HIGH-CAPACITY DRIVEN-PILE FOUNDATION FOR A 33-STORY HIGH-RISE IN MILWAUKEE, WISCONSIN, USA

Van E. Komurka, P.E., D.GE, F.ASCE, GRL Engineers, Inc., Cleveland, OH, USA, (216) 831-6131, vkomurka@grlengineers.com
William H. Walton, P.E., S.E., F.ASCE, GEI Consultants, Inc., Vernon Hills, IL, USA, (847) 984-3494, bwalton@geiconsultants.com
Ryan C. Rusk, P.E., M.ASCE, GEI Consultants, Inc., Vernon Hills, IL, USA, (847) 984-3494, rrusk@geiconsultants.com
Patrick J. Hannigan, P.E., M.ASCE, GRL Engineers, Inc., Cleveland, OH, USA, (216) 831-6131, phannigan@grlengineers.com

Keywords: depth-variable criteria, driven piles, dynamic testing, instrumentation, load testing, restrike testing, set-up, static testing

ABSTRACT

The project consists of a 33-story building with two levels of below-grade parking across the entire building footprint. Timely and cooperative teamwork among the owner/developer, geotechnical engineer, testing agency, structural engineer, construction manager, and pile-driving contractor resulted in an efficient, cost-effective, high-capacity driven-pile foundation on a constrained urban site with significantly variable subsurface conditions. Design objectives included utilizing the highest allowable pile loads reasonably installed from a drivability perspective, using readily available equipment. A pre-production test program was performed on 16-inch-diameter (406-mm-diameter) steel pipe piles, driven closed-ended and subsequently filled with concrete. The test program included dynamic monitoring of 10 piles during installation, and of select piles during long-term restrike testing with a drop hammer, with one instrumented static load test. CAPWAP® analyses were performed on both end-of-initial-drive and beginning-of-restrike dynamic test records. The site exhibited significant soil set-up, even after relatively short wait times. Test-program objectives included characterizing set-up profiles. A design set-up profile was used to develop depth-variable driving criteria for both 500- and 600-kip (2,224- and 2,669-kN) allowable load piles. Test program results permitted more-accurate reduced capacities to be assigned to damaged piles, and to piles which experienced practical refusal. These more-accurate assigned reduced capacities decreased the number of replacement piles required by 50 percent, saving both time and cost.

PROJECT DESCRIPTION

The project, located in Milwaukee, Wisconsin, USA consists of a 33-story mixed-use residential building with two levels of below-grade parking across the entire site footprint. Column service loads range from 1,400 to 4,200 kips (6,228 to 18,683 kN). The project is located immediately adjacent to an existing two-story masonry retail building with a partial basement, supported on shallow spread-footing foundations.

Subsurface Conditions – The generalized soil conditions are presented in Fig. 1. Site grades were relatively level, with a ground-surface elevation of approximately +53 feet (16 m), Milwaukee City Datum ("MCD"). Fill material from previous construction was encountered to approximate Elev. +33 feet (10 m). Below the fill, medium dense silty sand, to silt with sand, was encountered to approximate Elev. +23 feet (7 m). Underlying deposits consisted of lean clay, with undrained shear strengths ranging from approximately 1,000 to 3,000 pounds per square foot (psf) (48 to 144 kPa). Silty sand, to silt with sand, exhibiting increasing relative density with depth, was found below the lean clay to approximate Elev. -92 feet (-28 m), where hard lean clay (glacial till) with undrained shear strengths in excess of 5,000 psf (239 kPa) was encountered. Borings did not extend below Elev. -103 feet (-31 m).

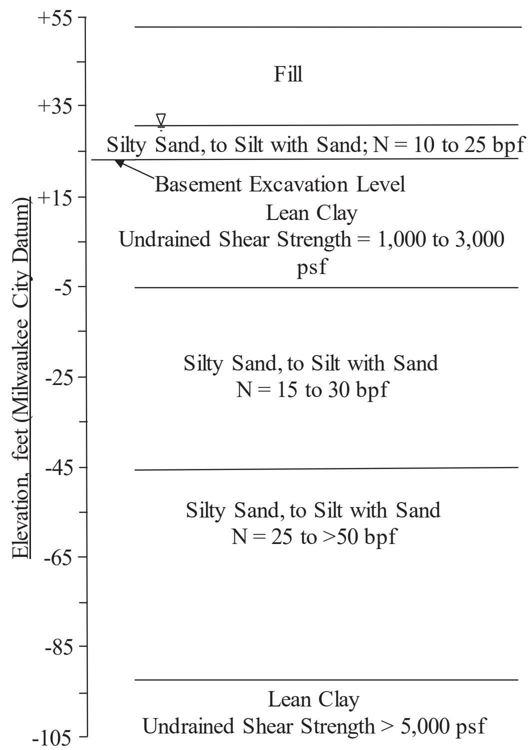

Fig. 1. Generalized soil profile

Foundation Selection – The project owner had previously completed foundation work for a similar building a short distance away that was founded on a mat, which had encountered significant difficulties during subgrade preparation. As a result of this experience, the design team was tasked with identifying alternate foundation systems.

In the Milwaukee area, closed-end pipe piles with diameters ranging from 10.75 to 16 inches (273 to 406 mm) are readily available, and multiple contractors are experienced with their installation. Drilled shafts and augered cast-in-place piles were evaluated, but were considered to have higher installation risks due to the many sand and silt layers throughout the soil profile. This, in combination with area contractors being less-experienced with installing these types of deep foundations than with installing driven piles, eliminated these options from consideration.

Based on a review of the project service loads, it was determined that the project would benefit from a program to maximize pipe-pile allowable loads while maintaining reasonable installation methods, including readily available pile sections, and hammer types and sizes. The project design team reviewed the option of 16-inch-diameter closed-end pipe piles, and estimated that piles with an allowable load on the order of 500 to 600 kips (2,224 to 2,669 kN) could reasonably be obtained utilizing a carefully planned and executed test program which included both dynamic and static load testing.

Test Program Objectives – A pre-production test program was performed. The drivability of 16-inch piles was assessed, including the ability of the closed-end section to penetrate dense upper soil layers prior to reaching desirable toe elevations (which ranged approximately from Elev. -40 to -74 feet (-12 to -23 m)), using readily available equipment. A major objective of the test program was to establish the highest long-term axial compression capacity to which the 16-inch piles could reasonably be installed, from which the maximum allowable load that could be used for design would be established. To evaluate tension resistance, it was desired to determine long-term capacity resistance distribution (relative shaft and toe capacity contribution, and capacity distribution along the shaft). Toe resistance relaxation of piles terminating in dense fine-grained granular deposits was also evaluated.

To aid in demonstrating high capacities, soil set-up (time-dependent capacity increase) was characterized. Set-up characterization included determining both total set-up magnitude and distribution along the shaft, allowing set-up to be incorporated into production-pile driving criteria. Test pile locations were selected to provide good general site coverage, to evidence variability in subsurface conditions, helping to estimate production-pile lengths. An axial compression static load test was performed, the purpose of which was to provide a full-scale proof-test demonstrating ultimate pile capacity, including load-transfer behavior. This information was then compared to the static load test pile's dynamic monitoring results and analysis, and used to correlate the dynamic model on which final production pile driving penetration resistance criteria were based.

TEST PROGRAM SCOPE

Installation

Test pile locations are presented in Fig. 2. Because site constraints precluded pile driving in the southern portion of the site, pre-production testing was performed on the northern half of the site. Test pile locations on the northern half of the site were selected based on providing general site coverage, accessibility, impact to existing site operations, reuse of test piles as production piles (to save cost), building loads distribution, and proximity to boring locations. All tested piles were steel pipe, having a nominal outside diameter of 16 inches (406 mm) and a nominal wall thickness of 0.50 inches (13 mm), and filled after driving with concrete having a minimum 28-day strength of 6 ksi (41 MPa). The piles were fabricated to ASTM A252 Grade 3 requirements, which specify a minimum yield strength of 45 ksi (310 MPa). The piles were fitted and driven with a 1-inch-thick (25 mm), 16.5-inch-diameter (419 mm), oversized toe plate.

