Bearing Capacity Reduction of Vibratory Installed Large Diameter Pipe Piles

Ethan C. Lamiman[1], EIT, A.M.ASCE, and Brent Robinson[2], PE, M.ASCE

[1]Staff Engineer, Froehling & Robertson, Inc., 833 Professional Place W., Chesapeake, Virginia 23320; ELamiman@fandr.com
[2]Senior Engineer, GRL Engineers, Inc., 30725 Aurora Road, Cleveland, Ohio, 44129; BRobinson@grlengineers.com

ABSTRACT: This paper presents a case history where 91.4-centimeter (36-inch) diameter open-end pipe piles were installed using both impact and vibratory installation techniques. Thirteen dynamically tested piles were installed along a new sheet pile containment wall located in the Southern Branch of the Elizabeth River, bounded by Chesapeake and Portsmouth, Virginia. The soil conditions encountered generally consisted of interbedded layers of silt, sand, and clay forming the Alluvium and the Norfolk Formations. The piles were advanced into the underlying Yorktown Formation bearing stratum consisting of clayey to silty sand with varying amounts of marine shell fragments. At two test pile locations, impact driven test piles were extracted and relocated by vibratory hammer and subjected to restrike driving with dynamic analysis to assess bearing capacity. Seven to fourteen day restrikes were performed on the thirteen hammer driven test piles. Restrikes at one or more months were performed on one impact driven pile, and both vibratory installed test piles. Signal matching analyses of restrike driving events indicate an approximate 50% reduction in overall bearing capacity of the vibrated piles compared with the driven piles. Additionally, long-term restrike driving of vibrated piles did not continue to gain capacity akin to driven piles.

INTRODUCTION

The installation of deep foundations has been loosely documented since the 12th century, and observed through archeology into prehistoric times (Boyer 1985). Beginning with human powered drop hammers to early steam machines and into today's diesel and hydraulic impact hammers, the understanding, design and specification of deep foundations is based upon the breadth of knowledge derived from the long history of impact driving. With the development of modern vibratory hammers come the trepidations of designers and engineers to specify their use for deep foundations requiring axial bearing capacities. The reservations for use in design and construction are mainly consequent to the lack of understanding of the soil-pile interaction during and subsequent to installation (Viking 2005).

Previous studies comparing measured bearing capacities of impact driven and vibratory installed methods have indicated differences between impact and vibratory driven piles. O'Neill et al. (1990) installed 102 mm diameter instrumented displacement piles in sands with relative densities of 65 and 90% using a vibrator and an impact hammer. For these displacement piles installed in a pressure chamber, pile bearing capacity was higher for impact driven piles in the medium dense sand and higher for vibratory installed piles in dense sand. Mosher (1987) reported on a number of case studies using a variety of pile types and hammer types. In a majority of cases summarized, piles installed in sand by impact hammer exhibited higher axial capacities than those installed solely with a vibratory hammer. This study summarizes a case history in which two open end pipe piles were installed with both an impact and vibratory hammer.

SITE DESCRIPTION

The construction site, administrated by the Environmental Protection Agency (EPA) Region 3, is under the construction jurisdiction the United States Army Corps of Engineers (USACE) Norfolk District. The project is identified as the Atlantic Wood Industries Super Fund Site, Off Shore Sheet Pile Containment Wall. Construction of the new containment wall phase of the project was accomplished by McLean Contracting Company. The project site is located on the western shore of the Southern Branch of the Elizabeth River, in Portsmouth, Virginia. The site is generally bounded to the north and west by private industrial properties and the Norfolk Naval Shipyard, to the south by the United States of America South Gate Annex property, and aerially bisected by the new South Norfolk Jordan Bridge.

