High-Capacity Pipe Piles for the Marquette Interchange Reconstruction

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ABSTRACT The confluence of Interstate Highways I-94, I-43, and I-794 immediately southwest of downtown Milwaukee, Wisconsin, comprises the Marquette Interchange. The interchange covers a large area with highly variable soil conditions. A design-phase test program consisting of 88 test piles was undertaken to quantify both the time rate and magnitude of soil set-up. The test program included 51 indicator piles, 6 axial compression load test piles, 7 companion piles, and 24 reaction piles. Multiple restrike dynamic tests were performed with the last restrike typically conducted between 27 and 56 days after initial driving. Significant increases in shaft resistance occurred, with the poorest soil conditions exhibiting nearly a five-fold increase. Based on the test program results, 324 mm, 356 mm, and 406 mm O.D., closed-end pipe piles with maximum design loads of 1,334, 1,779, and 2,224 kN, respectively were used for the interchange foundations. These high-capacity pipe pile foundations were chosen because of reduced substructure size, reduced utility impact, and the associated cost savings provided by high-capacity foundation units.

During interchange reconstruction, the construction schedule could not accommodate long-term restrikes. Therefore, a select number of test piles were dynamically tested during installation and during a short-term restrike at each of the 261 substructure locations. The capacity at the end of driving and the short-term set-up information was compared to the soil set-up magnitude and time rate of set-up quantified in the design-phase test program, and was used to develop pile installation criteria for the cost-effective, high-capacity pile foundations. Depth-variable driving criteria were used which incorporated the reduced end of driving capacity required with increasing pile penetration depth due to the increased soil set-up that occurred with deeper pile penetration depths.

INTRODUCTION

The Marquette Interchange is a four-level interchange connecting I-94, I-43, and I-794 near Milwaukee, Wisconsin. The interchange and associated main line structure include 23 bridge structures. Individual bridges range from 61 to 697 m in length, and cover an area approximately 2020 m long from east to west and 1320 m long from north to south. The interchange was reconstructed between 2005 and 2008 to improve traffic capacity and safety at a cost of 810 million dollars.

The geology in the interchange area is quite variable, and consists of alluvial and estuarine (organic) deposits over glacial till sheets with interbedded lacustrine deposits. Bedrock in the core area of the project is at least 60 m below grade. The project designer, Milwaukee Transportation Partners, delineated the interchange footprint into three soil sectors, Sectors A, B, and C. The majority of the project was situated in Sectors A and B, with a small portion located in Sector C.

Sector A consisted of 0 to 10 m of surficial fill materials overlying silty clays to the termination depth of the borings, or until bedrock was encountered. Unconfined compression strengths of the clay deposits ranged from 48 to in excess of 432 kPa. The moisture contents of the clay deposits ranged from 8 to 29%, with total unit weights from 20 to 23 kN/m^3. Sector A also included a significant number of 1.5 to 7.6 m thick layers or pockets of dense to very dense silty fine sand, or very dense sand and gravel with lenses of silt and clay.

The general soil profile in Sector B consisted of 1.0 to 8.8 m of surficial fill overlying 0.3 to 18.9 m of organic silts and clays. The organic deposits had unconfined compression strengths of 24 to 86 kPa, moisture contents of 18 to 328%, and total unit weights ranging from 12 to 20 kN/m^3. Interbedded layers of clay, gravel, sand, and silt of lacustrine and glacial till origin underlay the organic deposits. The clays were typically stiff to hard with unconfined compression strengths of 67 to 460 kPa, moisture contents of 10 to 30%, and total unit weights ranging from 20 to 24 kN/m^3. The thickness of the clay deposits in this sector ranged from 1.5 to 24.4 m. The clay deposits were generally underlain by dense to very dense sands and gravels. Where silty sands underlay the organic deposits, they were generally loose to medium dense, and underlain by dense to very dense gravels. The thickness of the granular deposits ranged from 0.9 to 18.4 m. A continuous bearing layer was not identified across this sector.

In Sector C, 1.8 to 5.2 m of surficial fill materials again overlay 10.6 to 17.1 m of organic silts and clays. These soils were underlain by 4.7 to 34.3 m of alluvial deposits that included medium to dense fine sands over dense to very dense gravels with sand. The alluvial deposits were underlain by stratified layers of glacial till and lacustrine deposits. The glacial tills included very stiff to hard silty clays layers that were 1.2 to 3.8 m or more in thickness, as well as very dense fine sand and gravel layers that were as thick as 20.5 m. Boulders were noted in some borings below a depth of 40 m.

Soil Set-up

In the Milwaukee area, piles driven into the described soil deposits typically have a significantly greater capacity with elapsed time relative to their capacity at the time of driving (Fellenius et. al. 1989, Komurka 2004). This phenomenon, referred to as soil set-up, occurs from an increase of shaft resistance developing over time after pile installation. Set-up is likely the result of the closing of the annulus around the pipe pile caused by an oversized boot plate, the remolding of disturbed soil against the pile shell, consolidation and strength gain as driving-induced excess pore water pressures dissipate, thixotropic soil strength gain (aging effects), or a combination of these effects. The rate of soil set-up depends on the type and size of pile and soil properties. The majority of soil set-up may occur within weeks to a few months after driving, but set-up may continue to increase for several months or years. Quantifying the capacity at the end of driving as well as the capacity gain from soil set-up results in a more-accurate capacity estimate, and therefore a more-economical pile installation. After an early non-linear phase, the soil set-up rate is generally linear with respect to log time (Komurka et. al. 2003). In soils with large set-up characteristics, restrike tests at about 2 hours, and 1, 10, and 30 to 40 days after installation can be extremely helpful in quantifying the soil set-up magnitude and time rate of soil set-up.

In 2000, the nearby 6th Street Viaduct was reconstructed. The Wisconsin Department of Transportation (WisDOT) had review responsibilities for this design-build City of Milwaukee project, and was thereby exposed to the cost-effectiveness of high-capacity pipe piles in a similar geologic profile. WisDOT standard practice for pile foundations at that time consisted of driving piles assigned an allowable load of 535 to 625 kN to a termination blow count based on a modified version of the Engineering News formula.

DESIGN-PHASE PILE TEST PROGRAM

WisDOT was receptive to a design-phase test program for the Marquette Interchange. The interchange designer proposed a design-phase pile test program consisting of forty-three indicator pile locations and six static load test sites. The goals of the pile test program were to confirm drivability of multiple pile sections, document attainable pile capacities, and to characterize the soil set-up magnitude, profile, and time rate. With this information the designer could then select the most-cost-effective pile section for a desired load at each substructure location.

Indicator Piles

The test pile program consisted of driving 324 mm O.D. x 9.5 mm wall closed-end pipe indicator piles at forty-three locations within the interchange footprint. All of the 324 mm piles were equipped with a 25.4 mm thick x 336.5 mm O.D. flat boot plate. At seven of these indicator pile locations, a 406 mm O.D. x 12.7 mm wall closed-end pipe was also installed. Five of the seven 406 mm piles were driven with a 25.4 mm thick x 419.1 mm O.D. flat boot plate, and two were driven with a conical pile tip. All of the

pipe piles conformed to ASTM A-252 Grade 3 steel having a minimum yield strength, f_y, of 310 MPa.

Indicator piles were dynamically tested during initial driving and during a short-term restrike typically 2.5 to 12 hours after initial driving. The test pile driving rig then moved to the next indicator pile location. One to three days after installation the indicator piles were filled with concrete having a specified minimum 28-day compression strength of 41 MPa. Long-term restrike tests were conducted on the concrete filled pipe piles using a drop hammer system typically between 12 to 16 days after installation and again between 26 to 36 days after installation.

