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PILEDRIVER

"A Driven Pile… Is a Tested Pile!"

PDCA 18th Annual International Conference & Expo 2014

- Desert Deep Foundations
- Indemnity clauses: minimize your risk

The PDPI Demonstrations

Much to learn from the results

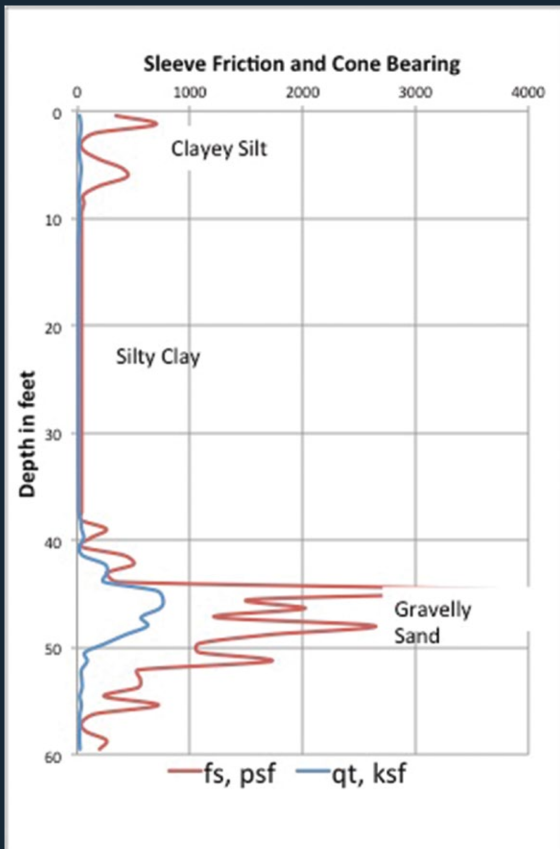
By Frank Rausche, Pat Hannigan, Van Komurka and Joe Caliendo

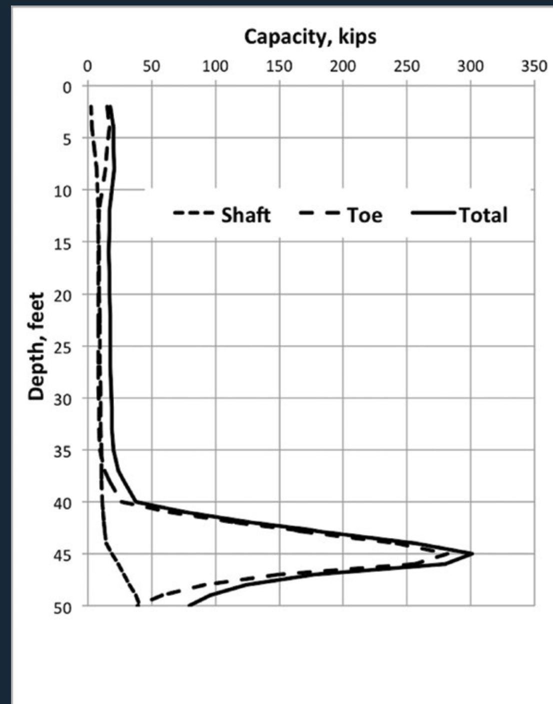
We reported in the Quarter 1 2010 issue of *PileDriver* on "Pile Driving Demonstrations – Then and Now," going back as far as 1972 to describe some of the challenges that organizers of pile driving demonstrations face. Unexpected weather conditions, unknown soil conditions, non-cooperating driving or testing equipment are all reasons for organizers being very cautious. However, the pile driving and pile testing demonstrations conducted as a part of PDCA's Professors' Driven Pile Institute (PDPI) every two years in Logan, Utah have been flawless for the seventh consecutive time, the most recent being this past summer. Since 2002, approximately 175 university professors have enjoyed not only the beautiful campus and surroundings of Utah State University, including its famous Aggie ice cream, but also the hands-on field demonstrations of pile driving and pile testing. These demonstrations provide some rather unique opportunities to observe the behavior of a few piles driven within a limited area to the same depth, and repeatedly tested both dynamically and statically. The authors believe that such unusual results should be of interest to, and shared with, the profession. As mentioned earlier in *PileDriver*, many people and organizations contributed to this data collection, and thanks are again in order to the suppliers of materials and in particular the pile driving equipment and operation.

