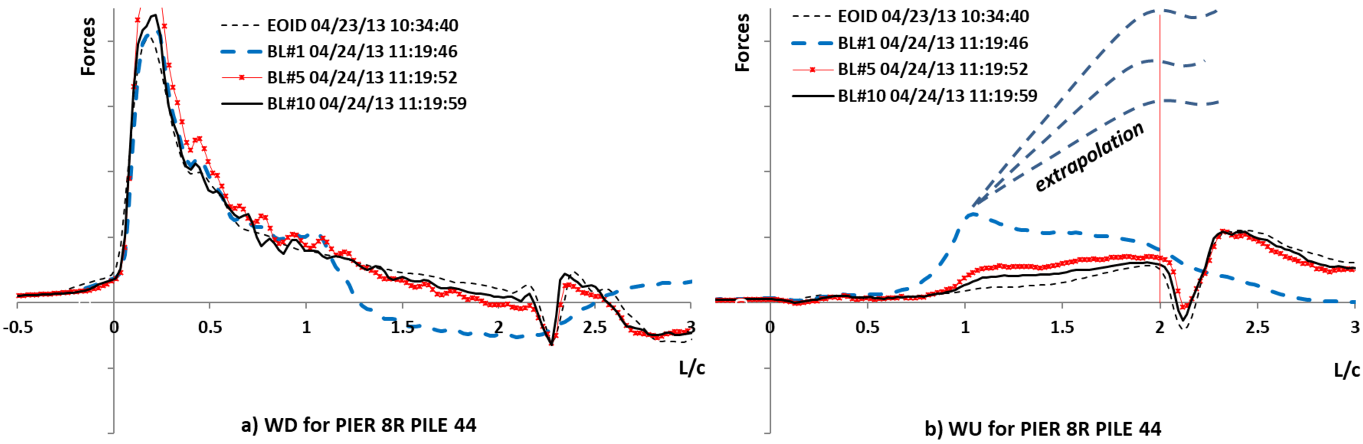



**Figure 5. Eller Drive – Signals for Pier 8R Pile 44**

# 5 SLT VERSUS DLT

## 5.1 Time-dependence of Bearing Capacity of Piles

Bearing capacity of piles change over time with gains (setup or sometimes referred to as freeze) or losses (relaxation). Therefore, to have a valid comparison between Static Load Test (SLT) and DLT results, both should be performed within a relatively short time frame. If the SLT and DLT were performed at significant different times, the authors employed the Skov and Denver, 1988 extrapolation for time-dependent **side resistances** $R_U$ of piles in soils with potential setup as follow:

$$R_U / R_{RSTR-1} = A \log(t/t_1) + 1 \qquad [1]$$

While the setup is typically not applicable for cohesionless soils (sand or sandy limestone), many researchers found that sand does have setup gain. For piles in sand per McVay et al., 1999 or per Kuo et al., 2007: A = 0.2 (minimum) using $t_1$ = 1 day.

Additionally, Svinkin and Skov, 2000 proposed a very similar equation:

$$R_U / R_{EOID} = B \log(t/t_0) + 1 \qquad [2]$$

For H-piles in clay and glacial material:

Svinkin and Skov, 2000: B=1.14 from a single case study using $t_0$ = 0.1 day.

Kam et al., 2011: $B = a / N_a^b$

$t_0$ = 1 minute = 0.000693 day.

a = empirical scale factor

b = empirical concave factor

$N_a$ = weighted average SPT N-value

For piles in sand: B = 0 per Svinkin and Skov, 2000 and Kam et al., 2011.