A total of 10 test piles were installed: 5 indicator piles, 1 static load test pile, and 4 reaction piles. Each pile installation consisted of a bottom section with a flat toe plate, and varying numbers of upper sections, which were spliced to the bottom section (and sometimes to each other) during a pause in driving using full-penetration butt welds against backup rings. The piles were driven using a Delmag D46-32 single-acting diesel hammer. According to the manufacture's literature, the Delmag D46-32 has a 10.14-kip (45.10-kN) ram, a rated maximum stroke of 12.05 feet (3.673 m), and a maximum rated energy of 122.2 foot-kips (165.7 kJ). The hammer was equipped with a four-step fuel pump, allowing it to be operated over a range of rated energies. Fuel setting selection during installation was intended to result in terminal equivalent blow counts in the range of 30 to 120 blows per foot ("bpf"; blows per foot are roughly equivalent to blows per 0.3 meter).

Dynamic testing using a Pile Driving Analyzer® ("PDA") was performed on each test pile for its full depth of driving to monitor hammer and driving-system performance, calculate pile driving stresses, assess pile structural integrity, and evaluate pile bearing capacity. To satisfy the test program objective of establishing the highest long-term capacity to which production piles could reasonably be installed, test pile driving tended to approach the upper limits of reasonable blow counts and driving stresses. Test-pile driving termination was a field decision which considered subsurface conditions apparent from borings, anticipated shaft set-up, desired ultimate capacity, blow count, potential to mobilize capacity during restrike testing, driving stresses, pile purpose, pile material available, and dynamic monitoring results available in the field during initial driving. A CAPWAP analysis was subsequently performed on a representative end-of-initial-drive ("EOID") dynamic test record from each test pile to further evaluate ultimate pile capacity, including the relative soil resistance distribution along the pile shaft, and at the pile toe. Details of the test pile installations are summarized in Table 1.

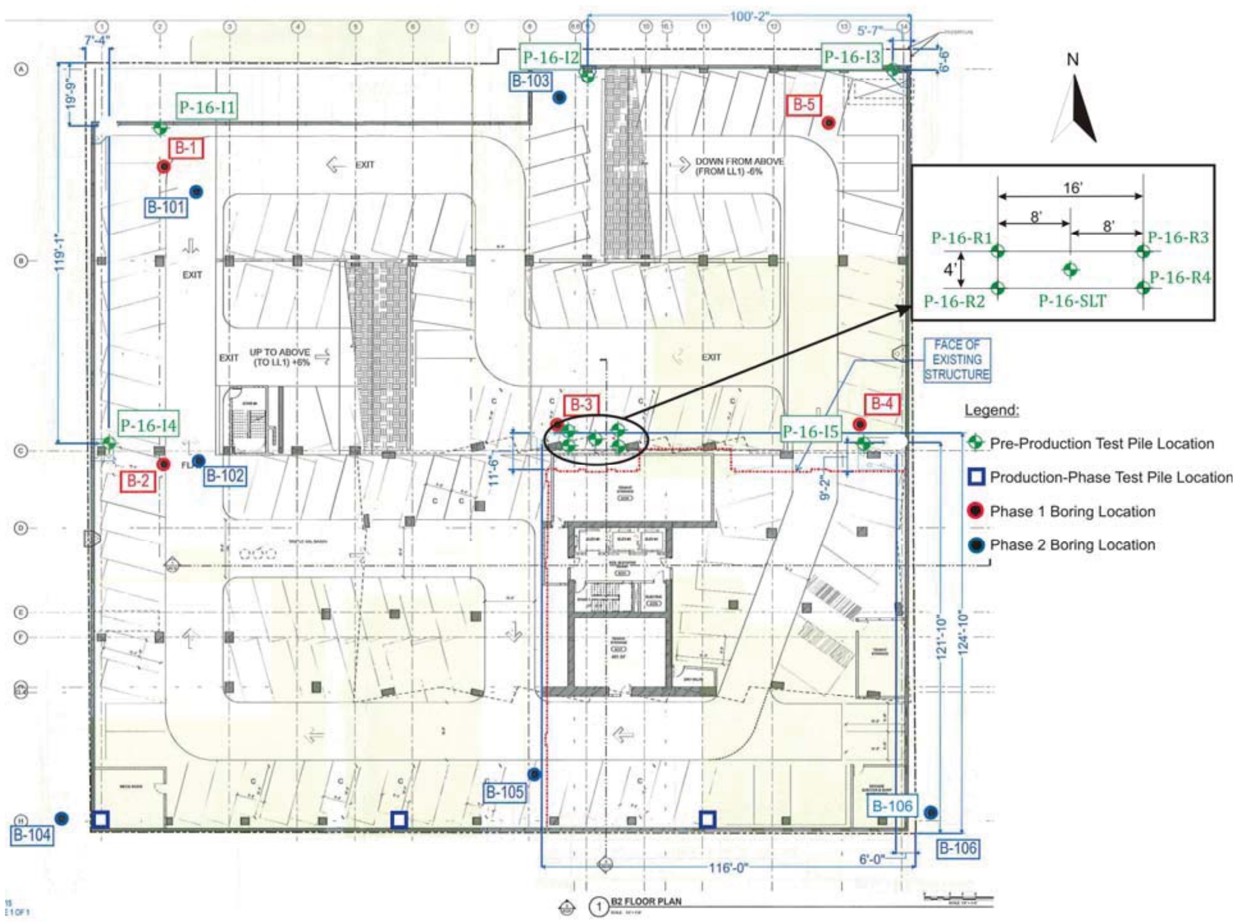

Fig. 2. Test Pile Location Diagram

Axial Compression Static Load Test

The static load test's purpose was to provide a full-scale proof-test demonstrating ultimate pile capacity, including load-transfer behavior. This information was then compared to the static load test pile's dynamic monitoring results and analysis, and used to refine the dynamic model on which production pile driving penetration resistance criteria were based.

Prior to filling the static load test pile with concrete, vibrating-wire strain gages ("VWSGs") were installed at various elevations within the pile. The VWSGs were located in the pile by attaching rebar strain meters ("sister bars") to a 1-inch-diameter (25-mm-diameter) steel water pipe which was installed in the pile with centering devices attached to it. The VWSG elevations were selected to have the uppermost gage coincident with the ground surface, and gages below coincident with major stratigraphy changes indicated by the most-proximate boring. The purpose of the internal strain gages was to determine the load in the pile section at discrete elevations, and by doing so, allow for load-transfer determinations.

Four surface-mounted VWSGs were attached to the static load test pile's exterior, equidistant around its perimeter, just above ground surface. These strain gages served two purposes: 1) to aid in assessing composite-section action, and 2) to aid in determining the composite-section modulus of elasticity (required to determine both the pile's theoretical elastic shortening, and the embedded strain gages' data reduction and load-transfer determinations).