Load Test Sites

In addition to the indicator pile locations, six load test sites were pre-selected across the interchange footprint based on structure load distributions, subsurface stratigraphy, and construction access. At each of the load test sites, six closed-end pipe piles were driven; including two 324 mm O.D. x 9.5 mm wall, two 356 mm O.D. x 12.7 mm wall, and two 406 mm O.D. x 12.7 mm wall. Test pile and reaction pile sections were varied at the load test sites such that three 324 mm and three 356 mm piles were statically load tested. All piles driven at the load test sites were equipped with a 25.4 mm thick flat boot plate except for one of the 406 mm piles at each site which was driven with a conical tip. All piles at the load test sites also conformed to ASTM A-252 Grade 3 steel.

At the load test sites, all piles except for the static load test pile were dynamically tested during restrike at approximately 2.5 hours and 24 hours after installation. Long-term restrike tests with dynamic measurements were also performed on concrete filled piles with the drop hammer system between 9 to 21 days after installation, and at an additional time of 27 to 56 days after installation. Static load test piles were not restruck until after the static loading test was performed to minimize disturbance to the soil set-up process. Instead, a companion pile (same pile section driven to the same penetration depth as the static load test pile) was driven at each load test site and the companion pile was restruck multiple times. Static load test piles were restruck with dynamic measurements within 48 hours of completing the static loading test for purposes of static and dynamic test correlation. These restrike tests occurred between 35 to 49 days after installation. Restriking the static load test piles allowed a few piles at the load test sites to be dynamically tested during restrike one additional time 77 to 86 days after installation.

Pile Driving Equipment

Two pile installation rigs were used during the test pile program. Indicator pile locations and test piles sites were assigned to different pile driving systems so that pile installation data was available on two hammer types to assist contractors in the subsequent construction-stage bidding.

A Delmag D46-32 single acting diesel hammer and a Junttan HHK-10A (a uniquely modified version of a HHK-9A) were used for pile installations and short-term restrike events. The Delmag D46-32 hammer had a ram weight of 4,600 kg and a rated stroke of 3.99 m, resulting in a manufacturer's rated energy of 165.7 kJ. This hammer was also equipped with a four-step fuel pump that allowed operation at reduced stroke levels.

The Junttan hammer had a ram weight of 9,970 kg and a rated stroke of 1.2 m, resulting in a rated energy of 117.7 kJ. The Junttan hammer was operated over a wide range of stroke heights depending upon the soil resistance encountered.

Long-term restrike tests on the concrete filled pipe piles were performed using a free-release drop hammer system with a 13,600 kg ram. Drop heights of up to 2.56 m were used with this drop hammer system which corresponds to a rated energy of 341.7 kJ.

Dynamic Monitoring and Analysis

Dynamic measurements were acquired during initial driving and during the multiple restrike events on all indicator piles, as well as on all piles driven at the load test sites. The dynamic measurements were processed in the field with a Pile Driving Analyzer (PDA) using the Case Method equations for assessing pile capacity, driving stresses, energy transfer, and pile integrity (Rausche et. al. 1985). These dynamic test results were subsequently processed versus pile penetration depth for initial drive events and versus blow number for restrike events using the PDIPLOT program (Pile Dynamics 2005). Figure 1 presents three graphs of dynamic test results versus pile penetration depth illustrating typical site variability. Case Method capacity results and the pile penetration resistance (blow count) versus pile penetration depth are presented in the left graph, energy transfer and hammer stroke are shown in the center graph, and the average maximum compression stress at the gage location and hammer stroke are presented in the right graph. The variable thickness and density of the soil layers is apparent in the changes in pile capacity and blow count. The potential challenge of driving a pipe pile for a high capacity is also illustrated by compression stresses at final driving that approached 290 MPa. Compression driving stresses are routinely limited to 0.9 f_y or 279 MPa for ASTM A-252, Grade 3 steel.

The termination of pile driving at indicator pile locations and load test sites was a field decision made by an experienced engineer. The engineer considered the subsurface conditions depicted by the project borings, pile penetration depth vs. capacity estimates from pre-driving static analyses, assumed soil set-up magnitude, as well as the dynamic monitoring results available in the field during initial driving.

A representative blow from all dynamic test events was analyzed by the CAPWAP signal-matching method (Rausche et. al. 1972). Figure 2 presents a typical force and velocity record acquired from the end of initial driving and from restrike events conducted 0.06, 0.7, 14.7, and 43.8 days after initial driving. The CAPWAP calculated

shaft resistance increased from 453 kN to 2122 kN, or nearly five-fold between the end of initial driving and the 43.8 day restrike event.
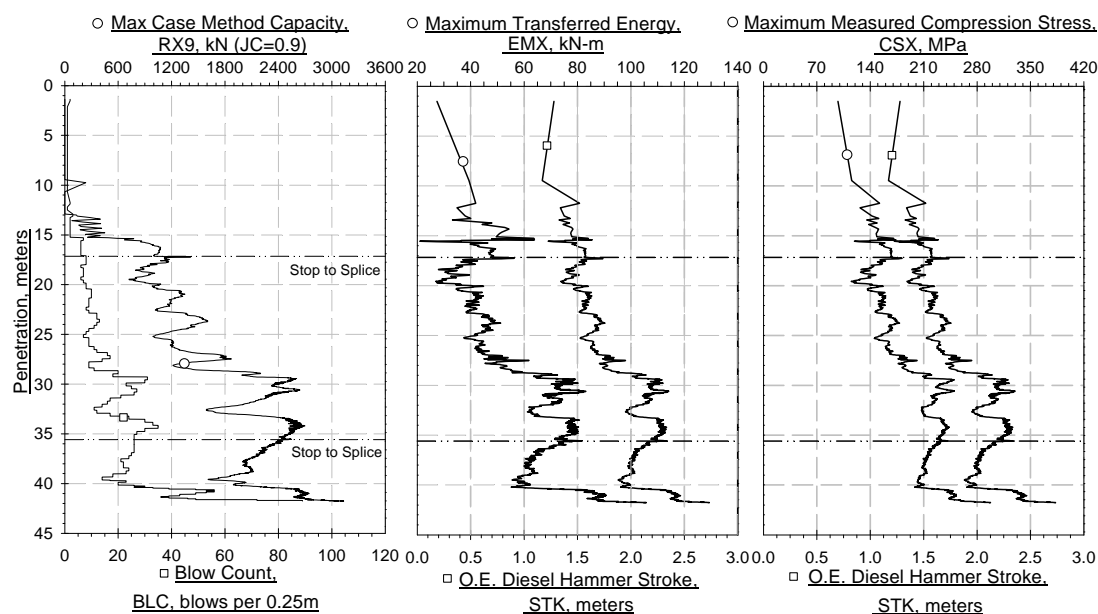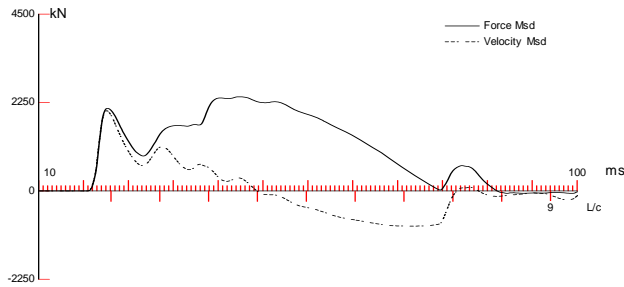

Fig. 1 Dynamic test results for 324 mm x 9.5 mm wall indicator pile in Sector C.

