Use of Dynamic Measurements to Drive High Capacity Concrete Piles in Washington's Potomac Formation

John E. Regan, P.E.[1] and Karl A. Higgins, III, P.E.[2]

[1]Associate Principal, GZA GeoEnvironmental, Inc, One Edgewater Drive, Norwood, Massachusetts 02062: Tel: 781-278-5741: john.regan@gza.com

[2]Principal, ECS Mid-Atlantic, LLC, 14026 Thunderbolt Place, Suite 100, Chantilly, Virginia 20151: Tel: 703-834-2325: khiggins@ecslimited.com

ABSTRACT: Located along the eastern shoreline of the Potomac River, the Gaylord National Harbor Hotel and Convention Center is a massive development project supported on deep foundations. Deep foundations were designed to derived support within the underlying Potomac Formation, a highly variable mixture of sand, gravel, and clay. While other traditional deep foundation options typically used in the Washington DC region were considered, prestressed, square concrete piles driven to a 1335 kN (150 ton) design capacity were deemed most feasible.

Square (355 mm) prestressed concrete piles were dynamically load tested and monitored during impact driving using the Pile Driving Analyzer (PDA). Dynamic soil forces resisting pile penetration within the Potomac Formation exhibited significant elastic behavior at times, resulting in large pile rebound and substantial energy loss in the hammer-pile-soil system. Large soil quakes were observed during penetration within the bearing soils resulting in significant pile breakage. This paper discusses some of the challenges faced with installing prestressed concrete piles in the Potomac Formation and presents a case history of observations, dynamic test results and remedies employed to safely achieve the required design loads using a single-acting diesel hammer.

INTRODUCTION

The Gaylord National Harbor Hotel and Convention Center is just one of a growing number of projects in the Washington DC Metro region founded on high capacity prestressed concrete piles. Located along the Potomac River, just east of the Woodrow Wilson Bridge connecting Maryland and Virginia, this massive project was constructed in two phases. The first phase extends inland from the eastern shoreline of the Potomac and is founded on square (355 mm) prestressed concrete piles which achieve suitable bearing above the Potomac Formation. The Phase II portion of this project lies in an area described on geologic maps and geotechnical reports as an erratic transition zone between the overlying Terrace River Deposits and the deeper Potomac Formation. Phase II was initially designed to be supported on drilled shafts bearing within the underlying Potomac Formation, a highly variable mixture of sand, gravel, and clay. Driven, prestressed concrete piles were proposed as an alternate to more traditional deep foundation elements (i.e. drilled shafts) typically employed in the Washington DC region.

The site is located along the eastern shore of the Potomac River in the Coastal Plain Physiographic Province, known as Selby Bay, in Prince George's County, Maryland. The upper soil beneath a relatively thin layer of fill material consists of "Terrace River Deposits", alluvial/fluvial soils consisting predominately of sands with interbedded layers of low to high plasticity clay soils. Marine deposits of the Potomac Formation, consisting of very dense sands and highly overconsolidated clays, underlay the Alluvial Deposits. Groundwater was encountered at depths ranging from 12 to 20 meters below existing grades, ranging in elevation from el. + 6 m to el.+ 1.2 m, and may represent perched water within the Alluvial soils, above the Potomac Formation.

The Potomac Formation, generally consisting of fine sands, silt and clay, is the oldest sedimentary deposit in the Washington metro area and represents the bearing stratum for many shallow and deep foundations. The silts and clays of the Potomac Formation are often referred to as "marine clays", and typically have high plasticity characteristics and significant shrink-swell potential and are highly overconsolidated. With bedrock typically encountered at significant depth, usually greater than 60 meters below ground surface, the Potomac Formation represents the bearing stratum for many shallow and deep foundations in the Washington DC Metropolitan area.

FOUNDATION DESIGN CONSIDERATIONS

Preliminary geotechnical findings suggested that 1335 kN (150 ton) auger cast-in-place (CIP) piles and 1335 kN (150 ton) drilled shafts represented that most cost effective foundation options available in the region to support the required building loads. While driven 355mm square prestressed concrete piles were recognized as an alternative, design capacities for driven piles commonly used within the Washington DC region are generally limited to 800-900 kN (80-90 tons) per pile, thereby rending their use more expensive than drilled foundation systems.