Table 1. Test Pile Summary

Pile	Driving Status	Hammer	Toe Elevation, feet (MCD)	Penetration Resistance: Reported	Penetration Resistance: Equivalent bpf	Elapsed Time Since EOID, days	Average Hammer Stroke, feet[1]	Average Transferred Energy, ft-kips	Average Maximum Pile Head Compression Stress, ksi	Maximum Pile Compression Stress, ksi[2]	Average Case Method Capacity (RX9), kips	Mobilized CAPWAP Capacity, kips: Shaft	Toe	Total
P-16-I1	EOID	D46-32	-43.4	22 / 3 in.	88	---	10.0	83	37.2	37.4	873	359	435	794
	BOR1	D46-32	-43.4	13 / 1 in.	156	3.913	11.0	94	43.9	46.2	1,158	834	415	1,249
	BOR2	APPLE	-43.4	1 / 0.15 in.	80	52.967	3.0	75	---	7.4	---	940	400	1,340
P-16-I2	EOID	D46-32	-70.2	38 / 3 in.	152	---	9.8	82	36.8	37.8	765	432	260	692
	BOR1	D46-32	-70.2	13 / 1 in.	156	0.808	11.8	96	42.4	43.6	965	722	255	977
	BOR2	APPLE	-70.7	1 / 0.25 in.	48	44.019	4.0	149	---	9.5	---	1,401	260	1,661
P-16-I3	EOID	D46-32	-63.3	20 / 3 in.	80	---	10.6	82	37.9	40.1	905	734	125	859
	BOR1	D46-32	-63.3	20 / 1 in.	240	1.821	12.2	100	44.1	47.0	1,112	1,040	145	1,185
	BOR2	APPLE	-63.3	1 / 0.2 in.	60	45.150	2.5	77	---	7.4	---	1,416	125	1,541
P-16-I4	EOID	D46-32	-63.0	21 / 3 in.	84	---	10.7	84	38.6	38.1	871	344	565	909
	BOR1	D46-32	-63.0	16 / 1 in.	192	1.847	11.3	96	39.6	42.7	1,038	541	495	1,036
	BOR2	APPLE	-63.0	1 / 0.05 in.	240	54.006	3.5	80	---	7.3	---	1,012	350	1,362
P-16-I5	EOID	D46-32	-52.0	25 / 3 in.	100	---	10.5	89	39.4	38.4	879	184	695	879
	BOR1	APPLE	-52.0	1 / 0.15 in.	80	36.249	3.6	94	---	7.3	---	604	700	1,304
P-16-R1	EOID	D46-32	-46.5	21 / 3 in.	84	---	10.5	82	36.0	36.3	786	202	595	797
P-16-R2	EOID	D46-32	-52.3	34 / 3 in.	136	---	10.7	82	36.8	40.0	880	208	645	853
P-16-R3	EOID	D46-32	-39.8	24 / 3 in.	96	---	10.0	82	38.1	37.1	765	107	710	817
P-16-R4	EOID	D46-32	-40.5	20 / 3 in.	80	---	10.5	90	37.6	37.8	781	106	708	814
	BOR1	APPLE	-40.5	1 / 0.2 in.	60	38.865	3.5	112	---	7.5	---	624	655	1,279
P-16-SLT	EOID	D46-32	-49.0	5 / 3 in.	20	---	9.1	65	33.6	32.6	456	178	295	473
	SLT	---	-49.0	---	---		---	---	---	---	---	977	368	1,345
	BOR1	APPLE	-49.0	1 / 0.05 in.	240	37.118	3.0	62	---	6.4	---	1,096	290	1,386

1. With the exception of the APPLE restrike on P-16-I5 which used a 40-kip (178-kN) ram, all the APPLE restrikes used a 48-kip (214-kN) ram.
2. For restrikes performed with the APPLE, the maximum pile compression stress acts on the composite (steel and concrete) section.

The static load test pile was loaded in axial compression by jacking against an overhead steel frame assembly. Reaction to overhead frame uplift was provided by four reaction piles. The assembly was designed by the pile-driving contractor to provide a maximum reaction of 1,530 kips (6,806 kN).

Restrike Testing

<u>Short-Term</u> – Displacement piles (such as closed-end pipe used for this project) terminating on dense fine-grained granular deposits (fine sands, and silts) can exhibit decreased toe capacity after driving, referred to as relaxation [Morgano and White 2004]. Capacity loss due to relaxation is unconservative with respect to long-term compression capacity, allowable load, safety factor, and settlement. Since the conditions encountered in the borings indicated that piles for this project might terminate on dense fine-grained deposits, the test program evaluated potential relaxation by performing short-term restrike testing on 4 unfilled test piles. Restrike testing consisted of redriving the pile a limited number of blows with the Delmag D46-32 hammer on Fuel Setting 4. Dynamic monitoring was performed during short-term restrike testing, and a CAPWAP analysis was performed on a representative beginning-of-restrike ("BOR") dynamic test record for each test pile. The elapsed times after EOID at which the short-term restrikes were performed are reported in Table 1.

<u>Long-Term</u> – It is recognized that piles which have been in place for some time typically develop greater capacity than their EOID capacity [Komurka et al. 2003a; 2003b]. This capacity increase with time

phenomenon is referred to as soil set-up, which is defined as the difference between the capacity at the time initial driving terminates and the capacity at some later time. Set-up can refer to total pile (both shaft and toe) capacity increase, shaft capacity increase, or toe capacity increase. Generally, set-up is considered to be predominately related to shaft capacity increase. Unit shaft set-up determination is typically appropriate for discrete pile sections, such as the sections between strain-gage elevations for static load test data, or some fraction of the total pile length for CAPWAP analyses. Unit shaft set-up for a particular pile segment is defined as the shaft resistance increase on the segment, divided by the pile segment surface area. Accounting for both end-of-drive capacity and set-up results in more-accurate capacity estimates, and therefore more-economical pile installations [Komurka 2004; Komurka and Arndorfer 2009].

To aid in determining shaft set-up magnitude and distribution, 8 test piles received long-term restrike testing (2 reaction piles were inaccessible for restrike testing). The elapsed times after EOID at which the long-term restrikes were performed ranged from 36.25 to 54.01 days, and are reported in Table 1. Generally, since set-up has occurred by the time the piles are restruck, a hammer capable of delivering more impact force than used for initial driving may be required to mobilize soil resistance [Likins and Rausche 2008]. Concreting closed-end pipe piles increases their impedance, which increases the applied impact force and also aids in mobilizing their capacity during restrike testing [Likins 2017].

Long-term restrikes were performed on the concrete-filled test piles using an APPLE drop-weight system. The APPLE system consisted of a support/guide frame, and a 40- to 48-kip (178- to 214-kN) ram. Ram weights and drop heights, and transferred energies, are reported in Table 1. A subsequent CAPWAP analysis was performed on a representative BOR dynamic test record for each restruck test pile.

TEST PROGRAM RESULTS

Driving Behavior

Blow Count – The test piles' equivalent blow count (measurements of blows per driving distance converted to units of bpf) profiles are presented in Fig. 3. A review of Fig. 3 indicates that none of the test piles experienced significant equivalent blow counts above approximate Elev. -30 feet (-9.1 m), final toe elevations ranged from -39.8 to -70.2 feet (-12.1 to -21.4 m), and final equivalent blow counts ranged from 20 to 152 bpf. P-16-SLT's final toe elevation of –48.0 feet (14.6 m) (and its associated relatively low terminal equivalent blow count of 20 bpf) was limited by the length of available pile material. Final penetration depths and blow counts are summarized in Table 1.

Case Method Capacities – The test piles were installed using a variable-stroke (and therefore variable-energy) hammer, and the piles' equivalent blow counts presented in Fig. 3 do not account for variations in stroke and transferred energy. For this reason, the equivalent blow counts of individual piles are not directly comparable. A more-direct comparison of driving behavior can be made using initial-drive Case Method geotechnical compression capacities estimated by the PDA based on dynamic monitoring results [Rausche et al. 1985]. PDA-estimated capacities for each blow were averaged for each foot of penetration. The test piles' Case Method estimated geotechnical compression capacity profiles as determined from PDA monitoring data are presented in Fig. 4. A review of Fig. 4 indicates that Case method EOID capacities ranged from 456 to 905 kips (2,028 to 4,026 kN). EOID Case method capacities are summarized in Table 1.