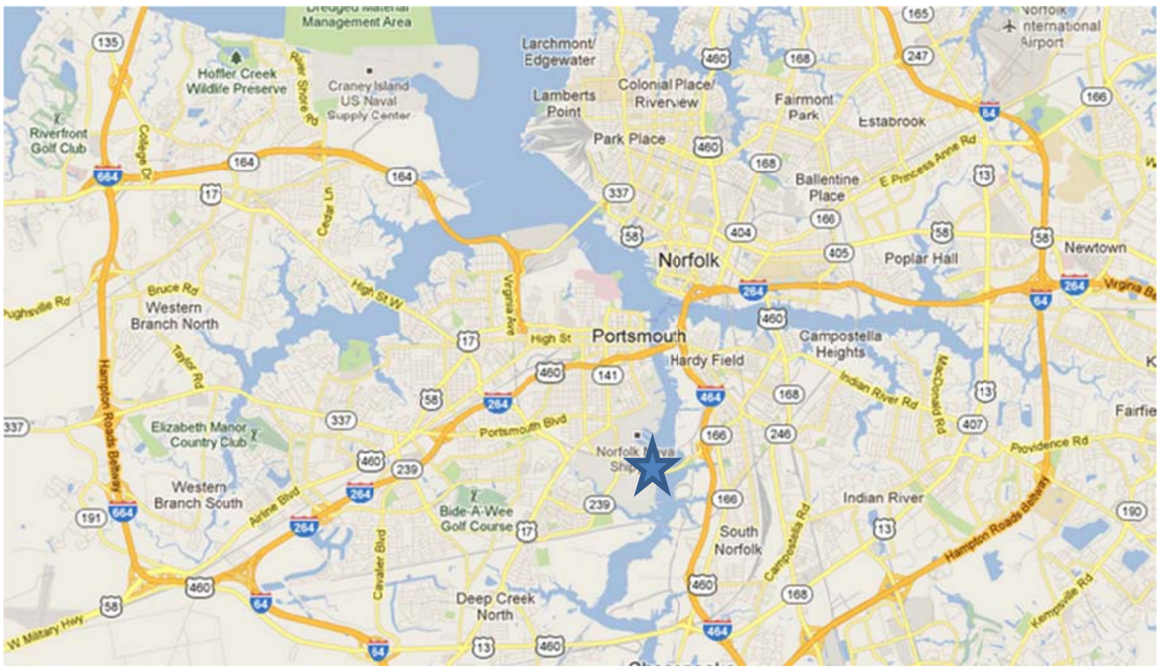

Figure 1. Project site (starred)
(Source: google.com, 2013)

The project generally consisted of a 550-meter (1,180-foot) steel combined pile bulkhead, with 35 meters (120 feet) of tied-back steel combination pile bulkhead along the river shoreline, and 112 meters (370 feet) of steel sheet pile cutoff wall constructed onshore. This construction phase of the containment wall was one of the preliminary stages of the remediation of the Atlantic Wood Industries Superfund Site.

Deep Foundation Elements

The foundation elements of the new sheet pile containment wall consist of a land based dead man wall at the southwest portion of the wall utilizing 610-mm (24-inch) square, precast, prestressed concrete (PPC) piles and 610 mm diameter spiral welded, open-end pipe piles driven alternately on a 4 Horiz: 12 Vert batter, 26.5 meters (87 feet) and 35.1 meters (115 feet) in length, respectively. The containment wall piles consist of plumb 1219-mm (48-inch) diameter open-end steel pipe piles at the southwest portion of the containment wall. The offshore portion of the containment wall consists of 914-mm (36-inch) diameter steel pipe piles with a 15.875-mm (0.625-inch) wall thickness battered 4:12 along the exterior and 1219-mm diameter open-end steel pipe piles along the interior driven plumb. The battered exterior 914-mm pipe piles were initially designed with an open-end condition, and later changed to closed-end pipe piles during the test pile program to achieve design axial compressive capacity requirements.

SUBSURFACE CONDITIONS

The project site lies within the Coastal Plain physiographic province of Virginia, which extends from the Fall Zone eastward to the Atlantic Ocean. Numerous transgressions and regressions of the Atlantic Ocean have deposited marine, lagoonal, and fluvial (stream lain) sediments. The regional geology is very complex, and generally consists of interbedded layers of varying mixtures of sands, silts, and clays.