During the PDPI, after three and a half days in the classroom, where static and dynamic analysis and introductions to field testing were presented, the field demonstrations always included: driving a 50-foot pile, restrike testing of an existing pile, static axial compression and static lateral load testing. The attendees also learned how to perform standard in-situ (CPT and SPT with energy monitoring) and surface wave soil testing. In addition, after two pre-stressed concrete segmental piles were installed in 2009, low-strain integrity testing also could be demonstrated. The variety of test demonstrations obviously yielded many different and interesting results. However, for the sake of clarity and brevity, this article will only deal with axial analysis and testing issues, both static and dynamic. While not complete (some driving records and static load test results are not available anymore), the available data provides ample opportunity for studying various methods of drivability prediction, capacity determination and pile behavior over long time periods. Comparisons between measured and predicted blow counts and capacity values will be demonstrated.

Test details

The demonstration site is located in a large mountain valley in Northern Utah, surrounded by 10,000-foot mountains, and it is not surprising that the site is under some artesian pressure. In fact, carelessly drilling too deep may cause some flooding of the site if the borehole is not quickly plugged. The soft predominantly silty clay soil profile at the site consists of lakebed sediments from ancient Lake Bonneville. Soil properties were identified by a 1998 CPT test (Figure 1) showing roughly 10 feet of very stiff silty clay over about 30 feet of soft varved silty clays. A layer of dense gravelly sand with a thickness between five and 10 feet was then encountered at a depth of approximately 43 feet. Below the dense gravelly sand followed layers of silt and clay. The gravelly sand layer was an ideal bearing layer for load test demonstration purposes and care was always taken not to punch through this layer or most of the pile capacity would be lost.





Left: Figure 1: Soil information. Above: Figure 2: Pile capacity based on CPT-Schmertmann

Piles tested statically were always vertically driven 12.75-inch-O.D. closed-end pipe piles of either one quarter or 0.375-inch wall thickness. They were roughly 50 feet long. The pile installed in 2003 was repeatedly restrike tested and statically loaded.

Static geotechnical analysis

Using the Schmertmann Method incorporated in GRLWEAP, shaft, toe and total pile capacity vs. depth were calculated (Figure 2). Other interpretation methods are available and would probably lead to different assessments of the pile bearing capacity. The Schmertmann method indicates a peak capacity of 300 kips at a depth of 45 feet. Typically though, piles were driven to depths between 42 and 44 feet where capacities between 120 and 260 kips would be expected. Obviously, small changes in pile toe depth would produce significantly differing toe and total capacities. Like all static analysis methods, the resulting capacity values are thought to occur long-term. Disturbances caused by pile driving would be expected to reduce the shaft resistance, while toe resistance is generally assumed to be unaffected. However, with the pile toe in the bearing layer, about 90 percent of the long-term resistance would be expected at the pile toe so that shaft resistance losses during driving would not be significant.

Different hydraulic and diesel hammers were used at the test site in different years. The first tests in 2002 were done with an IHC S90 hydraulic hammer (ram weight Wr=9.9 kips, maximum rated energy Er=65.9 ft-kips), a very powerful hammer for the required driving. However, it was run at an equivalent stroke of two feet to provide enough hammer blows for a meaningful demonstration. Frequently used was a Kobelco K13 open-end diesel hammer (Wr=2.9 kips; Er=25.4 ft-kips) that was run at a relatively low efficiency thereby providing the most hammer blows. Most recently, during the 2013 demonstration, a pile was installed and another restrike tested with an ICE® Pilemer IP3 hydraulic hammer, also referred to as a DKH-3U, (Wr=6.6 kips, Er=26.4 ft-kips). This hammer was run at a two-foot stroke for the driving of a pile, and at maximum stroke (about four feet) for the restrike test.

