SOLVING VARIABLE BEDROCK CHALLENGES WITH DUCTILE IRON PILES

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ABSTRACT

Working in urban environments presents unique logistical challenges beyond those typically associated with traditional geotechnical design and construction. These challenges can include constrained sites with limited access and laydown area, vibration concerns for adjacent structures, and the need to complete work within limited schedules. When challenging geotechnical conditions like variable bedrock elevations are present, the need to balance performance requirements with logistical constraints become the driving factor when selecting deep foundation systems. This paper focuses on the deep foundations used for a new elevated pedestrian bridge at a major health care facility in Tampa, Florida. In addition to the technical design requirements, the owner required a deep foundation system that could offer low vibrations, rapid production, and clean installation to limit impacts on their properties and the public roadway that separated them. The paper describes the project specific geotechnical and design conditions, describes Ductile Iron Pile design, and focuses on how installation of the system addressed the unique logistical challenges. In addition, the implementation of a full-scale load testing program to assess compression, uplift, and lateral design capacities is presented. This paper represents one of the few published references for Ductile Iron Piles on projects in the United States.

Keywords: ductile iron piles, variable bedrock, load testing

INTRODUCTION

Working in urban environments presents unique logistical challenges beyond those typically associated with traditional geotechnical design and construction. These challenges can include constrained sites with limited access and laydown area, vibration concerns for adjacent structures, and the need to complete work within limited schedules. When challenging geotechnical conditions like variable bedrock elevations are present, the need to balance performance requirements with logistical constraints become the driving factor when selecting deep foundation systems.

This paper focuses on the deep foundations used for a new elevated pedestrian bridge at a major health care facility in Tampa, Florida. The bridge connects a new patient tower to an existing clinic building. The pedestrian bridge is a steel-truss structure, supported by cast-in-place concrete columns and connects the second floor of the patient tower to the second floor of the clinic building. The 375-ft long bridge crosses a wide, busy arterial street and includes13 new foundations subject to heavy compression, tension, and lateral forces. Structural loads on the new foundations were reported to be as high as 1,100 kips (4,892 kN) in compression, 150 kips (667 kN) in tension and 150 kips (667 kN) laterally, driven by up to 3,000 ft-kips (4,069 kN-m) of overturning moment.

SUBSURFACE CONDITIONS

The subsurface conditions at the site as described by Terracon, Inc. (Terracon, 2021) consist of an upper stratum of loose to medium dense poorly graded sand (SP), poorly graded sand with clay (SP-SC), and poorly graded sand with silt (SP-SM) and was encountered to depths ranging from 10 to 19 feet (3 to 5.8 m) below the ground surface. The second stratum consists of soft to very stiff plastic clay (CH) and was encountered to depths ranging from 31 to 52 feet (9.4 to 15.8 m). The upper soil strata are underlain by limestone bedrock, which sampled as calcareous clayey sand with limestone fragments and was encountered until the boring terminal depths of 70 to 80 feet (21.3 to 24.4 m). The limestone bedrock has varying intervals of weathering, with weathered zone thicknesses ranging from 5 to 20 feet (1.5 to 6.1 m). A generalized profile of the subsurface conditions can be found in Figure 1.