As stated by Barker and Bjorken (1978), the site subsurface soil strata beneath the water generally consists of Alluvium of sand and marsh sediment, estuarine-beach, tidal marsh, and fluvial silt, sand and clay with organic material (peat); underlain by the Norfolk Formation of brackish marine silty sand and fluvial estuarine, silty sand. The Norfolk Formation is subdivided into the Upper and Lower member. The upper member is typically composed of brackish marine silty sand and fluvial estuarine clayey silty sand and may also include marine silt and clay deposits present in areas where the underlying Yorktown Formation has been eroded. The lower member of the Norfolk Formation generally consists of fine to coarse quartz sands with varying amounts of silt. Underlying the Norfolk Formation is the Yorktown Formation consisting of near shore marine fossiliferous, silty, coarse sand and coquina.

The Yorktown Formation is the typical bearing stratum for deep foundations in the Tidewater, Virginia, area and is considered to be over-consolidated. Very limited consolidation data indicates an average over-consolidation range of 2.1 to 3.2 (Martin, James, Powell & Bertoulin 1987).

The following generalized subsurface information shown in Table 1 contains a composite profile of collected site data and was derived from multiple subsurface exploration programs made public in the project's Contract Documents (USACE 2011). The depth of water to the Alluvium varied from five to twelve feet across the length of the wall.

Table 1. Generalized subsurface profile.

Depth (m)	Depth (ft)	Material Description	Range of N_{60} (blows per foot)	Stratum/ Formation
0–1.5	0–5	Water, Southern Branch of the Elizabeth River	–	–
1.5–4.6	5–15	Very Soft, ORGANIC SOIL, trace fine sand (OL, OH)	0	ALLUVIUM
4.6–13.7	15–45	Very Soft, FAT CLAY, trace fine sand (CH)	0 to 1	UPPER NORFOLK
13.7–21.3	45–70	Very Loose to Medium Dense, Fine to Coarse SAND, varying amounts of silt (SM, SP-SM)	0 to 13	LOWER NORFOLK
21.3–33.5^{+}	70–110^{+}	Medium Dense to Dense, Silty to Clayey, Fine to Coarse SAND, contains varying amounts of marine shell fragments	12 to 40	YORKTOWN

TEST PILE PROGRAM

Test pile installation was performed by McLean Contracting Company, based out of Glen Burine, Maryland, with regional operations in Chesapeake, Virginia. The impact driven test pile program included high-strain dynamic pile analysis during initial drive and pile restrike for two 610-mm PPC piles, seven 914-mm diameter open-end, spiral welded pipe piles, and six 914-mm diameter closed-end, spiral welded pipe piles. One 1219-mm diameter open-end, spiral welded pipe pile was dynamically monitored during initial installation.

A static load test of a 914-mm diameter closed-end, spiral welded pipe pile was performed at the southeast corner of the wall, where a member of the reaction frame, one 610-mm diameter vibratory installed, spiral welded pipe pile, was dynamically tested during restrike to confirm uplift capacity of the reaction frame.

During the test pile program, two previously impact driven 914-mm diameter test piles (66A and 111A) were extracted with a vibratory hammer following restrike, relocated in the vicinity of the initial installation, and vibratory installed to the same approximate toe elevation. These two test piles were later dynamically tested through impact restrike driving.

For the purposes of this publication, the discussion herein will consider the comparison of the 914-mm open-end pipe piles installed by impact and vibratory driving.

Hammer Assemblies

The 914-mm impact driven test piles were installed with an American Piledriving Equipment (APE) model D80-23 open ended diesel (OED) hammer assembly. The APE D80-23 has a ram weight of 78.47 kN (17.64 kips) and a rated energy of 287.7 kN-m (212.2 kip-ft) at the maximum 3.51-meter (11.5-foot) stroke. The hammer assembly was installed in 1320-mm (52-inch) Gage Swinging Leads for pipe pile installations.