EOID CAPWAP Capacities – EOID CAPWAP-estimated capacities are reported in Table 1. A review of Table 1 indicates that EOID CAPWAP-estimated total capacities ranged from 473 to 909 kips (2,104 to 4,043 kN). CAPWAP estimation of total capacity can potentially be affected by the pile set for the CAPWAPed blow. For pile sets which are too low (blow counts that are too high), CAPWAP may underpredict capacity [Likins and Rausche 2008]. For pile sets which are too high (blow counts that are too low), CAPWAP may overpredict capacity [Rausche 1991]. For this reason, it is generally desired to have equivalent penetration resistances to between 30 and 120 bpf, for both end-of-initial-drive and

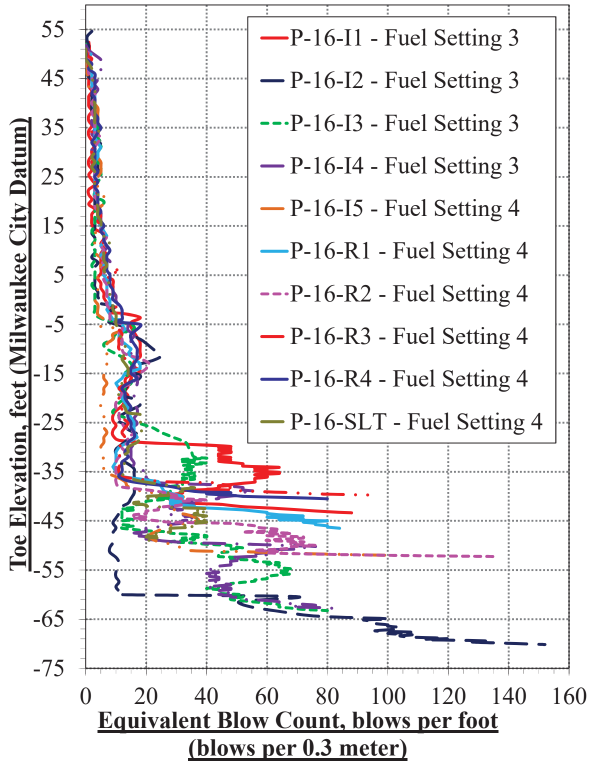

Fig. 3. Equivalent Blow Count Profiles

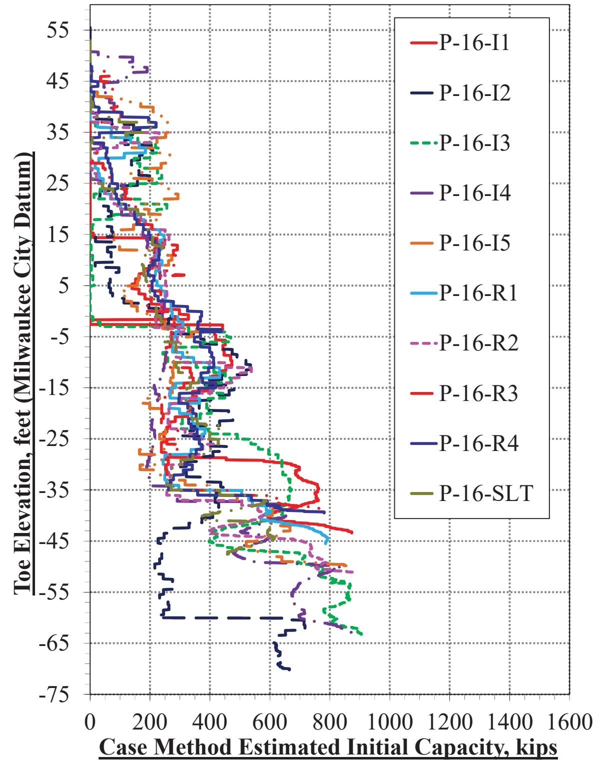

Fig. 4. Case Method Estimated Initial Capacity Profiles

restrike testing [Hannigan 2016]. To aid interpretation of CAPWAP-estimated capacity in this respect, the reported equivalent blow count or pile set associated with CAPWAPed blows is also presented in Table 1.

EOID CAPWAP Unit Shaft Resistance Distributions – The test piles' CAPWAP-determined EOID unit shaft resistance distributions are presented in Fig. 5. Occasionally, CAPWAP analyses may have difficulty distinguishing large toe resistance from large shaft resistance on the pile segment immediately above the toe. This difficulty tends to become more-pronounced at higher blow counts, and can potentially lead to over- or under-prediction of unit shaft resistance values for the pile segment(s) above the toe. This potential phenomenon associated with CAPWAP analyses can affect EOID data, as well as BOR data, and therefore can potentially affect calculated unit set-up values for the pile segment(s) above the toe. To aid interpretation of Fig. 5 in this respect, the reported equivalent blow count associated with each CAPWAP-determined unit shaft resistance distribution is also presented in Fig. 5.

Short-Term Restrikes

One of the test program objectives, and the reason short-term restrikes were performed, was to evaluate the potential for toe relaxation of piles terminating in dense, fine-grained deposits. A comparison of EOID CAPWAP toe capacities to restrike toe capacities in Table 1 indicates no significant toe relaxation. The pile which evidenced the maximum apparent toe relaxation was P-16-I4. However, the apparent relaxation can likely be attributed to both the short- and long-term restrikes' high equivalent blow counts (192 and 240 bpf, respectively) not fully mobilizing all available toe capacity, not to real toe relaxation.

Static Load Test

Capacity – During the static load test, a discrepancy between applied loads indicated by the load cell and jack pressure became apparent. Using strain-gage data, and elastic modulus values from previous similar static load tests, it was determined in the field that the jack pressure was likely the more-accurate of the two readings. Subsequent office analysis agreed with this field assessment. Accordingly, reported load test applied loads are based on jack pressure readings.

The load-movement behavior of the static load test pile is presented in Fig. 6. Several methods exist for data interpretation, and ultimate capacity definition, from load-movement results [Fellenius 1990]. For this project, the criterion selected to define capacity was that developed by Davisson (1972). Using this criterion, the static load test pile was assigned a geotechnical axial compression capacity at the time of the static load test of 1,346 kips (5,987 kN).

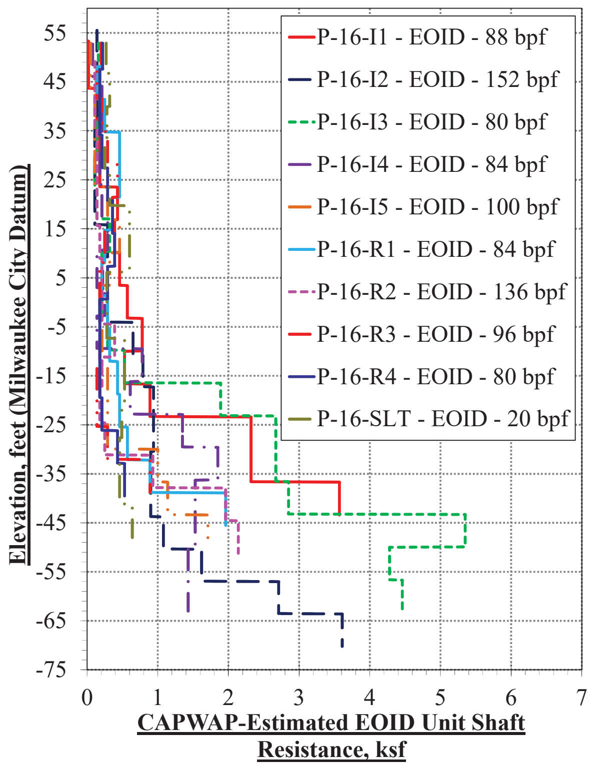

Fig. 5. CAPWAP-Estimated EOID Unit Shaft Resistance Profiles

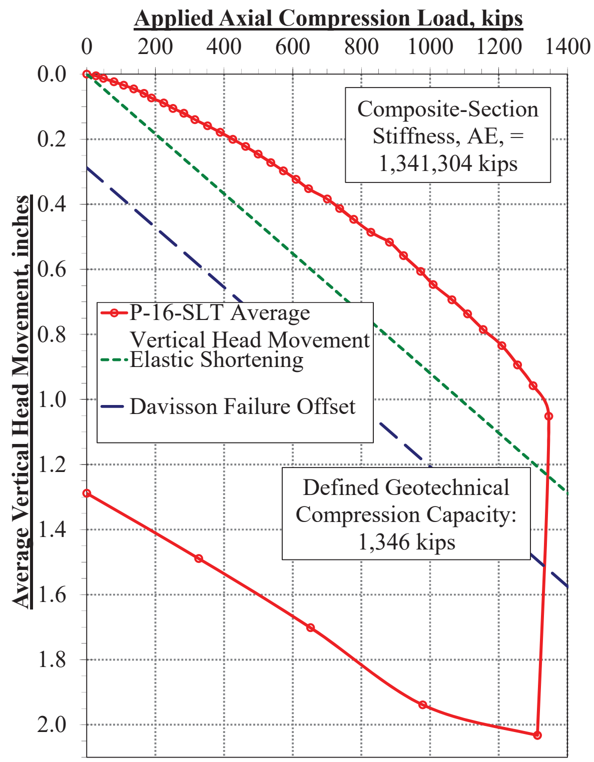

Fig. 6. Compression Static Load Test Load-Movement

Load Transfer – In our experience, concrete at different elevations within a pile may attain differing elastic moduli values during curing. Accordingly, the internal strain gage data at each strain-gage location were evaluated individually using the tangent modulus method presented by Fellenius (2001). This location-specific evaluation yielded a range of back-calculated composite-section elastic moduli, which were used to reduce the below-grade internal strain gage data and determine load-transfer behavior. The static load test pile's EOID CAPWAP analysis estimated residual loads. This estimated residual load distribution was used to correct the strain-gage-determined load transfer at the static load test's maximum applied load.