All CAPWAP analyses were performed using the residual stress option which considers residual compression stresses locked into the long pipe piles during driving. In general, a CAPWAP analysis performed using the residual stress option will shift a portion of the shaft resistance to the pile toe compared to an analysis performed without using the residual stress option.

Static Loading Tests

One static axial compression loading test was performed at each of the six static load test sites. Four of the piles installed at each test site served as reaction piles and were driven in a square pattern with the static load test pile driven in the center.   The purpose of the static loading test was to confirm pile capacity after soil set-up, and to aid in determining the shaft resistance distribution from embedded vibrating-wire strain gages.   The load transfer data was compared to the CAPWAP shaft resistance distributions, and this information was then used to quantify soil set-up at load test and indicator pile sites across the interchange.
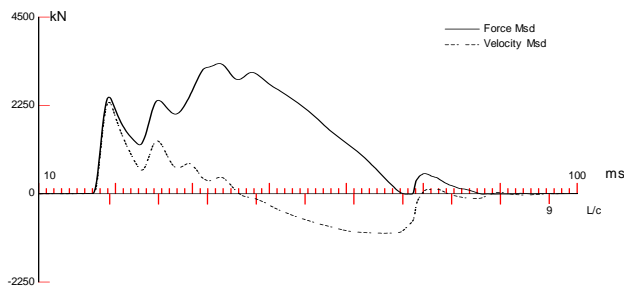
The static loading tests were performed by jacking against an overhead steel frame assembly attached to the four reaction piles. The assembly was modular for easy breakdown and reassembly at the next load test site, and was designed to apply a maximum test load of 5,340 kN. A calibrated load cell was the primary means of
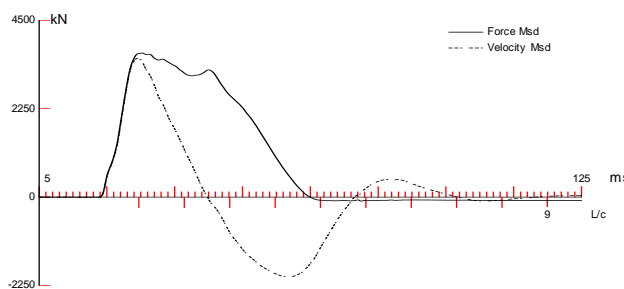
Fig. 2 Force and velocity records on 324 mm companion pile at Site F, Sector A.

measuring the applied load, with the jack pressure gage used as backup. A spherical bearing plate was included with the system to reduce eccentric loading. The pile head movement under applied loads was measured by two linear variable displacement transducers (LVDT's) mounted diametrically opposed on the pile shell. Backup vertical movement measurements were provided by two mechanical dial gages mounted 90 degrees offset from the LVDT's and diametrically opposed to one another. The LVDT's and dial gages were located equidistant below the pile head, and equidistant out from the pile shell. Movements were referenced to two independently supported horizontal reference beams.

Axial compression loading tests were performed in general accordance with the ASTM D-1143 quick test loading option. Each load increment was maintained for 10 minutes. Instrumentation was read immediately after each load increment application, and immediately prior to the next load increment application. At test's end, the pile was unloaded in 4 approximately equal decrements. Pile capacity was determined in accordance with the Davisson offset method (Davisson 1972). The static loading test load-deflection plot for a 40.8 m long, 324 mm O.D. x 9.5 mm wall, concrete-filled pipe pile is presented in Figure 3.

A comparison of the pile capacities determined from static loading tests and from CAPWAP analysis of restrike data on the static load test pile after the load test is presented in Figure 4. Two of the six load test piles, B and E, had capacities in excess of the 5340 kN limit of the loading system. However, the capacities of these piles were mobilized dynamically with the large drop hammer system resulting in the CAPWAP capacities being 1200 to 1400 kN greater than the maximum applied static load.

Axial Compression Load Transfer Instrumentation

Prior to filling the axial compression load test piles with concrete, vibrating-wire strain gage sister-bar assemblies were installed in the piles. The top most internal vibrating-wire strain gage (VWSG) was located at the ground surface elevation. Four re-usable vibrating-wire strain gages were surface mounted, 90 degrees apart on the exterior surface of each static load test pile at the same elevation as the internal strain gage. These internal and external strain gages provided redundancy in measuring the jack and load cell applied load, aided in confirming composite-section action of pile steel shell and concrete, and assisted in determining composite section (steel and concrete) elastic modulus. Elastic moduli values were determined from VWSG data as described by Fellenius (2001). Another internal strain gage was located just above the pile toe to provide information on load reaching the pile toe. The remaining internal strain gage stations were placed at selected pile depths based on test site stratigraphy to provide information regarding shaft resistance distribution and load transfer. Figure 5 presents the load transfer plot obtained from a static loading test conducted on a 356 mm O.D. concrete filled pipe in soil Sector B.
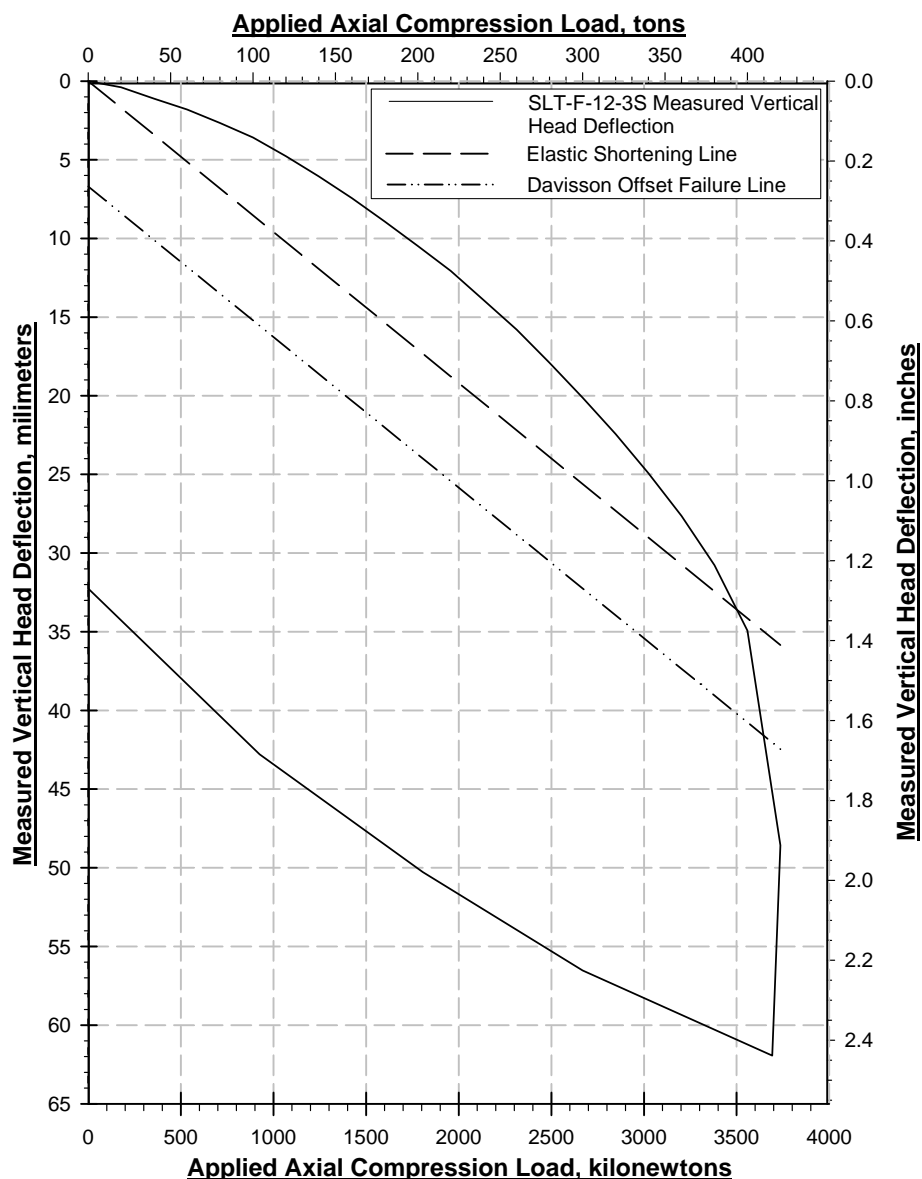
Applied Axial Compression Load, tons

Fig. 3 Static loading test result from Site F in soil Sector A.