Vibratory installation of 914-mm diameter piles was performed utilizing an ICE model 4450 vibratory hammer with HPSI model 500 power unit. The ICE 4450 has a dynamic force of 1619.2 kN (364 kips), an eccentric moment of 51 kg-m (4400 in-lbs), a frequency of 1,600 vpm, and an amplitude of 30 mm (1.17 inches).

Impact Driven Test Pile Installations

The 914-mm diameter impact driven test piles were installed plumb at approximate test pile locations identified alpha numerically by line and bent. The 914-mm piles were located along line A, with a bent spacing of 269.6 centimeters (106.125 inches) on center. From the southeast corner of the combination wall at pile location 45A, 914-mm test piles were linearly aligned to the wall terminus at pile location 146A.

The 914-mm test piles were initially installed and restruck with the APE D80-23 hammer assembly. Following the impact driven test pile program, two piles at location 66A and 111A were extracted and relocated at a distance of approximately 3 to 4.5 meters with an ICE 44-50 vibratory hammer.

A preconstruction wave equation analysis and driveability study was performed for the APE D80-23 hammer assembly installing the 914-mm open-end pipe piles. The preconstruction drivability study indicated a driving resistance of 40 to 50 blows per 0.3 meters considering a 90 percent partially plugged condition at the toe. Using a coring model of toe resistance, the driveability study predicted driving resistances of 13 to 14 blows per 0.3 meters. Of the test piles measured following installation, the depth of soil within the pile was observed to approximate the mud line.

Impact driven test piles were installed with the APE D80-23 hammer assembly using the lowest fuel setting (fuel setting 1). As shown by Figure 2 for impact driven open-end pipe piles, there was consistency across the site with regards to driving resistances. Open-end test piles were driven to an approximate embedment depth of 31.7 m (104 feet) below the water line.

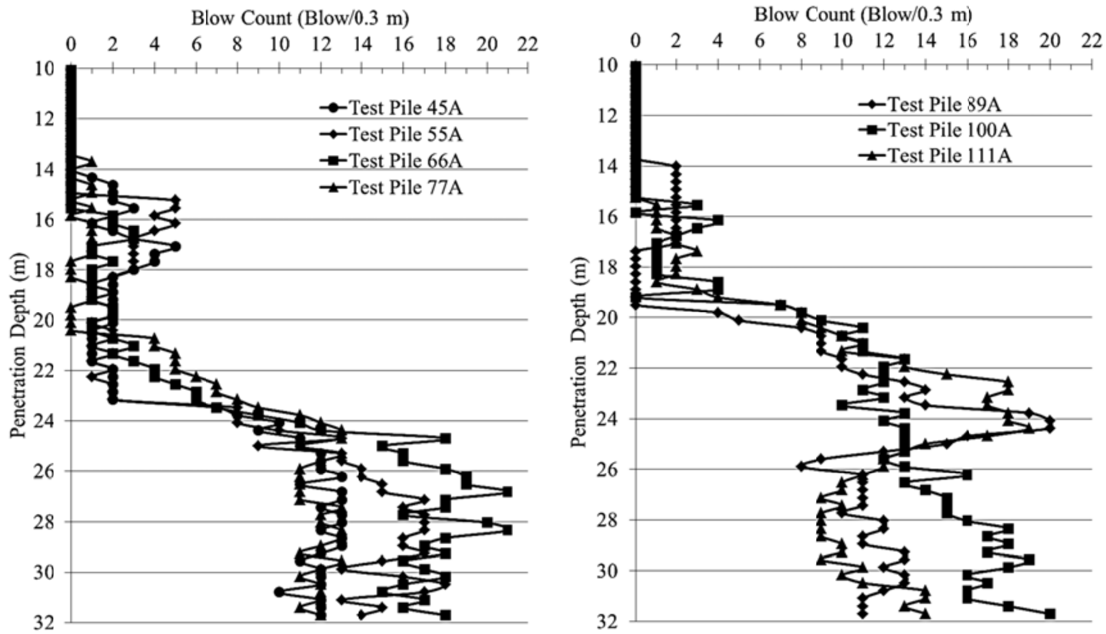

Figure 2. Impact driven pile installation summary.