The static load test pile was restruck 0.88 days after static load test completion. A comparison between the static load test pile's mobilized shaft, toe, and total capacities determined by strain-gage data, and CAPWAP analysis of BOR dynamic results, is presented in Table 2.

Table 2. Static Load Test and Beginning-Of-Restrike CAPWAP Capacities

Capacity Component	Static Load Test Capacity, kips (kN)	BOR Dynamic Load Test Capacity, kips (kN)
Shaft	977 (4,346)[A]	1,096 (4,875)
Toe	368 (1,637)	290 (1,290)
Total	1,346 (5,987)	1,386 (6,165)

A. The static load test pile's shaft resistance was estimated by subtracting the strain-gage-determined load at the toe (incorporating residual loads) at the maximum sustained load from the maximum sustained load (1,346 kips (5,987 kN)).

Long-Term Restrikes

Rather than trying to determine only set-up magnitude, it is more-useful to characterize set-up distribution along the pile shaft. This allows for the development of depth-variable driving criteria [Komurka 2004]. EOID and long-term restrike shaft resistance profiles determined from CAPWAP were used to determine unit, and cumulative, shaft set-up profiles.

Long-Term Restrike Unit Shaft Resistance Profiles – The test piles' CAPWAP-determined long-term unit shaft resistance distributions are presented in Fig. 7. For piles P-16-I1, P-16-I2, P-16-I3, and P-16-I5, the long-term resistance over the lower portion of the pile was not fully mobilized. This resulted in long-term unit shaft resistance profiles that decrease in magnitude over the last two or three CAPWAP soil segments.

Long-Term Unit Shaft Set-Up Profiles – A pile's unit shaft set-up profile can be determined by subtracting its EOID unit shaft resistance profile from its long-term restrike unit shaft resistance profile (i.e., subtracting the profiles in Fig. 5 from the profiles in Fig. 7). This was done for each test pile on which a long-term restrike was performed. The resulting unit shaft set-up profiles are presented in Fig. 8.

Long-Term Cumulative Shaft Set-Up Profiles – Using the relationship between a piles' toe elevation and its embedded shaft area, the unit shaft set-up profiles presented in Fig. 8 can be used to calculate estimated cumulative shaft set-up profiles presented in Fig. 9. These cumulative set-up profiles present the estimated set-up each pile would have experienced had its driving been terminated at any toe elevation above its final toe elevation.

Set-Up Accuracy Scenarios – Since set-up is determined from the difference between two capacity determinations (EOID and some later time), its accuracy depends upon the accuracy of both capacity determinations. In the case of capacity estimates obtained from dynamic load testing data and subsequent CAPWAP analysis, potential over- or under-prediction of capacity must be assessed, and taken into account when evaluating apparent set-up results. For this project, such assessment was discussed previously. Potential set-up accuracy determination scenarios are presented in Table 3.

Design Cumulative Shaft Set-Up Profile – Based on a review of the test piles' cumulative shaft set-up profiles presented in Fig. 9, a lower-bound set-up profile was selected for design. As indicated in Fig. 9, the design cumulative set-up profile selected was that determined for Test Pile P-16-I4. The lower portion P-16-I5's profile was discounted for the reasons discussed previously.

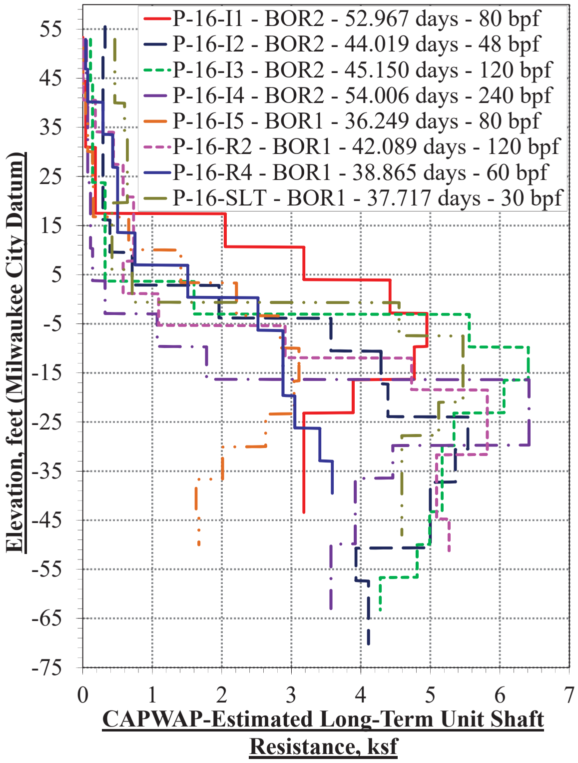

Fig. 7. **CAPWAP-Estimated Beginning of Long-Term Restrike Unit Shaft Resistance Profiles**

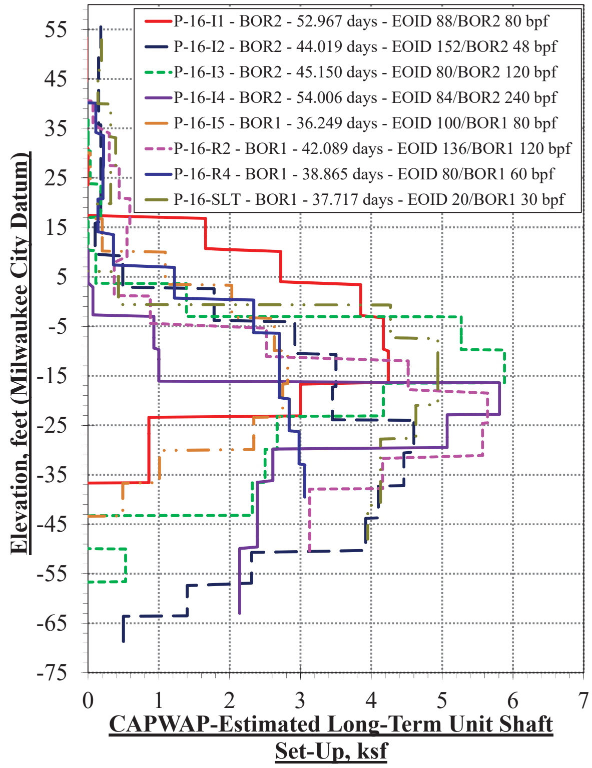

Fig. 8. CAPWAP-Estimated Long-Term Unit Shaft Set-Up Profiles

Set-Up Rate

Multiple restrikes to determine set-up time rate were not included in the test program. However, set-up which occurred during a pause in driving to splice one of the test piles provides insight into set-up time rate.

The bottom section of Test Pile P-16-I2 was driven to an embedded depth of 56.2 feet (17.1 m), when driving was paused to splice on a second section. Following splice completion, the pile was driven to an embedded depth of 115.6 feet (35.2 m) (toe Elev. -60.0 feet (-18.3 m)), when driving was paused for 97 minutes to splice on a third section. Immediately prior to pausing to splice, the equivalent blow count was 10 bpf with a hammer stroke of 7.5 feet (2.3 m), and a CAPWAP-estimated shaft resistance of 143 kips (636 kN). When driving resumed, the equivalent blow count was 70 bpf with a hammer stroke of 9.5 feet (2.9 m), and a CAPWAP-estimated shaft resistance of 535 kips (2,380 kN).