Unit Shaft Resistance and Unit Shaft Set-up

Unit shaft resistances determined from the static loading tests and from CAPWAP analysis on the load test pile restrikes performed after static test completion were compared to assess the accuracy of the unit shaft resistance distributions. Residual stresses due to driving, determined from the CAPWAP analyses performed using the residual stress option, were accounted for when evaluating the VWSG load transfer data. Figure 6 presents the unit shaft resistance values determined from the load transfer data in Figure 5, along with the unit shaft resistance values determined from restrike CAPWAP analysis on the post loading test restrike.
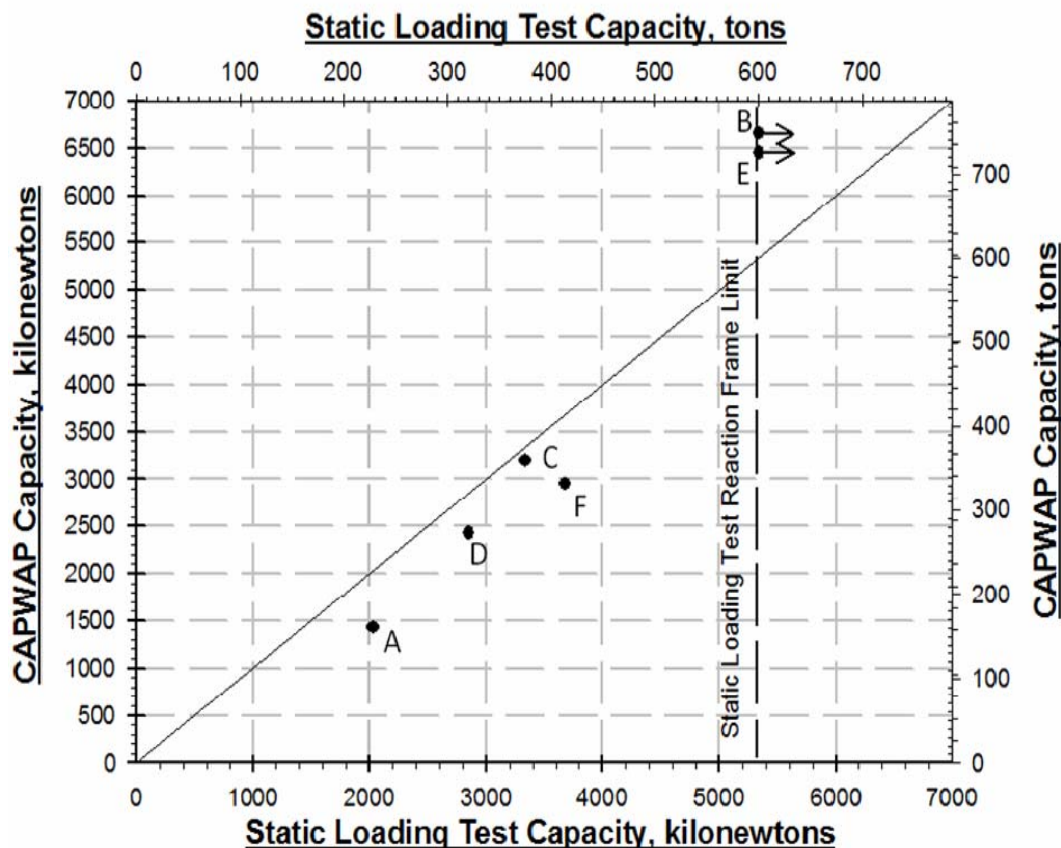
Static Loading Test – Pile Details

Load Test	Soil Sector	Pile Diameter, mm	Penetration Depth, m		Load Test	Soil Sector	Pile Diameter, mm	Penetration Depth, M
A	A	324	17.7		D	B	324	34.6
B	B	356	31.6		E	A	356	32.2
C	B	356	35.7		F	B	324	40.2

Fig. 4 Capacity correlation between static loading tests and CAPWAP results.

The unit shaft resistances from end of driving CAPWAP analysis were subtracted from the unit shaft resistances determined from the CAPWAP analysis on restrike events to estimate the magnitude and location of soil unit set-up. Figure 7 presents the unit set-up versus elevation from four restrike events on a 406 mm O.D. pile. Significant soil set-up developed on the lower portion of the pile between the 19 and 55 day restrike events.
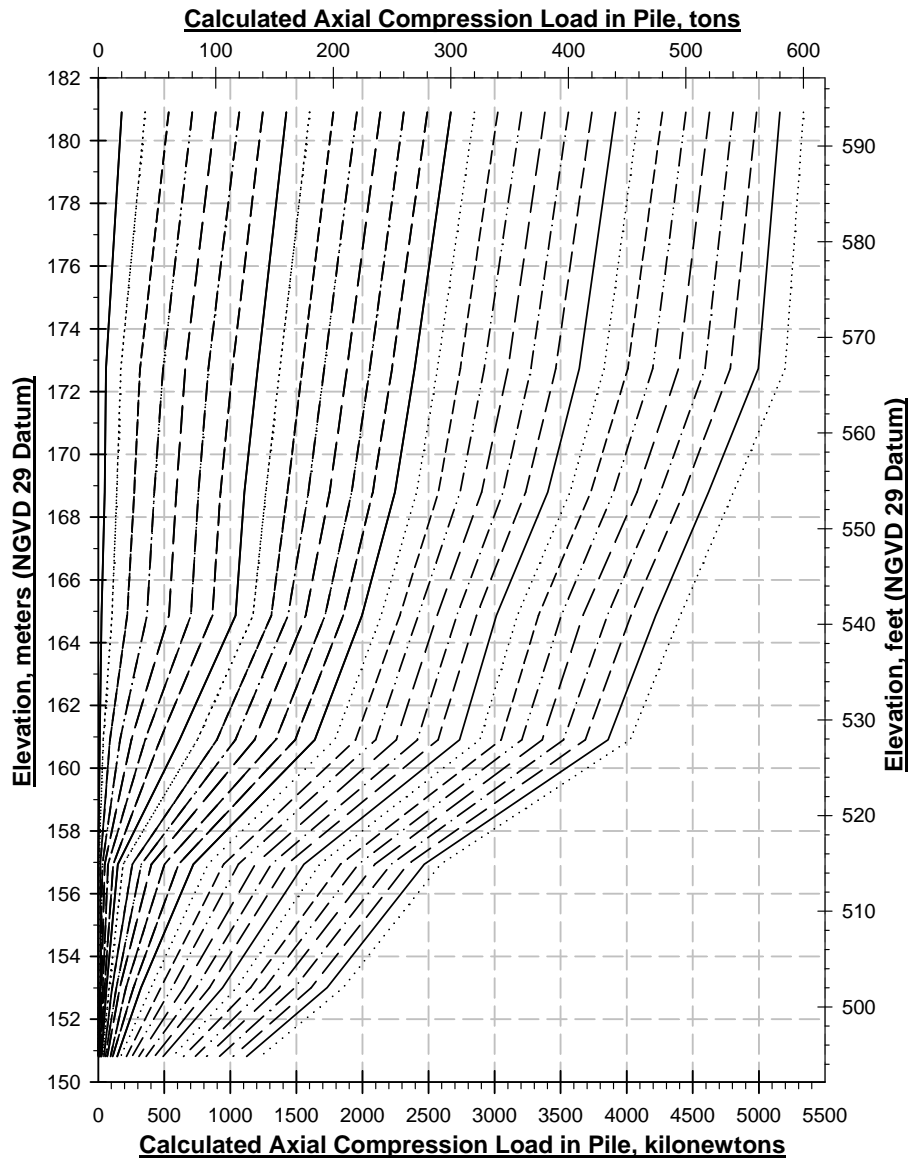
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Fig 5. Load-transfer plot for 356 mm O.D. concrete filled pipe pile in soil Sector B.