## 5.2 SLT/ DLT bias factor

The FHWA database contains a total of 23 H-piles having both SLT and DLT results. The DLT results in the FHWA database are basic Signal Matching Analysis results as shown in Table 1 as well as toe/shaft quakes and toe/shaft Case damping factors. The lumped Case damping is not available in the database. One pile (ID 451) has two static load test records: one pull out test and one compression test. Among these 23 SLTs, 21 of them reached Davisson (1973) failure criteria. For two SLTs that had maximum loads beneath the Davisson failure criteria, static load test segmental analyses (SLTSA, McVay et al., 2016) were performed to extrapolate the Davisson capacities. This is an iterative process to match pile's top load versus displacement during the measured static load test. The analysis employs Vijayvergiya (1977) nonlinear normalized load transfer side and tip resistance functions:

$$f_s = f_u * (z / z_{cr})^{1/3} \qquad [3]$$

where $f_s$ – mobilized unit resistance

$f_u$ – ultimate unit resistance

z – segment displacement

$z_{cr}$ – limiting segment displacement (often termed as quake); For end bearing, $z_{cr}$ = 5 to 10%B; For side resistance, $z_{cr}$ = 2.5 to 10 mm, with the ultimate skin and tip resistances iterated to match the recorded pile deformations.

Table 1 shows 8 short piles with penetration depths of approximately 15 m or less, 7 of them all have bias factors (λ = Measured Davisson Capacity/ DLT Predicted Capacity) less than 1.0 with an average of 0.75 (over-prediction of the DLT method, compared to SLT). The lone pile having bias factor more than 1.0 in this short pile group is the pullout test, where bias factor λ = Measured Pullout Capacity / Signal Matching Predicted Side Resistance. For the short piles, there is no unloading effect, no difference in plugging condition. As the piles are too short, side resistance is small, thus the piles are not able to plug in both static and dynamic events. The author has examined the short piles and found that the Signal Match Analyses can be improved, and the results would be equivalent to the static load tests. In those analyses, the lump CASE damping ($J_C$) is typically from

2.0 to 10.0 (whereas typical $J_C$ reported in literature is 0.9 or less).

There are 10 long piles with penetration depths exceeding 23 m. The bias factors are higher than 1.0 for each long pile evaluated. The average bias factor is 1.48, which indicates serious under-prediction of the DLT method (Table 1) for long piles.

**Table 1. FHWA Long H-pile Database with both SLT and DLT Results**

| # | L | | LP | | Shape | | State | Hammer | Type | Ram | Pile Driving Record | SLT Davisson $R_U$ | | DLT | | Bias |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | ft | m | ft | m | | | | | | kg | | kips | kN | kips | kN | |
| 842-1 | 94.7 | 28.9 | 75.8 | 23.1 | 14X73 | 360X109 | VT | MKT DA-35B | OED | 1 400 | M | 330 | 1 468 | 197 | 876 | 1.68 |
| 805 | 85.0 | 25.9 | 78.0 | 23.8 | 14X73 | 360X109 | SC | Vulcan 512 | ECH | 5 440 | E | 316 | 1 406 | 215 | 956 | 1.47 |
| 842-2 | 95.0 | 29.0 | 90.4 | 27.6 | 14X73 | 360X109 | VT | MKT DA-35B | OED | 1 400 | M | 388 | 1 726 | 179 | 796 | 2.17 |
| 804 | 90.0 | 27.4 | 90.7 | 27.6 | 14X73 | 360X109 | SC | Vulcan 520 | ECH | 9 070 | E | 570 | 2 535 | 566 | 2 518 | 1.01 |
| 605 | 100.0 | 30.5 | 96.2 | 29.3 | 14X73 | 360X109 | MN | ICE 90-S | OED | 4 080 | E then P | 770 | 3 425 | 652 | 2 900 | 1.18 |
| 788-1 | 120.0 | 36.6 | 103.2 | 31.5 | 14X89 | 360X132 | OH | Vulcan 512 | ECH | 5 440 | E then P | 590 | 2 624 | 569 | 2 531 | 1.04 |
| 788-3 | 120.4 | 36.7 | 105.0 | 32.0 | 12X53 | 310X79 | OH | Vulcan 506 | ECH | 2 950 | E-M | 313 | 1 392 | 308 | 1 370 | 1.02 |
| 451 | 155.0 | 47.2 | 116.5 | 35.5 | 14X117 | 360X174 | LA | Delmag D30 | OED | 2 990 | E-M | 690 | 3 069 | 295 | 1 312 | 2.34 |
| 351 | 119.5 | 36.4 | 118.3 | 36.1 | 14X89 | 360X132 | IA | Kobelko K-25 | OED | 2 500 | E-M then P | 993 | 4 417 | 731 | 3 252 | 1.36 |
| 772 | 150.3 | 45.8 | 135.4 | 41.3 | 14X117 | 360x174 | ME | Kobelko K-45 | OED | 4 500 | E then P | 1 635 | 7 273 | 1 104 | 4 911 | 1.48 |
| 798 | 34 | 10.4 | 28.3 | 8.6 | 12X74 | 310X110 | PA | LB 520 | CED | 2310 | E then P | 305 | 1357 | 405 | 1802 | 0.75 |
| 798 | 35 | 10.7 | 31.5 | 9.6 | 10X57 | 250X85 | PA | LB 520 | CED | 2310 | E then P | 335 | 1490 | 446 | 1984 | 0.75 |
| 798 | 50 | 15.2 | 33.6 | 10.2 | 12X74 | 310X110 | PA | LB 520 | CED | 2310 | E then P | 244 | 1085 | 455 | 2024 | 0.54 |
| 798 | 36 | 11.0 | 34.6 | 10.5 | 10X57 | 250X85 | PA | LB 520 | CED | 2310 | E then P | 305 | 1357 | 428 | 1904 | 0.71 |
| 798 | 50 | 15.2 | 35.6 | 10.9 | 12X74 | 310X110 | PA | ICE 640 | CED | 2720 | E then P | 485 | 2157 | 561 | 2495 | 0.86 |
| 798 | 50 | 15.2 | 35.7 | 10.9 | 10X57 | 250X85 | PA | ICE 640 | CED | 2720 | E then P | 372 | 1655 | 524 | 2331 | 0.71 |
| 609 | 40 | 12.2 | 36 | 11.0 | 14X73 | 360X109 | MS | Delmag D19 | OED | 1860 | M | 500 | 2224 | 524 | 2331 | 0.95 |
| 777 | 40 | 12.2 | 36 | 11.0 | 14X73 | 360X109 | NM | KOBE K-25 | OED | 2500 | NA | 183 | 814 | 154 | 685 | 1.19 |