Prestressed concrete pile capacities have traditionally been limited in the Washington region to 800-900 kN (80-90 tons). While substantially higher design loads (i.e. 1335 kN) are common throughout many regions in the US, the variability of Washington's Potomac Formation presents challenges to ensure safe and economical installation of these large displacement piles. By executing a comprehensive dynamic test pile program to confirm that individual pile capacities in excess of 1335 kN (150 tons) were possible, the project schedule and cost constraints supported a driven pile foundation system over more traditional drilled cast-in-place piles or drilled shafts.

Square (355mm) prestressed concrete piles having a minimum 28-day compressive strength of 41.4 MPa (6,000 psi) and a prestress of 4.8 MPa (700 psi) were installed using a Delmag D46-32 single acting diesel hammer to an ultimate pile capacity of 2670 kN (300 tons). The D46 has a ram weight of 45.1 kN (10,143 lbs), a maximum stroke of 3.7 meters (12 feet), yielding a maximum rated energy of 165 kN-m (122,000 ft-lbs). The hammer cushioning material consisted of 5.1 cm (2 inches) of aluminum and conbest and the pile cushioning material consisted of 23.5 cm (9.25 inches) of plywood. Driven pile lengths ranged between 23 meters and 33.5 meters long. Maximum compressive and tensile stresses for this pile type are 29.6 MPa (4.3 ksi) and 7.1 MPa (1.1 ksi), respectively (per AASHTO Standard Specification for Highway Bridges).

Preliminary Driving Criteria – Phase I

Preliminary driving criteria were developed for the test pile program utilizing a combination of static pile capacity analyses (i.e. Meyerhof's formulae), along with pile-hammer-soil models developed using GRLWEAP, a one-dimensional wave equation program. Target pile lengths were developed based on the site geology and calculated end bearing and skin friction components.

The initial Test Pile Program performed for Phase I consisted of dynamic load testing using the Pile Driving Analyzer (PDA) to measure pile stresses, pile-soil displacement (i.e. quake), hammer performance and to estimate pile capacity. The PDA was used to make dynamic force and acceleration measurements of the test piles. These measurements were evaluated in the field to determine pile capacity, pile stress and hammer performance. Dynamic force and acceleration measurements obtained from the PDA during initial drive and restrike, and CAPWAP analyses were evaluated to develop a minimum embedment depth into the underlying bearing stratum. Target pile lengths varied by approximately 12 meters across the Phase I portion of the site.

Pile driving results during Phase I confirmed that the pile design load of 1335 kN (150 tons) could be achieved with reasonable driving effort using the Delmag D-

46 operating at a 2 meter (6.5 foot) stroke. Using the Delmag D-46 with plywood cushioning, the Phase I piles were installed without pre-augering. The production driving criteria was established such that pile installation would be terminated above the minimum embedment length provided that a minimum penetration resistance of 8 blows per 25.4 mm (8 blows per inch) for 305 mm (1 foot) was achieved using a 2 meter (6.5 foot) stroke. Restrike testing subsequently confirmed that significant pile set-up was achieved over time. Many of the production piles, therefore, were simply driven to the target tip elevation, developing the required capacity principally in the Terrace River deposits (i.e. Fluvium or Alluvium).

PHASE II DEVELOPMENT - *Geologic Transition*

The Phase II portion of the Site was planned in an area where erratic and abrupt changes in the underlying geology were identified by soil explorations. While portions of the site are underlain by relatively thick Alluvium and ultimately, at deeper depths, by the Potomac Formation, the eastern portions of the site, in the vicinity of the Phase II building addition, are underlain by thinner amounts of upper alluvium, and the Potomac Formation is encountered at shallower, more erratic depths. The soil stratigraphy resembles a wedge that grows thicker toward the west and the Potomac River; the Potomac Formation soils can be found at the ground surface at points just east of Phase II. This abrupt change in site geology presented challenges with respect to driven pile installation. Due to these known geologic conditions, the Phase II dynamic test program considered a higher density of dynamic test piles to determine pile lengths.