Restrike Driving of Test Piles

Impact driven and vibratory installed test piles were each subjected to restrike driving using the APE D80-23 hammer assembly in conjunction with dynamic analysis. Restrike driving was performed following a range of setup periods from 5 to 47 days for impact driven piles and 26 to 68 days for vibratory installed piles.

The first test pile was restruck on the maximum fuel setting 4 with the APE D80-23 following a 13 day setup period. It was observed that the static capacity was much lower than anticipated where the signal matching with CAPWAP® software (Pile Dynamics, 2006) indicated an ultimate compressive capacity of 3200 kN (720 kips) at a driving resistance of approximately three blows per inch. Restrike driving was then attempted on fuel setting 1 on the same pile following an additional 6 day setup period after the first restrike where the signal matching analysis indicated an ultimate static capacity of 4220 kN (950 kips) at a driving rate of approximately four blows per inch. The total shaft resistance was observed to increase slightly from the reduction of fuel settings from 2800 to 3035 kN (630 to 680 kips), but the static end bearing resistance increased substantially from 405 to 1185 kN (90 kips to 265 kips). A summary of these results are presented in the following Table 2 for capacity evaluation at the end of initial installation (EOID) and successive high and low energy restrike (BOR) driving events.

Table 2. Pile number 55A energy variation during restrike driving results.

Driving Event	Date of Event	Fuel Setting	Static Pile Capacities			Hammer Performance	
			Shaft (kN)	End (kN)	Total (kN)	Max. Force FMX (kN)	Energy Transfer EMX (kN-m)
EOID	Dec-21-11	1	2180	378	2558	6272	75.5
BOR1	Jan-3-12	4	2802	400	3203	7922	104.5
BOR2	Jan-9-12	1	3025	1179	4204	5551	55.0

Following these observations, subsequent test piles were restruck utilizing fuel setting 1. Based on these results, it is likely that the reduced fuel setting and associated energy transfer into the pile allowed for any soil mass plugged onto the pile to move with the pile.

Vibratory Installed Test Piles

Test pile installation by vibratory hammer was performed with the ICE 44-50 hammer hydraulic vibratory driver for test piles 66A and 111A. This hammer has an eccentric moment of 51 kg-m (4400 in-lbs). Vibratory installation was timed at approximately nine minutes each to an embedment depth of 31.7 m (104 feet) below the water line. Dynamic measurements were not obtained during vibratory installations. Wave equation analyses were performed with dynamic parameters recommended by Rausche (2002), and indicated predicted driving times of 3 or 6 minutes, depending on whether the N-value method (SA) or the general soil type (ST) static analysis option, respectively, was used to predict static capacity.

RESULTS AND CONCLUSIONS

In general, the total static bearing capacity of the vibratory installed 914-mm diameter, 15.875-mm walled, open-end, spiral welded pipe piles was observed to be approximately 50 percent of a sister pile installed by impact driving. Additionally, the loss of bearing capacity attributed to shaft resistance was observed to vary between the individual soil strata. The comparison of signal matching results from restrike testing of sister piles initially installed with the vibratory hammer and impact driven is shown in Table 3.

Table 3. Static resistances of impact and vibratory installed sister piles.