Notes: L = Total Pile Length at time of driving (At time of SLT, piles were typically cut off above ground); LP = Embedded Pile Length; BORi = Begin of Restrike # i.

Detail pile driving records are in the database. The summary here only indicates a snapshot of the pile driving records: M = Medium driving (10 to 60 bpf); E = Easy driving (less than 36 bpf); P = Practical refusal (more than 200 blows per 2.5 cm).

Finally, a residential project in Maitland (Florida) in granular soil condition with soil profile summarized in Table 2 is presented. The design engineers expected the 12x53 H-piles to reach capacity at a depth of 32 m or less, with a Nominal Bearing Resistance of 2220 kN. In late 2012, a group of 3 DLT test piles failed to achieve 2220 kN at EOID: The RMX values for TP3 (32.6 m embedment), TP4 (38.1 m embedment), and TP5 (41.8 m embedment) reached 1350, 1160, and 1330 kN at EOID, respectively. Restrikes were performed on these 3 piles for 10 blows each. Typically, one blow would indicate a capacity increase, i.e., RMX=1910 kN as shown in Figure 6.b at 3-day restrike. The average value for subsequent blows was about the same as the EOID resistance of 1350 kN.

**Table 2. TP3 Soil Profile**

| Elev (m) | | Elev (ft) | | SPT N | $N_{60}$ | Soil |
|---|---|---|---|---|---|---|
| 21.5 | 19.8 | 70.5 | 65 | 12.0 | 14.9 | Sand |
| 19.8 | 18.4 | 65 | 60.5 | 6.4 | 7.9 | Sand |
| 18.4 | 15.4 | 60.5 | 50.5 | 2.7 | 1.0 | Peat |
| 15.4 | 11.1 | 50.5 | 36.5 | 4.3 | 5.4 | Sand |
| 11.1 | 9.8 | 36.5 | 32 | 14.0 | 17.4 | Sand |
| 9.8 | 3.7 | 32 | 12 | 27.0 | 33.5 | Sand |
| 3.7 | 0.6 | 12 | 2 | 71.0 | 88.0 | Sand |
| 0.6 | -0.9 | 2 | -3 | 10.0 | 6.0 | Organic Silt |
| -0.9 | -2.4 | -3 | -8 | 30.0 | 37.2 | Sand |
| -2.4 | -4.0 | -8 | -13 | 13.0 | 16.1 | Sand |
| -4.0 | -7.0 | -13 | -23 | 23.5 | 29.1 | Sand |
| -7.0 | -11.3 | -23 | -37 | 37.0 | 45.9 | Sand |
| -11.3 | -13.7 | -37 | -45 | 38.0 | 47.1 | Sand |