Dynamic Test Pile Program

Seventeen test piles were dynamically load tested and monitored using the PDA during initial drive and restrike. In addition, subsequent CAPWAP analyses were performed on selected blows. Each test pile location was pre-augered 15 to 17 meters (50-55 feet) below existing grade using a 355 mm (14 inch) diameter drill stem. Pre-augering was performed to improve the energy transfer from the pile top to the pile tip and improve pile embedments into the varying underlying soil conditions.

Three test areas were defined to coincide with changes in pile behavior. While pile penetration resistances required to achieve the pre-determined target tip elevation generally varied from 2 to 11 blows per 25.4 mm (28-130 blows per foot), difficult driving was concentrated in the northeast corner of Phase II. Ultimately, five of 17 test piles were broken during driving. The breakage was abrupt, and little warning was observed with regard to stresses measured at the pile heads. After further examination of the PDA data with CAPWAP analyses, it was determined that the piles developed tensile cracks and broke in the lower 1/3 to ½ of the pile length.

CAPWAP analyses performed on data recorded at the end of initial drive indicated pile capacities ranging from 1780 kN (400 kips) to 2800 kN (630 kips) with skin friction components ranging from 180 kN (10 %) in the southern most area to

1290 kN (70%) in the northeast area of the Site. CAPWAP analyses performed on data recorded at the beginning of restrike, typically following a 3 to 6 day set-up period, indicated a "Case Method" ultimate pile capacity of 2625 kN (590 kips) to 4400 kN (990 kips) with skin friction components ranging from 735 kips (25% percent) to 2310 kips (85%). With a few exceptions, the data obtained at the beginning of restrike testing indicated a significant increase in penetration resistance ranging from 20% to 260%.

High Quake – Pile Rebound

Dynamic soil forces resisting pile penetration within the Potomac Formation exhibited significant elastic behavior at times, resulting in large pile rebound and substantial energy loss in the hammer-pile-soil system. This elastic behavior, known as Quake, is defined as the displacement at which the initial elastic deformation of the pile and soil achieves its ultimate load and goes plastic. Studies have shown that soil quakes greater than 0.4-inches significantly alter the wave equation results (Likins, 1983).

Dynamic data from Phase II test piles indicated computed quake values during restrike between 16mm and 25mm (0.64 to 1.0 inches), with toe quake values as high as 20 mm (0.78 inches). Table 1 below provides a summary of select dynamic measurements for test piles driven within the Potomac Formation, including Dmax, the maximum pile displacement of the pile head (both elastic and plastic) under a single hammer blow. While measured compressive and tensile driving stresses were maintained within allowable limits (are 29.6 MPa (4.3 ksi) and 7.1 MPa (1.1 ksi), respectively) sudden pile breakage occurred at various tip elevations with moderate driving resistance.

Table 1 – Dynamic Measurements of Test Piles driven into the Potomac Formation prior to breakage

Test Pile	Total Embed. Length (m)	Embed. Length through Potomac (m)	Pile Tip Elev. (m)	Blow Count (blows per 25.4mm)	Measured Tensile Stress (MPa)	CAPWAP Toe Quake (cm)	Max Pile Displ. DMax (cm)
A7	19.8	19.8	+2.1	1	1.4 - 6.2	2.0	5.1
A7-R	18.3	18.3	+3.6	2	1.4 - 4.1	3.3	3.4
A7-R2	22.6	22.6	-0.6	3	2.1 - 6.9	2.7	3.4
A8	20.1	20.1	+1.2	3	1.4 – 5.5	2.0	4.7

Estimating static pile capacity requires a balance of energy between the total energy delivered by the hammer to the pile and the total work done by the resisting forces, both elastic and plastic, during penetration. The effect that large quakes have on the efficiency of the pile/hammer/soil system during impact can be significant. The occurrence of high soil quakes requires more energy to fully mobilize the pile tip from elastic to plastic than would be needed under normal quake conditions. This behavior is represented below in Figure 2 by comparing the relationship of Pile/Soil resistance to elastic and plastic (i.e. permanent) Displacement, under each hammer blow (Smith, 1960)