Allowable Geotechnical Load Selection

Compression – This project's foundation piles were designed using allowable stress design ("ASD"). In ASD, allowable geotechnical load is equal to geotechnical capacity divided by a safety factor. Based in part on the type, amount, and quality of the testing performed and results obtained, a safety factor of 2.0 was selected for the project piles. To select the highest practical allowable load for the project, the highest practical capacity to which piles could be installed was established first.

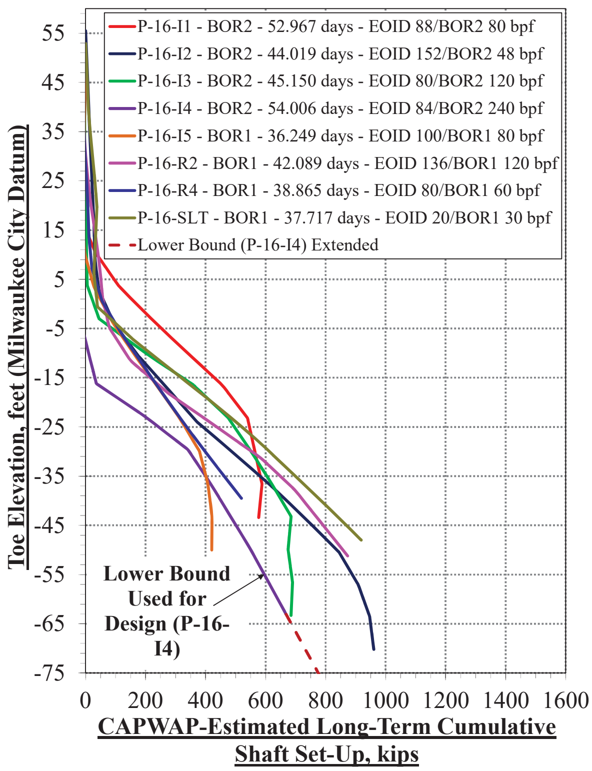

Fig. 9. CAPWAP-Estimated Long-Term Cumulative Shaft Set-Up Profiles, Including Design Profile

Table 3. Potential Set-Up Accuracy Scenarios

	Long-Term Capacity		
	Potentially Underestimated	Reasonably Estimated	Potentially Overestimated
Potentially Over-Estimated	Set-Up Potentially Underestimated	Set-Up Potentially Underestimated	Set-Up Potentially Indeterminate
Reasonably Estimated	Set-Up Potentially Underestimated	Set-Up Reasonably Estimated	Set-Up Potentially Overestimated
Potentially Under-Estimated	Set-Up Potentially Indeterminate	Set-Up Potentially Overestimated	Set-Up Potentially Overestimated

Long-term capacity profiles can be estimated for each test pile by adding each pile's Case method initial capacity, and cumulative set-up, profiles together [Komurka 2004]. A benefit of this is approach is that it provides an estimate of long-term capacity not only for a test pile's terminal toe elevation, but also for any shallower terminal toe elevation. Estimated lengths can be estimated for lesser capacities, which improves length estimates and economic evaluations.

It follows that to use test program results to select the highest practical capacity to which piles can be installed, the design cumulative set-up profile should be added to each test pile's Case method initial capacity profile. These profiles are presented in Fig. 10.

Based on the results presented in Fig. 10, 1,200 kips (5,338 kN) was considered the highest practical capacity to which production piles could be installed, and a maximum allowable load of 600 kips (2,669 kN) per pile was selected for the project. Test Pile P-16-SLT was excluded from this evaluation, because its depth (and therefore its end-of-drive and long-term capacity) was limited by the length of available pile material.

Tension – Resistance to tension loads is considered to be provided by only the pile shaft. Long-term shaft capacity profiles based on BOR CAPWAP analyses are presented in Fig. 11. When dynamic test results are used to assess tension capacity, GRL recommends that the shaft resistance be reduced to 80% of the CAPWAP-determined value to account for the difference in loading direction. GRL also recommends that safety factor of 2.5 be applied to this reduced shaft resistance value to account for analysis sensitivity in separating shaft and toe resistances. The resulting safety factor with respect to CAPWAP-estimated ultimate shaft capacity is 3.125. The profile resulting from applying a safety factor of 3.125 to a lower-bound beginning-of-restrike cumulative shaft capacity profile is also presented in Fig. 11.

A review of Fig. 11 indicates that the allowable tension resistance following the above procedure is a function of pile toe elevation. Accordingly, tension resistance requirements must be incorporated into production-pile driving criteria by requiring both a minimum embedment depth, and a minimum blow count below the minimum embedment depth. Minimum embedment depths may govern installed lengths.

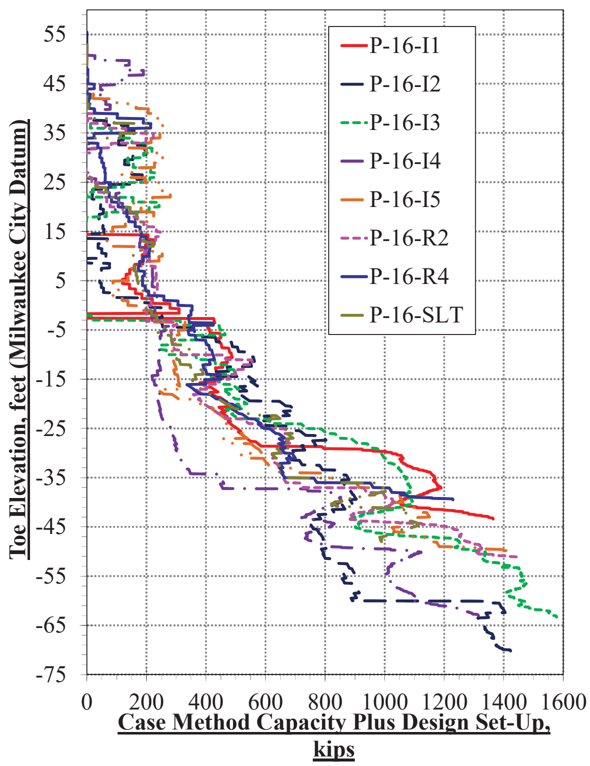

Fig. 10. Design Set-Up Profile Added to Individual Case Method Initial Capacity Profiles

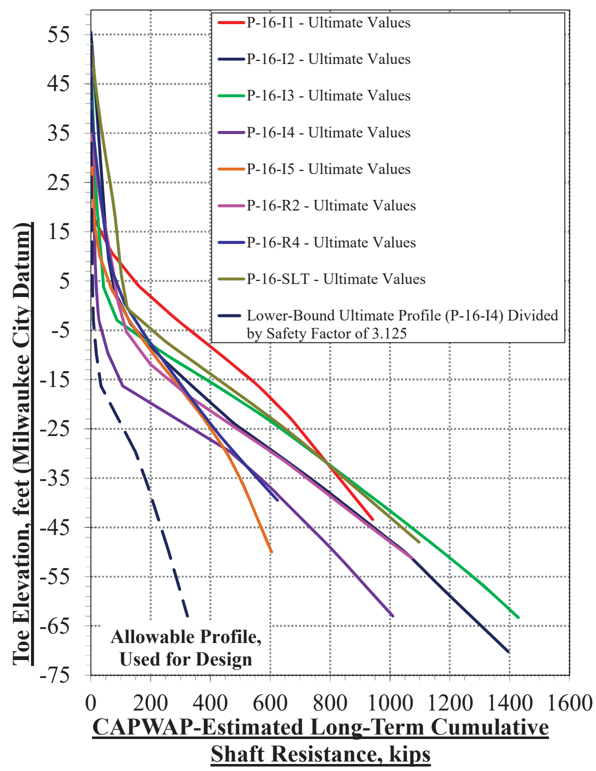

Fig. 11. CAPWAP-Estimated Beginning of Long-Term Restrike Cumulative Shaft Resistance Profiles, Including Design Profile Used for Allowable Tension Resistance

Driving Criteria

Depth-Variable Driving Criteria – Production pile driving criteria were developed using the wave-equation software GRLWEAP. A number of wave equation input parameters were refined by matching EOID dynamic results determined from CAPWAP with wave equation results [Hannigan et al. 2016]. Since set-up begins virtually immediately after driving termination, driving operations were required to be as continuous as possible; driving multiple bottoms prior to completing each pile's installation was precluded.