*Notes: Resistances in organic soils are still counted by taking half of the SPT N values, then correlated into side resistance in the SLTSA. Pile Tip was at elevation -11.3 m.*

Due to the DLT results, a static load test was performed on TP3 (the shallowest embedment pile among the three test piles). SLT result indicate that the ultimate pile capacity is higher than 2220 kN, which is the maximum applied load of (Figure 7).

On the restrike DLT blow (permanent set = 6.35 mm), the consultant's Signal Matching report gave a match quality of 1.89 with a pile toe movement of **1.05 cm**. The SLTSA performed by the authors indicated that the half plug shape would best simulate the measured static load test results. DLT and SLT results are summarized in Table 3 and Figure 7 where Davisson capacity is expected to exceed 2560 kN and the bias factor $\lambda$ is estimated to be 1.77.

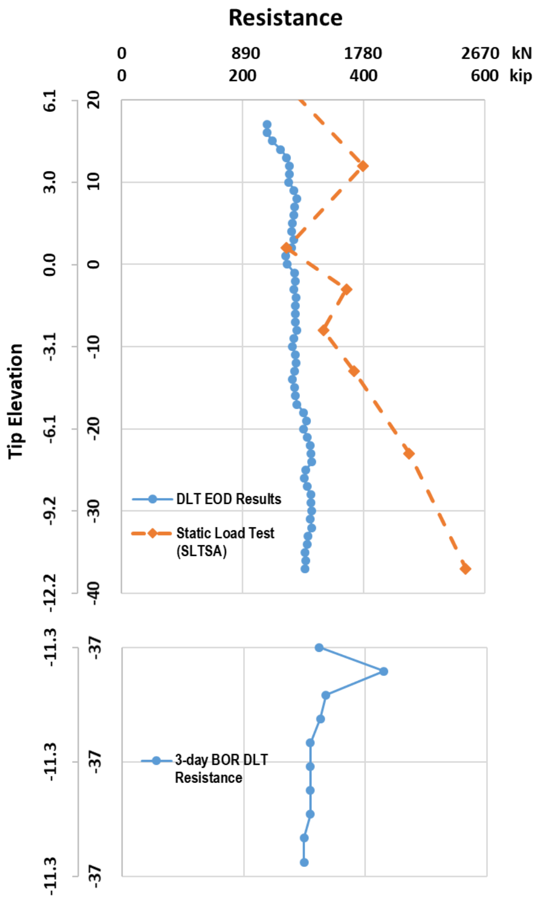



**Figure 6. TP3 DLT Results and SLTSA Estimates**

As seen in the FHWA and Florida sources, the DLT and Signal Matching results are significantly lower than the static load test resistances for long H piles. The lower DLT end bearing can be attributed to a smaller toe area in the DLT, even at BOR (at SLT condition, the toe area is plugged, while at dynamic condition, it is not). The lower DLT side resistance can be attributed to one or both of the following factors:

i) Side resistance setup (freeze) is quickly lost upon 1 or 2 blows.
ii) Side resistance unloading reduces the apparent capacities. Unloading method may not be able to add back the full unloading losses if item i) above already happens.

**Table 3. TP3 Summary Results**

| | DLT | | SLT |
|---|---|---|---|
| | EOD | BOR | |
| Skin (kN) | | 1180 | 1 890 |
| Tip (kN) | | 270 | 670 |
| Total (kN) | 1350 | 1450 | 2560 (1) |
| Bias Factor | 1.90 | 1.77 | |

(1) This total resistance is the last dot on the SLTSA curve in Figure 7. It is simulated based on the same magnitude of toe displacement of **1.05 cm**. This 2560 kN has not yet reached the Davisson offset line.