Plots of the cumulative soil set-up versus pile toe elevation were developed for each pile with long-term restrike data. An example of a cumulative soil set-up versus toe elevation plot is presented in Figure 8. This figure also contains a plot of the average Case Method capacity versus toe elevation. By adding the Case Method capacity and cumulative soil set-up at a given elevation, a plot of the estimated long-term capacity versus toe elevation was developed.
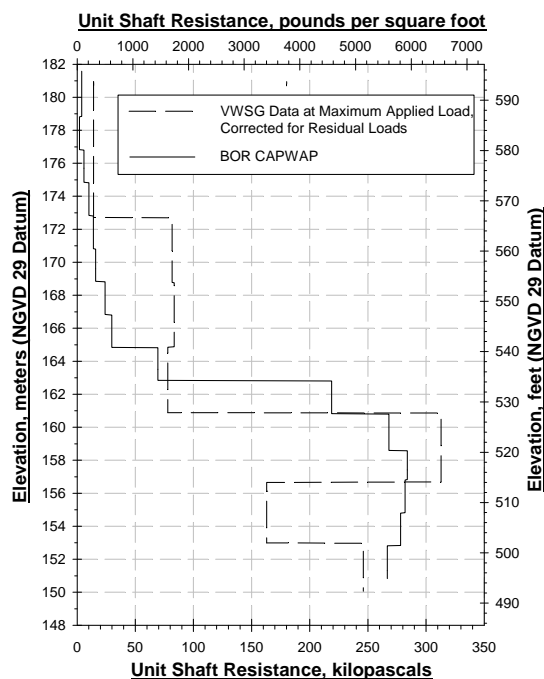
Unit Shaft Resistance, pounds per square foot

Fig. 6 VWSG and CAPWAP determined unit shaft resistances on 356 mm pile in Sector B.
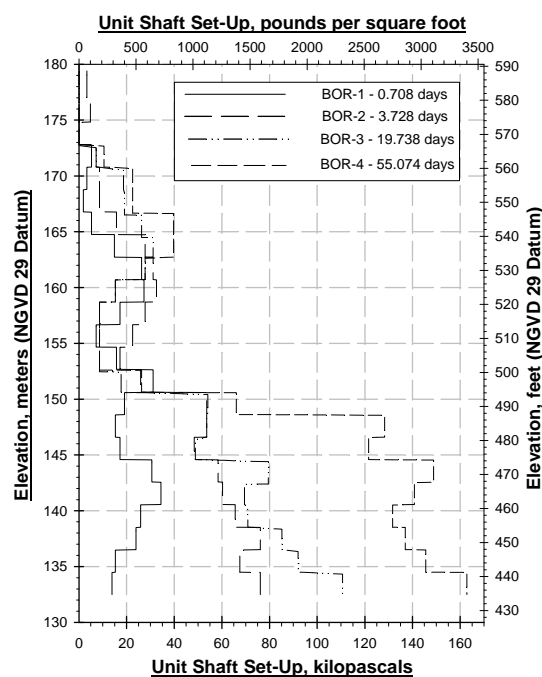
Fig. 7 Unit set-up from CAPWAP analyses on multiple restrike events on 406 mm pile in Sector B.

Foundation Design Applications

Due to cost considerations as well as the expectation that smaller diameter lower design load piles would be used, most of the indicator piles were 324 mm O.D. pipe piles. To assess the advantages of larger diameter piles and higher design loads, capacity data was extrapolated from the 324 mm O.D. section to 356 mm O.D. and 406 mm O.D. piles for pile length and foundation cost assessments. Capacity results for the 324 mm piles were multiplied by 1.13 for 356 mm piles, and by 1.40 when extrapolating 324 mm results to 406 mm piles. The multiplication factors were determined following a parameter study that considered variation in the soil resistance distributions as well as the ratios of the shaft and toe areas. This extrapolation appeared valid based on capacity comparisons at indicator pile locations and static load test sites where multiple pile sections were installed.

The average Case Method capacity at a given pile toe elevation was divided by a safety factor of 2.25 to yield an allowable load at the time of driving at that toe elevation. Similarly, the cumulative soil set-up at a given toe elevation was divided by a safety factor depending upon pile diameter to yield an assigned portion of soil set-up at that toe elevation. Initially, a higher safety factor was applied to the contribution from soil set-up to account for the variability in soil conditions and pile size. These two components were combined into an allowable geotechnical load versus pile toe elevation plot. One of these allowable load versus pile toe elevation plots was then

assigned to each substructure location based on the proximity of the substructure location to the indicator pile or static test sites as well as the soil stratigraphy. Figure 9 presents plots of the allowable geotechnical load versus pile toe elevation for 324 mm, 356 mm, and 406 mm pipe piles developed in this manner.
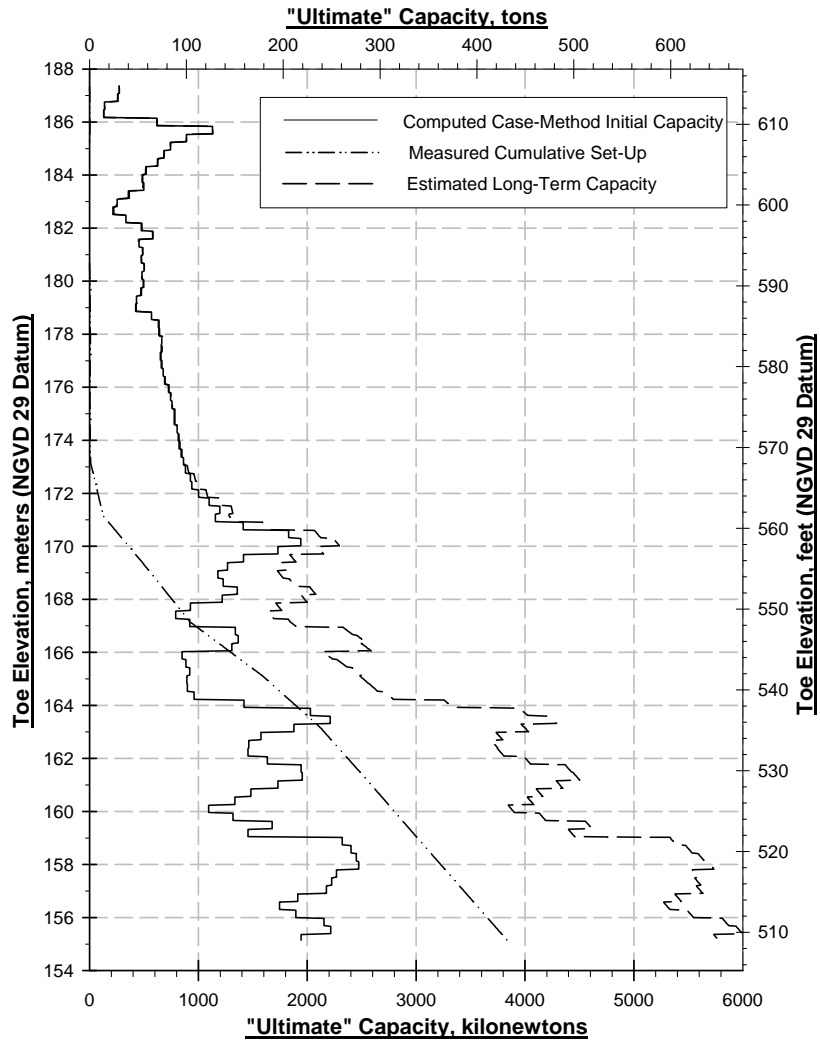