Pile driving demonstration in 2013

The 2013 test procedures were very similar to those of previous demonstrations. On the morning of Friday, June 28, a restrike test was first performed on the test pile driven during the 2003 PDPI. This same pile (12.75-inch O.D. × 0.375-inch wall, 48.4-foot long, 42.5-foot penetration depth) also had been statically tested in the afternoon of the previous day (see below). With a stroke of four feet, the IP3 transferred at first rather low energies (less than 10 foot-kips or 38 percent of rated), but improved to about 18 foot-kips (68 percent of rated); it appeared that the variability of the hammer output was primarily caused by hammer misalignment. This is frequently a problem with restrike tests; during extended driving sequences, hammer-pile alignment is generally more easily maintained. In general, this problem makes restrike blow counts rather unreliable and that affects then, of course, the reliability of bearing capacity by dynamic formulas or wave equation analyses. In the present case, the first four blows advanced the pile only a quarter inch, while the next three blows produced a set of three quarters of an inch (seven blows for the first inch). At the end of the 20-blow restrike, the blow count was two blows/inch, and the total movement of the pile during the restrike was four and a quarter inches.

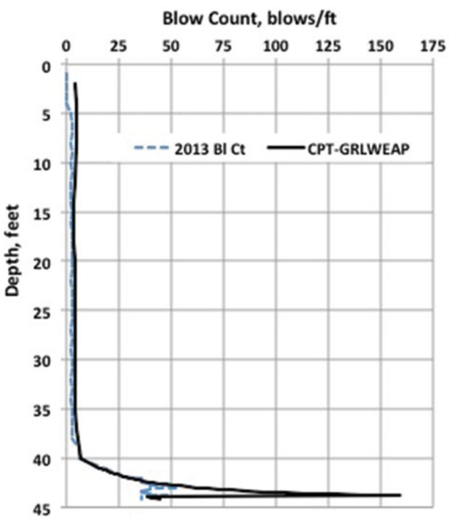


Figure 3: Driving resistance – measured and predicted

Plate 1: The instrumented 2003 pile under the IP3 hydraulic hammer

Next, a new pile (12.75-inch O.D. x 3/8-inch wall, 50 feet long) was driven by the IP3 hammer with a two-foot stroke to a depth of 41.5 feet. During a half-hour driving interruption, sensors for a Pile Driving Analyzer® (PDA), as well as for other equipment, were installed (Plate 1). When resuming driving, the blow count, which had been four blows for six inches (eight bl/ft) before the temporary stop, was now 15 blows for six inches (30 bl/ft). The pile was then driven to a penetration depth of 43.9 feet, with a final equivalent blow count of 48 bl/ft. To protect them, the sensors of the PDA, which had now reached the pile gate at the bottom of the leads, were removed during a short interruption. Then 12 more blows were applied to the pile with a four-foot stroke, yielding a four-inch penetration for an equivalent blow count of 36 bl/ft.

Drivability is normally checked by means of a wave equation analysis based on a calculated unit shaft resistance and end bearing distribution. CPT-based soil information is often considered the most reliable way of predicting pile capacity and thus blow counts. However, when we used the above described hammer-pile-soil system using the GRLWEAP program, then for a depth of 43.8 feet, the predicted blowcounts were much larger than observed (159 vs. 48 bl/ft for the two-foot stroke). Surprisingly, for the four-foot stroke, the agreement was much better (44 vs. 36 bl/ft, equivalent). Detailed comparisons are shown in Table 1 and Figure 3.