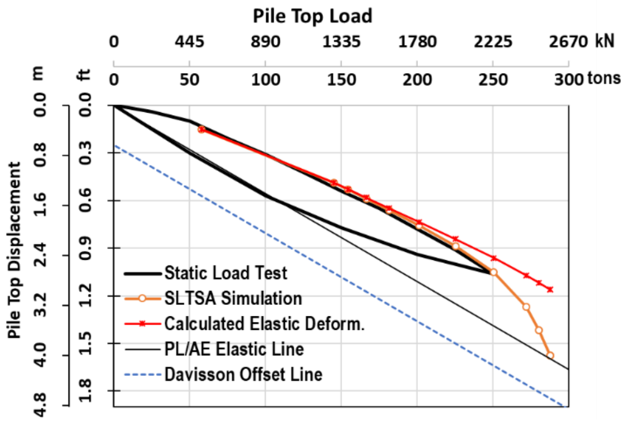



**Figure 7. TP3 Static Load Test Results**

## 6 CONCLUSIONS

This paper evaluates the DLT data and results from three data sources and suggests that DLT method is not a suitable tool to quantify the static pile capacities in certain H-pile configurations:

a) Under prediction (too conservative) ($\lambda$ = 1.48) for long slender H-piles, even with the lump CASE damping ($J_C$) of 0.1 to 0.2.

Due to large side resistance, long H-piles typically behave as plugged during static condition. However, during the DLT event, long H-piles may not behave as plugged due to high pile acceleration and high soil inertia forces. A low to moderate energy blow in the beginning of restrike may indicate that the pile is plugged with higher resistance. However, if the energy is too low then the hammer blow may not mobilize much resistance, while if the energy is too high – especially if the hammer is too large, the plug effect maybe eroded right away, even on blow #1.

Due to its ability to cut through dense or hard subsurface layers, H-piles tend to penetrate deeper than displacement piles, and unloading side resistance will often occur. Unloading in the deep embedment case makes the apparent total resistance on the DLT results much lower than the actual capacities. The addition of side resistance in the lower depths (as the pile penetrates to deeper depth) is being cancelled by the unloading side resistance in the upper portion of the pile. Therefore, if the pile is being driven much deeper, the RMX method EOID capacity does not appear to increase as much with depth until a very competent rock layer is encountered which provides a higher end bearing value.

b) For short H-piles, the side resistance is low. Thus, there is no unloading effect nor difference in plugging condition. However, if DLT users still rely on the old fashion lump CASE damping $J_C$ of 0.4 to

0.9, then the method will yield over prediction ($\lambda$ = 0.75). For short H-piles, $J_C$ can be as much as 2.0 to 10.0 for the DLT to be equivalent to the SLT.

## NOTATION

| | |
|---|---|
| c | Stress Wave Speed |
| $J_C$ | Lumped Case Damping for RMX method |
| NBR | Nominal Bearing Resistance (Capacity) |
| Setcheck | A restrike test, but typically with short wait time of less than 24-hr |
| SLT | Static Load Test |
| SLTSA | Segmental Analysis based on Static Load Test |
| $\lambda$ | Bias Factor (SLT Capacity / Predicted Capacity) |

## REFERENCES


Bridge Software Institute - BSI (2015). *FB-Deep Help Manual.* Gainesville, FL: University of Florida.

Coyle, H.M. and Ungaro, R. (1991), "Improved Design Procedures for Vertically Loaded H-piles in Sand", in *Journal of Geotechnical Engineering.* ASCE. Pp. 507-526.

Davisson, M. T. (1973). "High Capacity Piles," in *Innovations in Foundation Construction,* Soil Mechanics Division, Illinois Chapter of the ASCE. Chicago, IL: Illinois Institute of Technology. Pp. 81-112.

Florida Department of Transportation (FDOT, 2021), *Standard Specifications for Road and Bridge Construction.* Tallahassee, FL.