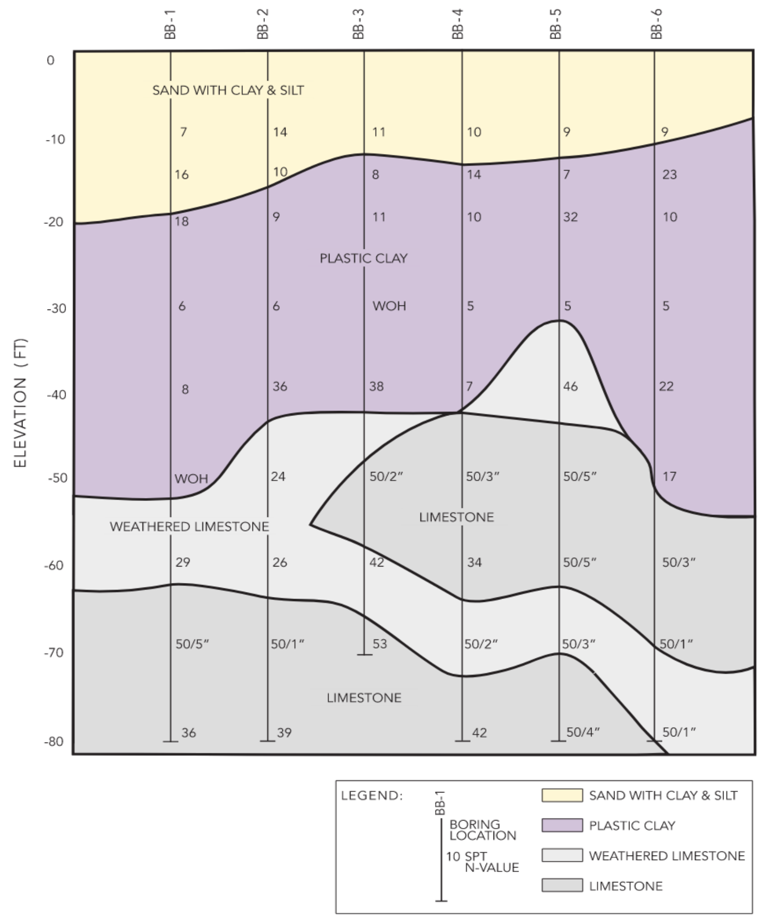

Figure 1. Generalized subsurface profile

FOUNDATION DESIGN OPTIONS

Due to the magnitude of loading associated with the new pedestrian bridge, a deep foundation system was required. In addition to the technical design requirements, it was important to the owner that the selected deep foundation system offer low vibrations, rapid production, and clean installation to limit impacts on both properties and the public roadway that separated them. Due to the variability of strength and depth of competent bedrock (45 to 65 feet) (13.7 m to 19.8 m), the depth to establish reliable pile capacities would vary widely. While cast-in-place deep foundations are normally ideal for sites with variable end-bearing layers, this option was ruled out due to concerns that high grout loss and overages could occur in the historically porous weathered limestone zones. Straight-shaft drilled piers were also dismissed due to variation in column loading conditions and the need for foundation redundancy at bridge column locations. It was believed that drilled piers would not offer an efficient design option to the project team. Pre-cast concrete piles were ruled-out due to the expected large amounts of cut-off waste and/or splicing needs that would result from the variable pile bearing elevations.

The design team decided to consider driven steel pipe piles and driven Ductile Iron Piles for deep foundation construction at the site. The geotechnical specialty contractor provided budget estimates for installing up to 95, 12.75 in (324 mm) diameter steel pipe piles and for installing up to 163, exterior-grouted 4.65 in (118 mm) diameter Ductile Iron Piles with an 8.7 in (220 mm) outer-diameter grouting shoe. The project team chose to use exterior-grouted Ductile Iron Piles due to a combination of cost and schedule benefits.

DUCTILE IRON PILE SYSTEM

For nearly a decade in North America and more than 30 years in Europe, engineers and contractors have used specially manufactured Ductile Iron Piles (DIPs) to provide a reliable and cost-effective alternative to more traditional deep foundation systems that address urban challenges. Ductile Iron Piles are a low vibration, driven pipe pile system. The modular, small-diameter piles are manufactured in standard 16.4 ft (5 m) or 18 ft (5.5 m) lengths to allow ease of installation on constrained sites. Ductile Iron Piles are manufactured using a centrifugal-casting process that transforms lamellar graphite cast iron into spheroidal graphite or ductile cast iron. The DIP material exhibits a design yield stress of 46.4 ksi (320 MPa) and modulus of elasticity of 24,600 ksi (170,000 MPa) (TRM 2014). Each pile is cast with a bell and spigot to form a Plug and Drive connection mechanism (Figure 2). The connection develops strength through a combination of the tapered compression fit as well as a friction (cold) weld that occurs during the high-frequency driving process, eliminating the need for field splicing. The piling material is delivered in bundles containing between 6 and 18 pile sections, depending on pile diameters.