Given such differences in blow count, the energy provided by the hammer might be questioned. But Table 1 shows that the

Depth	Stroke	CPT Capacity	GW* Predicted Blow CT	Actual Clow CT	Transferred Energy GW	Transferred Energy PDA	Pile Top Stress GW	Pile Top Stress PDA
ft	ft	kips	Bl/ft	Bl/ft	ft-kips	ft-kips	ksi	ksi
43.8	2	231	159	48	10.1	10.8	23.4	23
44.2	4	253	44	36	20.5	17-19**	33.3	26-28**

Table 1: Summary of drivability results – 2013 pile. * GW – GRLWEAP ** Based on end of restrike measurements

PDA-measured transferred energy values for the two-foot stroke were very close to the GRLWEAP-calculated values (10.8 vs. 10.1 ft-kips). Surprisingly though, for the four-foot stroke operation, the energies were not quite as expected, and these differences were even greater for the compressive pile top stresses. So while the hammer performance for the continuous driving sequence was well predicted, for the four-foot stroke the hammer output may have been somewhat low. One therefore has to conclude that the resistance during driving was lower than predicted from CPT records. While we normally expect that shaft resistance is lower during driving than long-term predicted by the CPT or other static method, in the present case, shaft resistance was insignificant which means the toe capacity was lower than expected during driving. The reason could be high pore water pressures, either pile driving-induced or artesian.

Pile capacity results

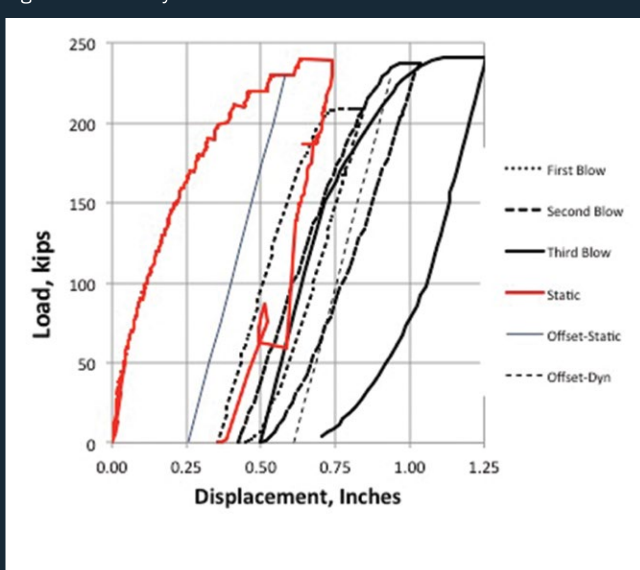
Bearing capacity was calculated by

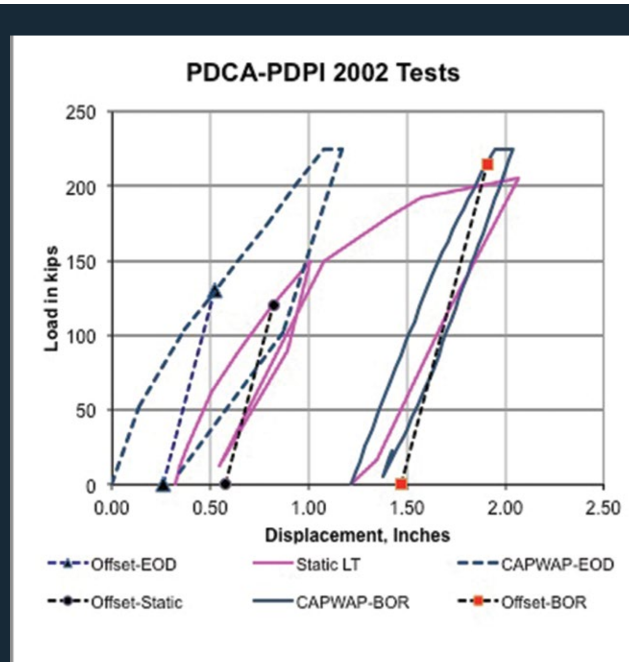
- The Schmertmann CPT Method
- GRLWEAP wave equation based on blow count and hammer stroke
- Dynamic formulas (Modified Gates and Engineering News)
- CAPWAP® signal matching (performed following the test)
- iCAP® real-time signal matching (normally performed during the test)