	Pile No.		66A	66A	111A	111A
	Installation Method[1]		IMP	VIB	IMP	VIB
	Installation Date		Jan-4-12	Feb-8-12	Jan-9-12	Mar-23-12
	Setup Period (days)		5	26	10	69
Stratum	Depth (ft)	Depth (m)	Unit Resist. (kPa)	Unit Resist. (kPa)	Unit Resist. (kPa)	Unit Resist. (kPa)
Upper Norfolk	17	5.2	6.70	1.92	5.27	0.00
	24	7.3	21.55	3.83	12.45	4.31
	30	9.1	25.86	5.75	21.07	10.05
	37	11.3	30.16	7.18	29.69	12.93
Lower Norfolk	44	13.4	54.58	14.36	33.52	17.24
	50	15.2	54.58	16.76	40.22	18.67
	57	17.4	54.58	28.73	46.44	25.86
	64	19.5	58.89	32.56	58.89	25.86
Yorktown	70	21.3	62.24	30.16	61.29	25.86
	77	23.5	67.03	33.04	60.81	28.73
	84	25.6	67.03	33.04	58.89	32.56
	90	27.4	67.03	36.39	56.98	32.56
	97	29.6	67.03	36.39	56.98	35.91
	104	31.7	67.03	19.15	50.27	32.56
Static End Bearing Resistance (kN)			1468	685	1201	672
Total Static Capacity (kN)			5605	2713	4715	2469

1 – IMP is Impact hammer, VIB is vibratory hammer

Averaged between the two vibratory installed piles, the highly plastic clays of the Upper Norfolk Formation underwent an approximate reduction of 73 percent of the shaft resistance as compared with the two impact driven sister piles. The shaft resistance within the loose sands of the Lower Norfolk Formation were observed to be reduced by approximately 55 percent under vibratory installation; and the medium dense to dense silty, and clayey sands of the Norfolk Formation lost approximately 49 percent of the shaft resistance during vibratory installation.

The average end bearing resistance of the vibratory piles was observed to be 50 percent of the impact driven sister piles, and 50 percent of the total static capacity. Given the SPT N-values and general description of the sands in the Norfolk Formation as loose to medium dense, the lower capacity in the vibratory hammer driven piles is consistent with the lower capacities observed by O'Neill et al. (1990) in their model piles driven to medium dense sands with relative density of 65%.

ACKNOWLEDGMENTS

The authors would like to express their gratitude to McLean Contracting Company who made this data possible and available for use through their dedication to the advancement and understanding of pile foundation installation methods.

REFERENCES

Boyer, N.B. (1985). "Resistance to technological innovation: the history of the pile driver through the 18th century". Technology and Culture Vol. 26, No. 1.

Viking, K. (2005). "The vibratory pile installation technique, part I". PDCA Pile Driver Magazine, Winter Issue.

Google.com,(2013). https://maps.google.com/maps?q=richmond,+va&hl=en&ll =36.807086,-76.294212&spn=0.216059,0.482712&sll=36.849543,-76.298161&sspn=0.107695,0.085316&hnear=Richmond,+Virginia&t=m&z=12

Barker, W.J. & Bjorken, E.D. (1978). "Geology of the Norfolk South quadrangle, Virginia: Virginia Department of Mineral Resources Publication 9". Text and 1:24,000 scale map.

Martin, R.E., James, J.S., Powell, G.W. & Bertoulin, M. (1987). "Concrete pile design in Tidewater Virginia". ASCE Journal of Geotechnical Engineering, Vol. 113.

United States Army Corps of Engineers (2011). "Remedial design phase 1C – offshore sheet pile containment wall Atlantic Wood Industries Superfund Site". Unrestricted Construction Solicitation and Specifications W91236-11-R-0020 Volume 2 of 2.

Rausche, F. (2002). "Modeling of vibratory pile driving". Proceedings of the International Conference on Vibratory Pile Driving and Deep Soil Compaction: Louvain-La Neuve, Belgium.

O'Neill, M.W., Vipulanandan, C., Wong, D. (1990). "Laboratory modeling of vibro-driven piles"**,** *Journal of geotechnical engineering*, v 116, n 8, p 1190-1209, Aug 1990.

Mosher, R. (1987), "Comparison of axial capacity of vibratory-driven piles to impact-driven piles." Final Report, Department of the Army.

Pile Dynamics, Inc. (2006). "CAPWAP® Background Report" Pile Dynamics, Inc. Cleveland, Ohio.