Fig. 8 Plots of Case Method capacity, cumulative soil set-up, and estimated long-term capacity vs. toe elevation for 356 mm pile in Sector A.

Based on cost information solicited from contractors, as well as historical pile material and pile driving costs, an approximate installed cost per 0.3 m of pile was developed for each candidate pile section. The estimated allowable geotechnical load at a given toe elevation divided by the pile installation cost to that toe elevation yielded a plot of the pile support cost in dollars per kN of allowable load. A typical plot of pile support cost versus pile toe elevation is presented in Figure 10.

Using figures similar to Figures 9 and 10 for a specific substructure location and foundation load, the project designer could quickly estimate the pile length for a given pile section, as well as the approximate cost for a potential pile group configuration. Group settlement and lateral load considerations also influenced final section selection.
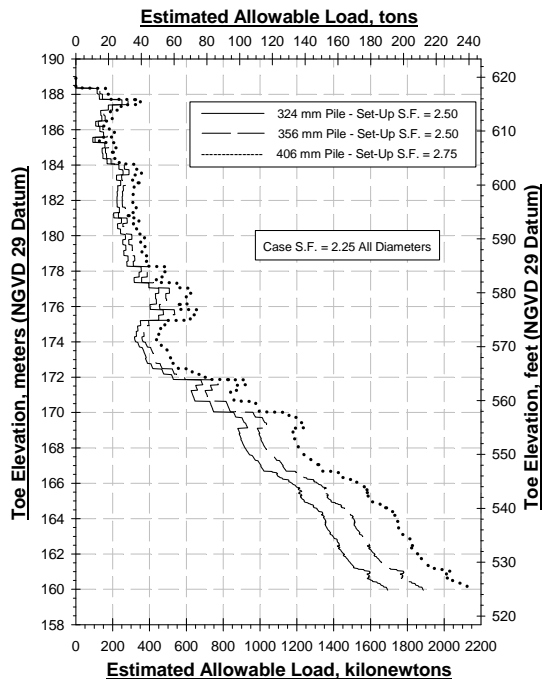

Fig. 9 Allowable geotechnical load vs. pile toe elevation for three candidate sections pile sections in soil Sector A.
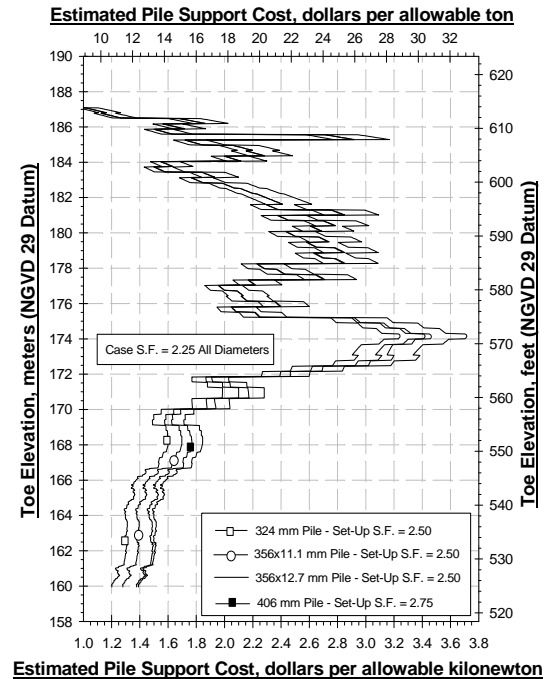
Fig. 10 Pile support cost vs. pile toe elevation for four candidate pile in soil Sector A.

Time Rate of Soil Set-Up

The time rate and magnitude of soil set-up was evaluated at each indicator pile location and load test site. The CAPWAP determined shaft resistance at end of initial driving was subtracted from the CAPWAP determined shaft resistance of a blow acquired near the beginning of each restrike event. This difference in shaft resistance was the cumulative soil set-up at the pile toe elevation at a given time. The cumulative soil set-up magnitude from each restrike event was then plotted versus log time. A cumulative soil set-up versus log time plot from static load test site F in Sector B with 324 mm, 356 mm and 406 mm O.D. piles is presented in Figure 11. The shaft set-up determined from the shaft resistances noted earlier in Figure 2 are identified in Figure 11 as pile SLT-F-12-6C. The shaft resistance determined from the static loading test result presented in Figure 3 is also identified in Figure 11 as SLT-F-12-3S.

During production pile installations, short-term restrike tests were conducted to check if the soil set-up magnitude was developing at the expected depth and at the expected rate based on the indicator pile and load test site data.
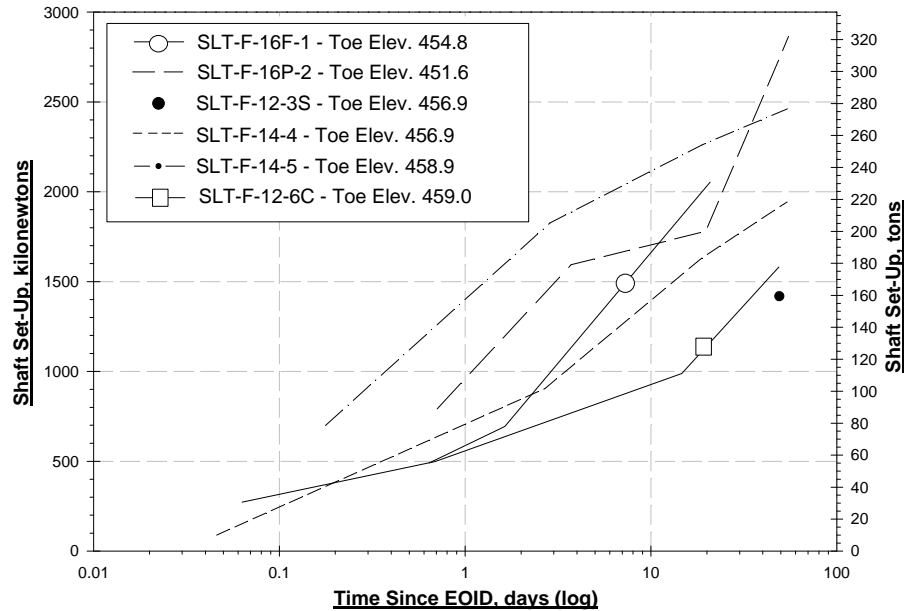
Fig. 11 Cumulative shaft set-up versus log time from a load test site.