Complete records are not available for all demonstration tests conducted through the years. For selected tests from 2002, 2009 and 2013, representing all three hammer types employed and for which we had sufficient data available, we calculated the results and listed them in Table 2. Pile capacity values calculated by the Schmertmann method are also shown Table 2. The highest CPT-calculated capacity of almost 300 kips was expected for a depth of 45 feet, which was not reached by the various demonstration piles since there was concern that the piles would break through the bearing layer and lose most of their resistance, and that the piles would potentially penetrate the confined artesian aquifer below. (Note that for an actual project, it may not be advisable to have pile groups terminate in this relatively thin layer which is underlain by soft materials).

The load test performed on June 27 on the 2003-installed test pile resulted in a maximum load of 240 kips, and an Offset Criterion (also called Davisson Criterion) capacity of 230 kips. For the first three blows of the restrike test (June 28), evaluated and plotted by the signal matching method CAPWAP, the corresponding values were 209, 237 and 241 kips. Figure 4 shows the load-set curves from both the static load test (which preceded the dynamic test) and the three dynamic restrike test analyses (the displacements

Below: Figure 4: Load-set curves from restrike and static load tests, performed on June 27 and 28, 2013 on 2003 driven pile. Right: Figure 5: 2002 dynamic and static load set curves





Test Year Test Type Pile Year	Equiv. Blow Count	Static Offset / Max*	Hammer	Stroke	CAPWAP Offset / Max	iCap	Wave Eq. GRLWEAP	Mod. Gates	EN-Ru
	Bl/ft	kips		ft	Kips	kips	kips	kips	Kips
2002 EOD – 2002	40	N/A	IHC S90	2	125/225	N/A	160	276	596
2002 BOR – 2002	60	120/225	IHC S90	2	200/225	N/A	252	N/A	N/A
2009 EOD – 2009	50	N/A	K-13	4.5	153	N/A	160	222	456
2013 EOD – 2013	48	N/A	IP3	2	163	150	170	222	453
2013 EOD – 2013	36	N/A	IP3	4	N/A	N/A	240	320	731
2013 BOR – 2003	84***	230/240	IP3	4	237/241	223**	342	N/A	N/A
2013 EOD – 2003	24	N/A	IP3	4	205	205	183	270	528

Table 2: Ultimate capacities from static and dynamic tests, and from Modified Gates and EN formulas. *Offset Criterion/Maximum Load; **Average over first 6 blows; ***1st inch of restrike

are cumulative in this presentation). The automatic signal matching method iCAP, which is operator independent, was also performed on this restrike test and yielded a capacity prediction of 229 kips when averaging the first six restirke blows.

It is also of interest to review the results obtained during earlier tests. For example, compared to other test results available, the static test of 2002 indicated relatively large settlements (Figure 5). This pile had been installed the day prior to the static test and, therefore, had only a 28-hour setup time. According to the offset criterion, it failed at a low 120 kips, while supporting a maximum load of 225 kips. Similarly, the EOD dynamic test reached the offset criterion at 125 kips while indicating a maximum load support of 225 kips (Figure 4). Probably due to the consolidation caused by the static test and the additional waiting time, the restrike indicated a much stiffer behavior and thus an offset capacity much closer to the maximum load.

Pile capacity was also evaluated by blow-count-based methods such as dynamic formulas and wave equation analyses (Table 2). We selected the FHWA-endorsed Modified Gates Formula and the surprisingly still popular Engineering News (EN) Formula. For uniformity, ultimate capacities were calculated with the EN formula by removing the theoretical safety factor of six, yielding ultimate rather than “safe load” values. Note that early restrike blow counts are not applicable to dynamic formulas,