Long-term capacity is the sum of two components: EOID capacity, and set-up. The pile test program characterized both these components individually as functions of pile toe elevation. This provides the flexibility to provide driving criteria for driving from grades which differ from test-pile driving grades. And if desired, separate safety factors could be applied to initial capacity and to set-up when developing production–pile penetration resistance installation criteria [Komurka et.al. 2006].

Cumulative shaft set-up is a function of pile embedded shaft area. The greater a pile's embedment length, the more set-up occurs, and therefore the less EOID capacity is required. Since required initial ultimate capacity decreases with increasing embedment depth, so does required penetration resistance. Therefore, the lower-bound cumulative shaft set-up profile presented in Fig. 8 can be used to develop depth-variable production-pile driving criteria which reflect decreasing required initial capacity with increasing embedment depth. This concept is illustrated in Fig. 12. Resulting production pile installations will be driven only as deep as necessary to achieve (after set-up) their required long-term ultimate capacities, reducing unnecessary driven length.

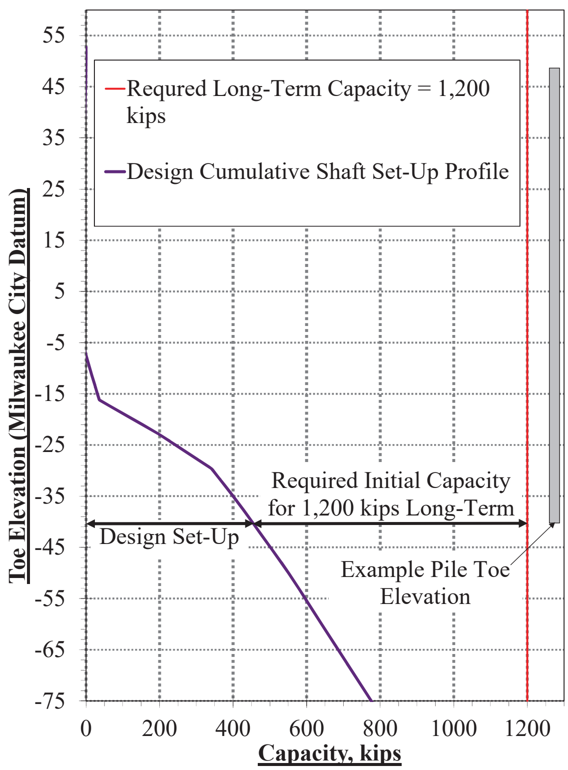

Fig. 12a. Determination of EOID Capacity Profile for 1,200-Kip Long-Term Capacity

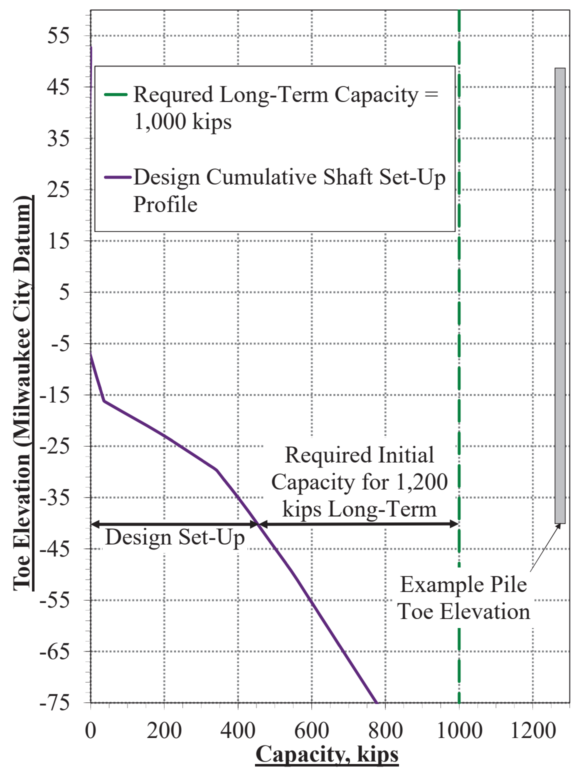

Fig. 12b. Determination of EOID Capacity Profile for 1,000-Kip Long-Term Capacity

Production-pile driving criteria are a function of the energy the hammer transfers to the pile during driving (transferred energy). Transferred energy is a function (among other things) of hammer stroke. For diesel hammers, the stroke (and hence transferred energy) is variable and indirectly only somewhat controllable. Therefore, since a variable-energy hammer was used for production-pile driving, stroke-dependent driving criteria were developed. An example of depth-variable stroke-dependent driving criteria is presented in Table 4. Table 4 presents depth-variable driving criteria with 3-foot (0.9-m) depth intervals. The depth intervals for criteria development can be selected based on the rate of set-up increase with increasing depth.

Multiple Allowable Loads – The approach presented above can be used to develop depth-variable driving criteria for multiple allowable loads using the results from a given test program. For this project, to further optimize the foundation design, the structural engineer identified a number of locations where 500-kip (2,224-kN) allowable load piles would more-efficiently support structure loads. Depth-variable driving criteria for 500-kip (2,224-kN) allowable load piles were readily developed using the approach illustrated in Fig. 12. To accomplish this, a different required EOID capacity profile was determined by subtracting the design set-up profile from a depth-constant required long-term capacity of 1,000 kips (4,448 kN).

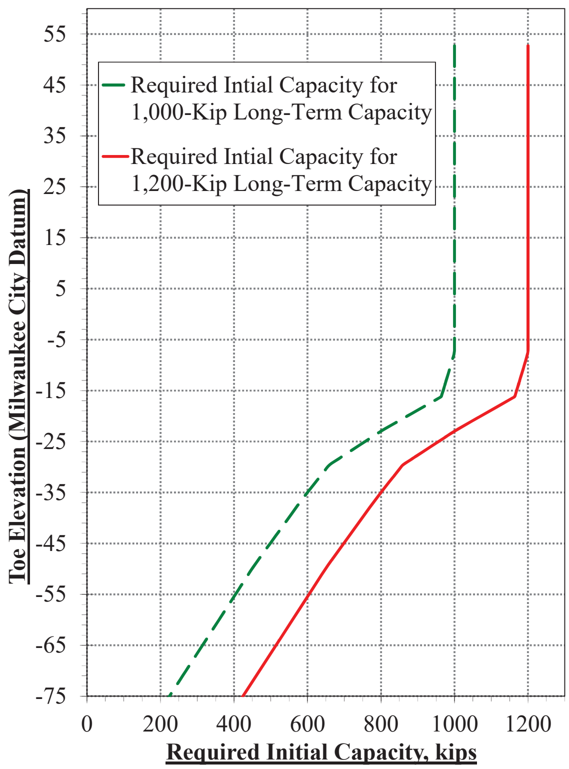

Fig. 12c. Required EOID Capacity Profiles for Multiple Long-Term Capacities

Multiple Driving Elevations – Production piles were driven from several different pile-cap subgrade elevations. Design set-up, and therefore required EOID capacity, are elevation-specific, not embedment-length-specific. Accordingly, several driving criteria tables were developed for both allowable loads. Each driving criteria table was applicable for driving from a small range of pile-cap subgrade elevations (on the order of 3 to 4 feet (0.9 to 1.2 m)).

PRODUCTION DRIVING/CONSTRUCTION MONITORING

As mentioned previously, site constraints precluded pre-production testing in the southern portion of the site. To confirm hammer performance, pile drivability, and driving behavior, full-depth dynamic monitoring was performed during the driving of three test piles on the southern portion of the site at production-driving startup.