Goble, G.G., Rausche, F., and Likins, G., (1980). "The analysis of pile driving – A state-of-the-art", in *Proceeding of the 1st International Conference on Application of Stresswave Theory to Piles*. Balkema, Stockholm, Sweden.

Grävare, M., Hermansson, I., Bjerendal, T., Grävare, C.J. (2014), "Experiences of High Strain Dynamics Pile Testing in Accordance with EC7 In Sweden", in *Proceedings of the DFI/ EFFC 2014 International Conference on Piling and Deep Foundations*: Stockholm, Sweden.

Hannigan, P.J., Goble, G.G., Likins, G.E. and Rausche, F. (2006), *Design and Construction of Driven Pile Foundations*, Publication No. FHWA NHI-05-042.

Holloway, D.M. and Beddard, D.L. (1995), "Dynamic Testing Results Indicator Pile Test Program - I-880," in *Proceedings of the 20th Annual Members Conference of the Deep Foundations Institute*. Charleston, SC.

Hussein, M.H., Rausche, F. (1991), "Bearing Capacity of Deep Foundations from Dynamic Measurements and Static Tests - Ten Correlation Cases", in *Piletalk International 1991*: K. Lumpur, Malaysia. Pp. 49-53.

Hussein, M., Goble, G. (2004), "A Brief History of the Application of STRESS-WAVE Theory to Piles", in *Current Practices and Future Trends in Deep Foundations, Geotechnical Special Publication No. 125*, American Society of Civil Engineers: Reston, VA. Pp. 186-201.

Hussein, M. H., Choate, L., Mangogna, R., and Gray, K. (2009). "Effective Use of Steel Piles for a Large Transportation Project in Central Florida," in *Geotechnical Special Publication No. 185.* American Society of Civil Engineers: Orlando, FL. Pp. 191-198.

Kalavar, S. and Ealy, C. (2000). "FHWA Deep Foundation Load Test Database," in *New Technological and Design Developments in Deep Foundations*. Pp. 192-206.

Kam W. N., Suleiman M.T., Roling M., AbdelSalam S.S., and Sritharan S. (2011), "*Development of LRFD Procedures for Bridge Pile Foundations in Iowa*". IHRB Project TR-583 Final Report.

Kuhns, G., Hussein, M.H., Gray, K. (2003), "Deep Foundations in the Challenging Geology of Central Florida", in *Proceedings of the 28th Annual Conference on Deep Foundations*: Miami Beach, FL; Pp. 33-43.

Kuo, C., Cao, G., Guisinger, A.L., and Passe, P. (2007), "A case history of pile freeze effects in dense Florida sands", in *Proceedings of the Transportation Research Board (TRB) Annual Meeting*. Washington, DC.

Likins, G.E., Rausche, F. (2004), "Correlation of CAPWAP with Static Load Tests", in *Proceedings of the Seventh International Conference on the Application of Stresswave Theory to Piles 2004*: Petaling Jaya, Selangor, Malaysia. Pp. 153-165.

McVay, M.C., Schmertmann, J., and Bullock, P. (1999), "*Pile friction freeze: a field and laboratory study*." Florida Department of Transportation 99700-7584-119 Final Report.

McVay, M.C., Wasman, S., Crawford, S., and Huang, L. (2016), "*LRFD Resistance Factors for Auger Cast In-Place (ACIP) Piles*", Florida Department of Transportation BDV31-977-12 Final Report.

Skov, R. and Denver, H. (1988), "Time-dependence of bearing capacity of piles," in *Proceedings of the Third International Conference on the Application of Stress-Wave Theory of Piles*. Vancouver, BC. Pp. 879-888.

Svinkin, M. R., and Skov, R. (2000), "Set-up effect of cohesive soils in pile capacity," in *Proceeding of the Sixth International Conference on the Application of Stress Wave Theory to Piles.* Sao Paulo, Brazil. Pp. 107-111.

Tomlinson, M.J., (1994), "Pile Design and Construction Practice - Fourth Edition", E&FN Spon, London 357-372.

Vijayvergiya, V. N. (1977). "Load movement characteristics of piles," in *Ports 1977 Conference Proceedings Vol II*, American Society of Civil Engineers: Long Beach, CA. Pp. 269-286.