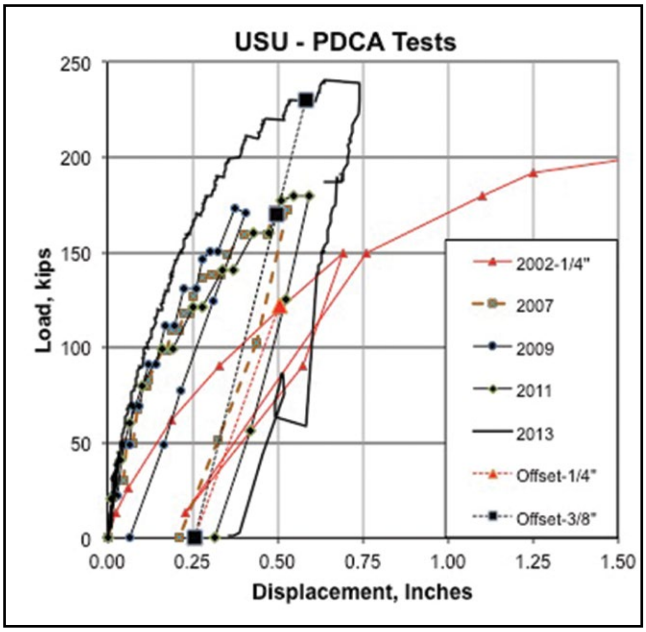


Figure 6: Several static load test results and offset lines

because their results already include an estimate of soil set-up. Only if it can be assured that at the end of restrike soil set-up effects were not present can the end of restrike (EOR) blow counts be used for formula evaluation. In Table 2, this was done for the 2013 restrike test of the 2003 pile. The Modified Gates results varied between 222 and 320 kips, while the EN formula suggested pile capacity values of between 453 and 751 kips. Actually, even the wave equation analysis cannot be trusted when used with restrike blow counts. Table 2 reports that wave equation analysis of the 2013 restrike of the 2003 pile indicates 342 kips capacity, much higher than the static test value. The reason is the low transferred energy during the early restrike mentioned above. In general, it has been observed that the variability of both transferred energies and blow counts during an early restrike cause unreliable wave equation predictions. This is the case not only for diesel hammers but, like in the present case, also for hydraulic hammers.

Of the seven static load tests performed at seven PDPI demonstrations, five load-displacement curves were available and are presented here (Figure 6) to give an impression of the variablity of pile capacity at the same site and for similar piles. Corresponding offset lines were also included; they indicate the following capacity values: 2002: 120 kips; 2007 and 2011 about 170 kips; 2009: did not fail according to the offset criterion; 2013: 230 kips. Again, the 2002 test stands out as one with considerably larger displacements than the other tests, and the 2013 load set curve reaches much higher capacities than the others, although the 2009 test could have reached a relativley high capacity, but it was stopped early. As mentioned, the larger-than-usual settlements in 2002 may have been due to a shorter set-up time period. The high 2013 capacity was explained by Utah State University professor emeritus Loren Anderson as a reduced artesian pressure since the previous year (2012) was much drier than other years. So it is also possible that the 2002 large settlements and thus low offset capacities were caused by unusually high rainfalls. Imagine if this were not recognized for

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an actual construction project: actual ultimate pile capacities could be just one half of what was determined during a static test conducted during a dry time period!

Conclusions

The PDPI test demonstrations produced a large number of results. Only a few of those results could be presented here. However, we believe that the results presented are not only interesting, but they can add to our general knowledge about pile behavior in sand under artesian pressures. This article was written to summarize pile driving and axial test results. No effort has been made to present a thorough analysis. Notwithstanding these disclaimers, the test results described in this article support a number of conclusions.

- Do not expect that all static tests at a given site indicate the same long-term capacity (even for piles of the same section, driven to the same depth and terminal blow count), or that long-term capacity is a constant
- Never expect that piles driven in sand, even predominately end-bearing piles, do not exhibit soil set-up
- Never expect that two piles driven to the same depth exhibit the same capacities.
- If a formula produces a good result in one situation, do not expect that it will work equally well under all circumstances, even at the same site
- Based on restrike tests, even the wave equation analysis does not always yield reliable results because of the uncertainty of hammer performance
- Static and dynamic loading tests produced comparable results in this case, and dynamic tests helped explain what is happening ▼