PRODUCTION PILE INSTALLATION AND CONSTRUCTION CONTROL

High-capacity pipe piles that achieved a large portion of their capacity through soil set-up were used on the West Leg, South Leg, and Core contracts for the Marquette Interchange. One contractor, Marquette Constructors, LLC, installed the 324 mm, 356 mm, and 406 mm O.D., closed-end pipe piles on all three contracts. The maximum design loads used on these sections were 1334, 1779, and 2224 kN, respectively. At the peak of the project, the contractor had 10 pile driving rigs driving production-phase test piles and/or production piles. All of the production pile driving was completed using single acting diesel hammers. The majority of production pile driving was performed by eight pile hammers; six Delmag D46-32 hammers and two Delmag D62-22 hammers. The manufacturer's rated energy for these hammers was 165.7 kJ for the Delmag D46-32 and 223.3 kJ for the Delmag D62-22. Occasionally, Delmag D30-32 hammers with a manufacturer's rated energy of 102.3 kJ were also used.

During the construction phase, one or two dynamic test piles were monitored at 261 substructure locations during initial driving and again during restrike 2 to 3 days after installation. A total of 474 of the over 5,000 production piles were dynamically tested in this manner. CAPWAP analyses were performed on the dynamic test data from the end of driving and the beginning of restrike on all dynamically tested piles.

Prior to test pile driving at each substructure location, a minimum capacity during driving was established. This minimum capacity was the required capacity less the anticipated cumulative soil set-up at a given pile penetration depth as illustrated in Figure 12.
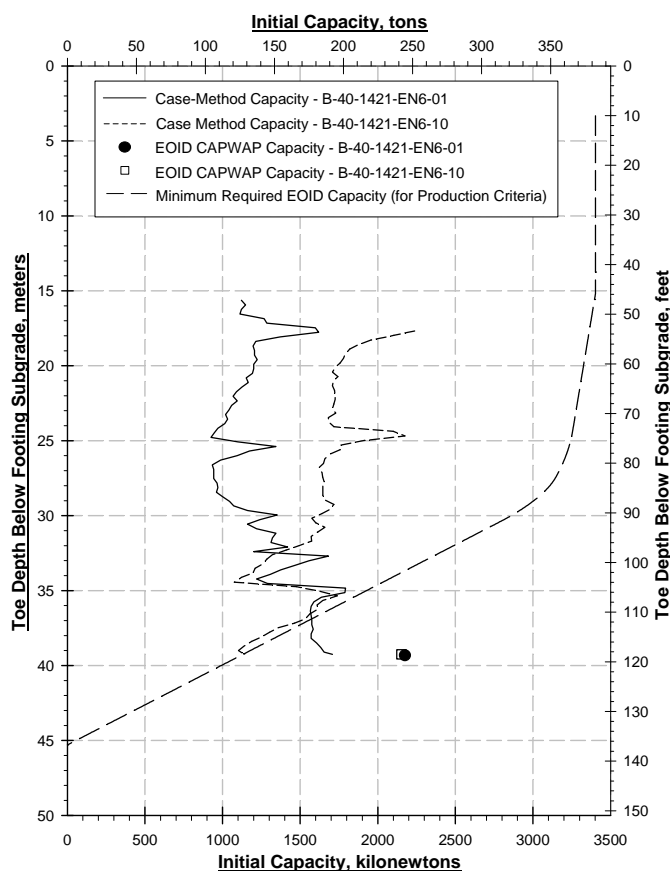
Fig. 12 Construction-phase dynamic test pile results, soil Sector A.

Also shown in Figure 12 are the Case Method capacity results versus pile penetration depth for the two construction-phase dynamic test piles. The construction-phase test piles were terminated when the Case Method capacity exceeded the required minimum capacity during.

CAPWAP analyses were performed on the dynamic test data acquired at the end of driving and at the beginning of restrike. The shaft set-up at the time of restrike was determined from the difference in the shaft resistances from these CAPWAP analyses, and the set-up compared to the design-phase test program results as shown in Figure 13. In the data presented, Pile EN6-01 had exceeded the anticipated soil set-up by 267 kN at the time of restrike whereas Pile EN6-10 was approximately 26 kN lower than anticipated.

The unit shaft set-up assigned to a substructure from the design-phase pile test program was then compared to the unit shaft set-up determined from the construction stage CAPWAP analyses as illustrated in Figure 14. An assessment was then made as to whether the set-up magnitude, location, and time rate was progressing in accordance with the design-stage data or whether modification in the set-up profile was required. The final unit set-up profile developed from this assessment was then used to establish a depth variable production pile driving criteria.
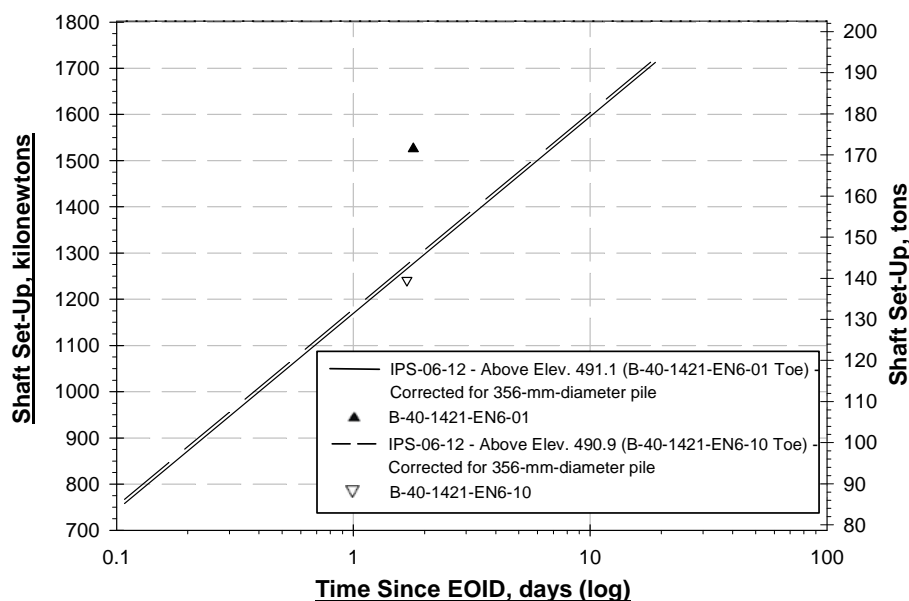
Fig 13. Comparison of test pile and production pile set-up magnitude at restrike.

Production Pile Driving Criteria

As illustrated in Figure 8, with increasing pile penetration depth, the percentage of the pile capacity carried by soil set-up could significantly exceed the capacity present at the time of driving.   Therefore, depth-variable driving criteria were generated for pile installation at each substructure location. The depth-variable driving criteria considered the decreasing capacity required at the time of driving with increasing pile penetration depth and increasing soil set-up. Since the required end of drive capacity decreased with pile penetration depth, the required pile penetration resistance or blow count also decreased with penetration depth. With single acting diesel hammers being used to drive the production piles, the depth-variable driving criteria also considered the field observed hammer stroke. The required pile penetration resistance in blows per 0.3 m of penetration was provided in tabular form for hammer stroke heights typically varying from 2.1 to 3.5 m at 0.15 m increments. A reduced pile penetration resistance was provided for every 0.6 m change in the pile penetration depth. These depth-variable driving criteria were generated using refined wave equation analyses following the procedure recommended in the FHWA *Design and Construction of Driven Pile Foundations* manual (Hannigan et. al. 1998).