With equivalent energy delivered to the pile/soil system, a pile will achieve substantially more permanent toe displacement (i.e. Set) under a normal quake condition (point c) than a pile driven under high quake conditions (point c’). However, the maximum displacement of the pile top under each hammer blow, which represents the total elastic and plastic displacement of the pile top, will be substantially lower for normal quake conditions (point b) than high quake conditions (point b’). Similarly, when driving a pile in high quake conditions, substantially larger energy transfer is required to produce the same permanent set (i.e. blow count where Set = 1/ Blow count), as compared to a pile driven in normal quake conditions.

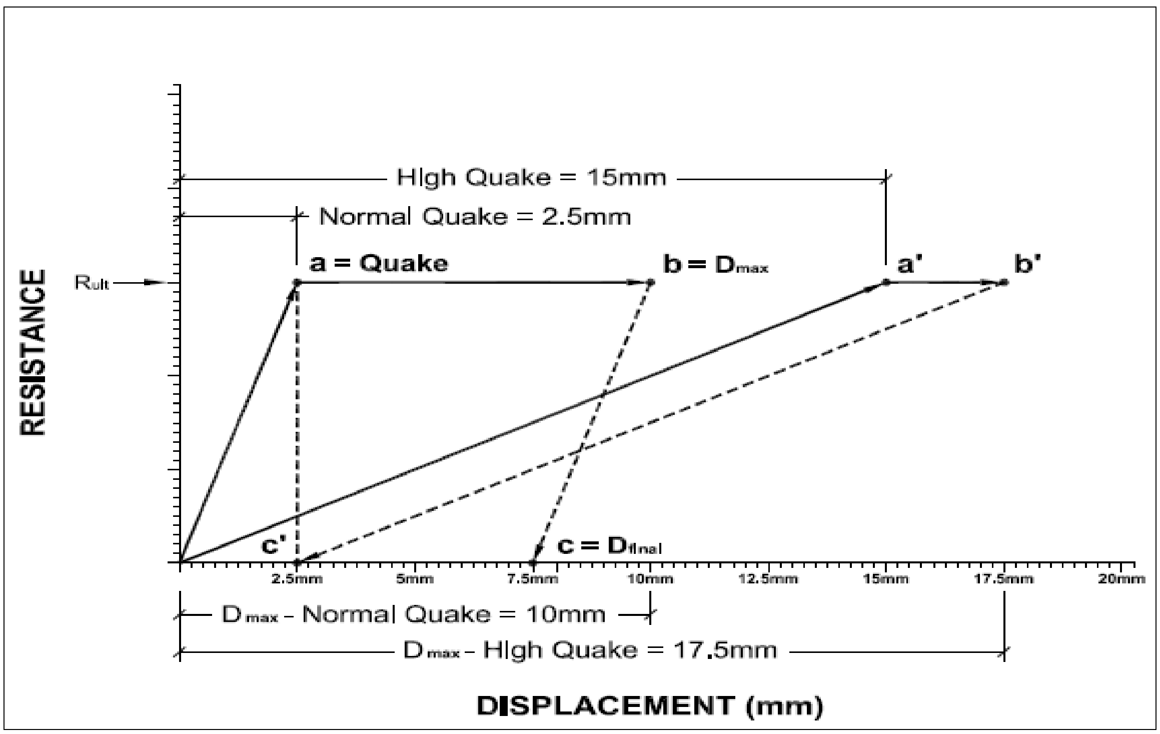

Figure 2 – Load vs. Displacement for Normal and High Quake conditions under each hammer blow (Smith, 1960).

This larger energy transfer is needed to overcome the elastic rebound of the high quake soil condition prior to fully mobilizing the full resistance at the pile tip (Likins, 1983). As such, high quake experienced in the overconsolidated Potomac Formation resulted in decreased energy transfer to the pile tip, thereby increasing the blow count required to mobilize the full pile resistance. High blow counts and corresponding high compressive stresses within the body of the piles contributed to the cyclic loading condition which decreased pile embedment lengths into the Potomac Formation and increased the occurrence of pile breakage (Hannigan, 1985).