During production-pile installation, hammer stroke and blow count were measured and documented with a Saximeter. The piles were installed to requisite blow counts based on depth-variable criteria, taking into account the hammer stroke and pile toe elevation. Internal pile inspection was performed by viewing the completed piles from the surface by dropping a weighted tape and light into the pile. Select piles were also internally observed using a downhole camera. Where pile damage was noted (i.e., partial collapse, gouging, bulging, rupture, etc.), piles were assigned reduced capacities based on shaft resistance (determined from the test program) above the damage. Other piles in the same group as the damaged pile(s) were assigned individual capacities base on depth and EOID blow count, and the need for additional piles was determined by the structural engineer.

The number of production piles damaged during driving was approximately 3 percent. Test program results were used to assist in both assigning reduced capacities to the damaged piles, and in assigning individual

Table 4. Depth-Variable Driving Criteria Table Example

Delmag D46-32 - I.D. No. 890

16"x0.50" Closed-End Pipe Piles

300-Ton Allowable Load - Driving Elevations +14.0 thru +17.9 feet

Penetration Depth, feet		Minimum Required Equivalent Blow Count, blows per foot									
		Hammer Ram Stroke, feet									
From	To	6.5	7.0	7.5	8.0	8.5	9.0	9.5	10.0	10.5	11.0
55.1	58.0							---	---	---	94
58.1	61.0						---	116	93	77	66
61.1	64.0				---	---	96	77	65	55	49
64.1	67.0		---	---	98	76	65	54	47	43	39
67.1	70.0	---	103	80	65	54	47	42	37	34	31
70.1	73.0	90	70	58	49	42	38	34	31	28	26
73.1	76.0	62	52	44	39	34	31	28	26	24	23
76.1	79.0	47	40	36	32	29	26	24	223	21	20
79.1	82.0	37	32	29	26	24	23	21	20	19	18
82.1	85.0	30	27	24	22	21	20	18	17	16	16
85.1	88.0	25	23	21	19	18	17	16	15	15	14

capacities to the undamaged piles in the group. This was done in close coordination with the structural engineer who evaluated the actual required load for each group, and for each pile within the group, where damaged piles occurred. These combined efforts reduced the number of replacement piles which would have otherwise been required by 50 percent.

Pile Support Cost – Pile support cost based on available support is defined as pile installation cost divided by allowable pile load [Komurka 2015]. For this project, dividing the total cost of all concrete-filled production piles by the sum of their allowable loads results in a pile support cost based on available support of $9.26 per available kip. Dividing the total cost of all concrete-filled production piles by the total of all the building's service loads results in a pile support cost based on utilized support of $12.45 per structure design kip.

SUMMARY AND CONCLUSIONS

The project consists of a 33-story mixed-use residential building with two levels of below-grade parking across the entire building footprint. A pre-production driven-pile test pile program included dynamic monitoring, an instrumented static load test, and long-term restrike testing on ten 16-inch-diameter closed-end pipe piles. A major objective of the test program was to establish the highest long-term axial compression capacity to which the 16-inch piles could reasonably be installed, from which the maximum allowable load that could be used for design would be established.

The site soils exhibited significant soil set-up, even after relatively short wait times, such as during a pause in driving to splice. From dynamic testing results, long-term cumulate shaft set-up profiles (set-up as a function of pile toe elevation) were developed. Estimated long-term capacity profiles which included both initial capacity and set-up were developed, from which a maximum allowable load for the project was selected. Long-term restrike cumulative shaft resistance profiles were used to determine a design tension resistance profile, from which minimum pile embedment lengths were established.

A design set-up profile was established for the project, and was used to develop depth-variable driving criteria for both 500- and 600-kip (2,224- and 2,669-kN) allowable load piles. The depth-variable criteria provided for increasing cumulative set-up with increasing embedment length, and so required decreasing terminal blow counts with deeper pile penetrations, thus saving production pile lengths and driving time. Test program results were used to assign reduce capacities to short or damaged piles, reducing the required number of replacement piles by 50 percent, saving additional cost and time.

ACKNOWLEDGEMENTS

The authors wish to acknowledge and gratefully thank members of the ownership, development, design, and construction team for their contributions to the project: Northwestern Mutual Life, Hines, Magnusson Klemencic Associates, Zenith Tech, Inc., and C.D. Smith Construction, Inc.

REFERENCES

Davisson, M.T. (1972). "High Capacity Piles," *Proceedings*, ASCE Lecture Series, Innovations in Foundation Construction, Illinois Section.

Fellenius, B.H. (2001). "From Strain Measurements to Load in an Instrumented Pile," *Geotechnical News Magazine*, Vol. 19, No. 1, pp. 35-38.

Fellenius, B.H. (1990). "Guidelines for the Interpretation and Analysis of the Static Load Test," *Short Course Text Notes*, Deep Foundations Institute, Hawthorne New Jersey.

Hannigan, P.J., Rausche, F., Likins, G.E., Robinson, B.R., and Becker, M.L. (2016). "Design and Construction of Driven Pile Foundations" Geotechnical Engineering Circular No. 12, Vol. I and II, National Highway Institute, U.S. Department of Transportation, Federal Highway Administration, Publication No. FHWA-NHI-16-010.

Komurka, V.E., Wagner, A.B., and Edil, T.B. (2003a). "A Review of Pile Set-Up," *Proceedings of the 51st Annual Geotechnical Engineering Conference*, University of Minnesota, St. Paul, Minnesota, pp. 105-130.

Komurka, V.E., Wagner, A.B., and Edil, T.B. (2003b). "Estimating Soil/Pile Set-Up," Wisconsin Highway Research Program Contract No. 0092-00-14, Report No. 03-05.

Komurka, V.E. (2004). "Incorporating Set-Up and Support Cost Distributions into Driven Pile Design," *Current Practices and Future Trends in Deep Foundations*, ASCE/Geo-Institute, Geotechnical Special Publication No. 125, 0-7844-0743-6, pp. 16-49.

Komurka, V.E., Winter, C.J., and Maxwell, S.G. (2006). "Applying Separate Safety Factors to End-of-Drive and Set-Up Components of Driven Pile Capacity," *Geotechnical Applications for Transportation Infrastructure – Featuring the Marquette Interchange Project in Milwaukee, Wisconsin*, ASCE/Geo-Institute, Geotechnical Practice Publication No. 3, 0-7844-0821-1, pp. 65-80.

Komurka, V.E., and Arndorfer, R.P. (2009). "Driven Pile Cost Comparison for Two Large Wisconsin DOT Bridge Projects," *Proceedings of the Deep Foundations Institute 34th Annual Conference on Deep Foundations*, (CD Rom).

Komurka, V.E. (2015). "Foundation Support Cost – Applications to Driven-Pile Design," *Proceedings of the International Foundations Congress and Equipment Expo 2015*, ASCE/Geo-Institute, Geotechnical Special Publication No. 256, 978-0-7844-7908-7, pp. 990-1005.

Likins, G.E., and Rausche, F. (2008). "What Constitutes a Good PDA Test?" *Proceedings of the Eighth International Conference on the Application of Stress-Wave Theory to Piles 2008*, Lisbon, Portugal, pp. 403-407.

Likins, G.E. (2017). "Dynamic Testing: Proper Practice," *Presentation for PDCA Dynamic Testing and Analysis Workshop*, Orlando, Florida.

Morgano, C.M., and White, B. (2004). "Identifying Soil Relaxation from Dynamic Testing," *Proceedings of the Seventh International Conference on the Application of Stress-Wave Theory to Piles 2004*, Petaling Jaya, Selangor, Malaysia, pp. 415-421.

Rausche, F., Goble, G.G., and Likins, G.E. (1985). "Dynamic Determination of Pile Capacity," *ASCE Journal of Geotechnical Engineering*, Vol. 111, No. 3, Reston, VA, pp. 367-383.

Rausche, F. (1991). "Recommended CAPWAP Matching Procedure, Remedies Against Faulty Capacity Predictions," *Notes from PDA Users Day in Cleveland*, *OH*, Pile Dynamics, Inc., Cleveland, OH, pp. 1-15.