Each pile driving system was assigned an identification number.   Driving system performance was recorded for each test event and monitored throughout the duration of the project. Figure 15 presents a plot of the potential energy ratio versus date for end of driving dynamic tests performed with the eight main driving systems. The potential energy ratio is defined as the average energy transfer to the PDA gage location at final driving divided by the product of ram weight and PDA computed hammer stroke. The

potential energy ratio considers that the hammer stroke varied substantially at final driving due to the soil resistance encountered, the magnitude of the anticipated soil set-up, and the resultant capacity required at final driving. Linear regression analysis of the data for each hammer typically illustrates a 2 to 5% reduction in the potential energy ratio over time. However, D46-32 hammers #1 and #3 have a more substantial 15 to 16% reduction in the potential energy ratio over time. The collected data illustrates the importance of documenting individual drive system performance on projects with multiple driving systems, as well as accounting for variation in drive system performance over time in production pile driving criteria. On this project, the variation in drive system performance over time was accounted for in the production pile driving criteria by modification of the hammer efficiency in refined wave equation analyses.
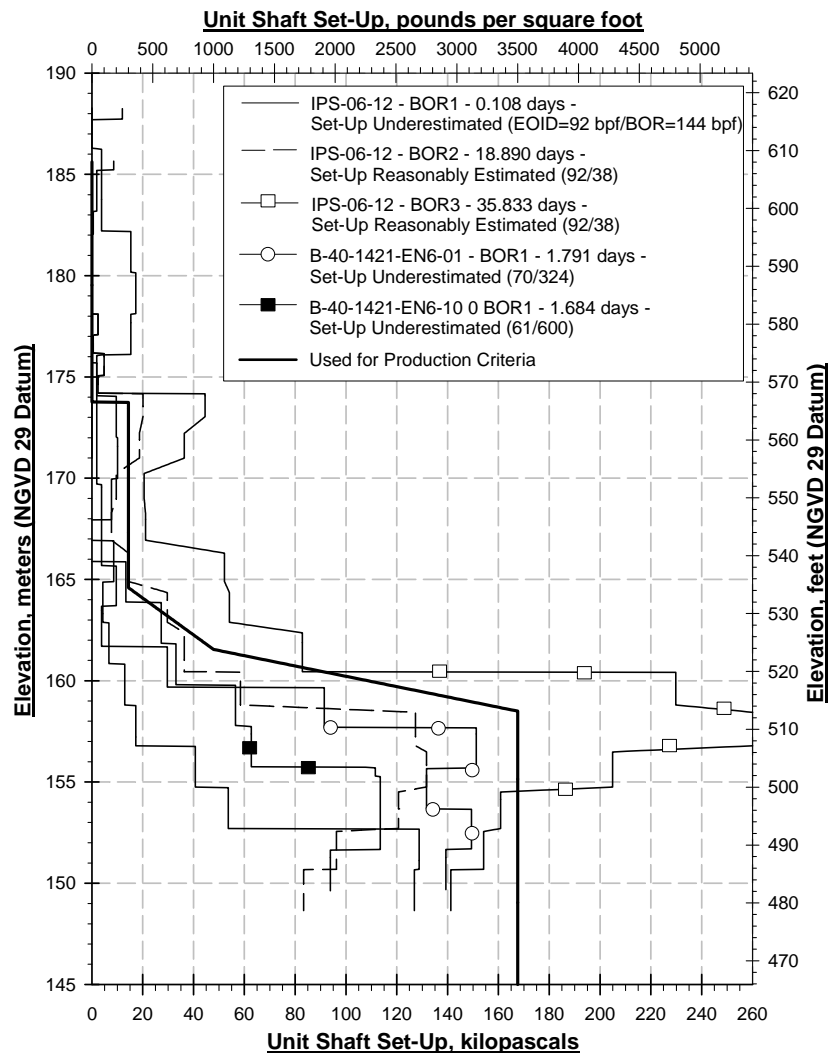

Fig. 14 Design-phase and construction-stage unit set-up comparison for a pier location in Sector A.
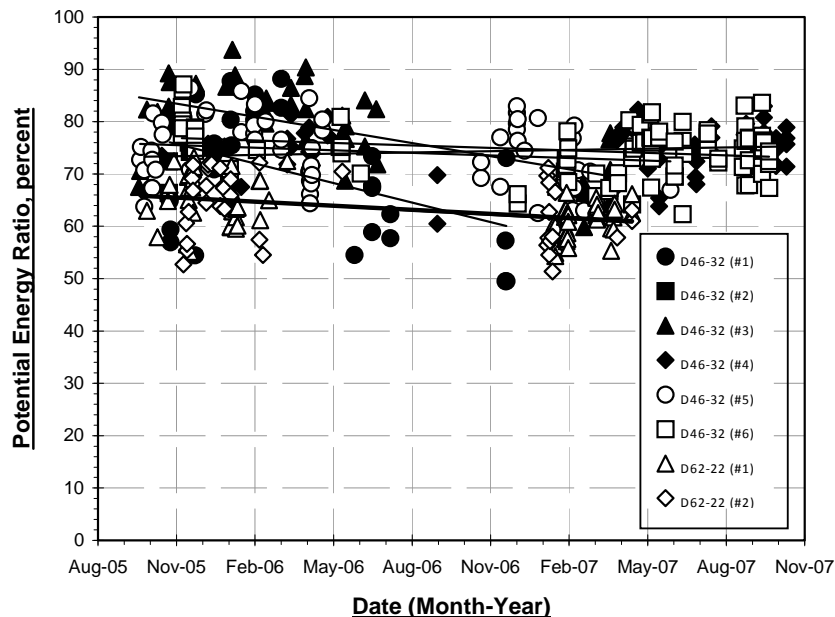
Fig. 15 Potential energy ratio vs. date for eight main pile driving systems

The construction stage testing and analysis activity was completed under very tight time constraints for data collection, analysis, and development of driving criteria. Contract specifications required the driving criteria to be available within 48 hours of completing the restrike tests at a rate of one substructure unit per day, Monday through Saturday, and in the order requested by the contractor. During the peak of project pile driving activity, it was requested that the quantity of driving criteria developed be increased to two substructure units per day. This request was satisfied to maintain the tight project construction schedule.

CONCLUSIONS

A design-phase test program confirmed high-capacity pile installations were economically attractive for the interchange final design. Based on the test program results, 25 to 50% higher design loads were used on the 324 mm, 356 mm, and 406 mm pile sections than anticipated prior to the test program. The test program also determined that the wall thickness of 324 mm pipe piles needed to be increased from 9.5 mm to 12.7 mm due to drivability considerations.

On average, a nearly five-fold increase in shaft resistance occurred between the end of initial driving and one month later. Through design-phase and construction-stage static and dynamic testing, the magnitude of soil set-up was quantified and incorporated into design and construction control.

Komurka and Ardorfer (2009) evaluated the cost-effectiveness of the high-capacity pipe piles installed on the South Leg contract of the Marquette Interchange to a nearby parallel structure in similar geologic conditions. The parallel structure was a

conventional DOT design based on standard lower-capacity piles driven using a dynamic formula with no static or dynamic testing. It was concluded that the pile support cost of the high-capacity pipe piles on the South Leg contract including testing was 27% less per unit of load supported than the nearby conventional design.

Energy measurements were made on eight pile driving systems over a 2.2 year period. Six of the eight pile driving systems showed a slight 2 to 5% decrease in performance over time, and two pile driving systems had a more substantial 15 to 16% reduction in performance. Pile installation criteria, based on wave equation analyses, were modified in accordance with variations in the measured driving system performance.

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