One dimensional wave propagation theory shows that impact loads (i.e. compression forces) imparted on piles with little to no tip resistance (i.e. early driving) cause tension reflections from the pile toe. As the pile tip resistance increases, the magnitude of these tension reflections are minimized. Pile driving systems are routinely selected so that the ram weight and stroke can be controlled to minimize early driving tensile stresses. Tension reflections are further increased, however, in high quake soil conditions due to the slow response time of the tension reflection along the pile. Under normal quake conditions, the full resistance of the pile tip are mobilized at the time of the first reflection at the pile tip thereby reducing the tension reflection. In high quake soils, the full toe displacement may not be mobilized at the time of first arrival of the peak input velocity, thus higher tension reflections are generated even under refusal conditions (Likins 1983).

As shown in Table 1, recorded blow counts for four test piles driven in the geologic transition zone ranged from 1 to 3 blows per 25.4 mm (12-40 blows per foot), demonstrating sufficient pile tip resistance to maintain acceptable tension reflections from the pile tip. This was substantiated by the measured tensile stresses which ranged from 1.4 to 6.9 MPa (0.2-1.0 ksi) The driving conditions, however, changed abruptly within the Potomac Formation as the frequency of pile breakage increased substantially with very little warning. Cyclical loading imparted by the driving system and large toe quakes enpcountered in the Potomac Formation induced tension reflections causing pile breakage little typically within the lower ½ to 1/3 of the pile.

Previous studies concerning high quake soils relate pile installation difficulties to excess pore pressure build-up during cyclical loading (i.e. impact driving) in saturated soil conditions (Likins, 1983 Hussein, 2006). Interestingly, on-site observations of open cut excavations in the Phase II area, in addition to examination of soil boring data, suggest that perched groundwater is present and that the upper Potomac Formation at the Site is dessicated and very hard. The large soil quakes observed are likely related to the degree of overconsolidation of this bearing stratum, as well as the development of excess pore water pressure that may increase during driving.

REMEDIES / RESULTS

The results of the dynamic test program were instrumental in developing measures to address the issue of excessive pile breakage. To improve energy transfer to the pile tip, production pile driving procedures were modified to include pre-augering through the upper 15 meters of overburden for all production pile locations prior to installation. In addition, plywood pile cushioning material was maintained at 235 mm (9.25 inches) to minimize pile top damage. While these measures helped minimize pile breakage, ultimately the required pile capacity of 1335 kN (150 tons) was lowered down to 890 kN (100 tons) along the easternmost one-third of the building footprint, where most of the test pile damage occurred. By lowering the capacity, the piles required less tip embedment into the Potomac Formation, and thereby the chances of pile breakage were limited. The remainder of Phase II was supported by 1335 kN (150 ton) piles consistent with Phase I.

Production testing included a careful review of driving records for damage, followed by frequent PDA tests on production piles and occasional Pile Integrity Testing (PIT). As a result of these changes, the frequency of production pile damage was low for Phase II, decreasing from approximately 10% to less than 5% for the balance of Phase II (500 piles).

CONCLUSIONS

For square (355mm x355 mm) prestressed concrete piles and other large displacement piles penetrating high rebound soils such as Washington's Potomac Formation, driving system energy dissipates through the elastic rebound of the pile tip thereby increasing the driving resistance (i.e. blow count). The driving inefficiencies experienced due to highly elastic conditions in the hard Potomac Formation induced cyclical loading conditions resulting in compounding stresses that increased the occurrence of pile breakage under hard driving conditions. The variability of the driving conditions encountered at this site relate to large energy losses experienced due to elastic compression of the pile tip (i.e. high tip quakes) in very hard (N values > 50), overconsolidated clays.

The use of the PDA to obtain real-time dynamic measurements during pile installation aided in the field evaluation of hammer performance, structural integrity, pile capacity estimates and driving stresses under each hammer blow. This valuable information guided the Project team in making on-site changes to the driving criteria and helped maintain critical project milestones. By implementing routine changes to the driving criteria, square prestressed concrete piles were successfully installed within Washington's Potomac Formation using a single-acting diesel hammer.

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