Figure 2. Plug and Drive connection

The system is installed using a medium-sized excavator and a hydraulic breaker hammer (Figure 3). The breaker hammer uses a modified shank that seats into the pile bell for driving, using the hammer's percussive energy. The hammer size (energy) is matched with the size of the Ductile Iron Pile and the subgrade driving conditions to develop the most efficient installation while not overstressing the pile material. Hammer sizes typically range from 3,000 ft-lb to 8,000 ft-lb (415 kg-m to 1,110 kg-m) class hammers for the range of pile sizes. The hydraulic hammers operate at frequencies ranging from 250 to 500 cycles per minute, delivering a much higher frequency content than traditional high-impact, low frequency driven piling systems. Vibration measurements have been recorded on past project sites with different pile diameters, all at distances of less than 1.5 meters (4.9 feet) from installation. Results of the monitoring near the pile installation show that vibrations are generally less than 1 in/sec (25 mm/s) and that they rapidly decrease with distance from the pile location. (DuroTerra, 2016)

Figure 3. Typical Installation Equipment Set-up

Ductile Iron Piles have the flexibility to be installed to develop capacity in either end bearing or in friction. End-bearing DIPs generate the load-carrying capacity through bearing on very dense soil or rock after reaching the project "set" termination criteria. Installation begins with the placement of a flat driving shoe or rock point on the ground at the pile locations. A pile section is then positioned onto the shoe and driven into the ground using the vertical percussion energy. Using the same excavator, a second pile section is swung into place to make the Plug and Drive (bell and spigot) connection and then driven. This process is repeated until the pile termination criteria is reached. After driving, the pile is cut to the plan top of pile elevation and filled with neat cement grout. If higher structural capacity, or tensile resistance, is needed a center reinforcing bar can be placed into the wet grout.

Similar to micropiles in soil, friction DIPs generate compression or tension capacity by creating a grouted bond zone within a competent soil layer. (FHWA, 2005) Piles designed for frictional resistance are installed by first fitting an oversized conical grout shoe over the end of the hollow pile. Conical grout shoes range in diameter from 5.9 in (150 mm) up to 14.6 in (370 mm) depending on pile size and design requirements. The pile is then driven into the ground using the same excavator and hammer along with a grouting drive shank to continuously pump cement grout through the interior of the pile. The grout exits through the grout ports in the conical shoe (Figure 4), filling the annular space on the pile exterior during driving. The pile shaft becomes encapsulated in grout, developing a bond zone. Additional pile sections are driven until the pile extends to the design length within the bond zone. A center reinforcing bar can be inserted if required for tension or additional compression capacity.

Figure 4. Conical shoe with grouting ports

A unique feature of driven Ductile Iron Piles is the system's adaptability for sites with variable bedrock. When piles are driven to refusal or the established "set" criteria, some length of pile will remain above the pile cut-off elevation. This is particularly common at sites with variable bedrock elevations. The remaining free section of pile can be cut off with a chop saw and saved. An end-bearing cap or conical grouting shoe can then be placed on the bottom of the cut-section of pile which is then used as a starter section and driven to start the next pile location. Since the top of a cut pile section will always have a bell, a new pile section can easily be added to continue pile driving operations. For sites with variable bedrock, Ductile Iron Piles greatly reduce material waste and lead to the efficient use of pile materials.

Quality control/quality assurance monitoring is largely focused on pile alignment/verticality and recording penetration rates (amount of time to drive incremental lengths as marked on the pile section) during driving. The rates of penetration can be used as indicators of geotechnical resistance and pile capacity. For end-bearing piles, the pile is advanced until a particular rate of advancement or "set" is achieved. "Set" criteria, defined as the pile deflection measured over a time interval (typically 10 to 50 seconds), is a commonly used criterion that is either established or verified by full-scale load tests. (TRM, 2014) This criterion has developed through decades of experience and testing of the system and requires compatibility between hammer size (energy) and pile/cap dimensions for driving. For friction piles, the penetration rates can be used to identify the depth which corresponds to the start of the bond length. The pile is then advanced a particular distance into the competent layer based on the penetration rates.

DEEP FOUNDATION DESIGN FOR THE PEDESTRIAN BRIDGE

Design of the bridge involved supporting a 170 ft (51.8 m) curvilinear main span which, at the farthest reach, would be cantilevered 24 feet (7.3 m) beyond the plane of the supporting piers, and 20 feet (6.1 m) above McKinley Drive. This required foundation elements to resist significant overturning moments and permanent net tension loads at several locations.

Coordination between the structural, geotechnical, and specialty pile design teams led to a foundation plan which incorporated two 35 x 22 x 5 ft (10.7 x 6.7 x 1.5 m) concrete pile caps, with 35 piles each, for the main span piers, and 11 other pile caps with between 5 and 20 piles each, for a total of 184 piles. All piles were designed for a 75 kip (334 kN) allowable compression load and a 7.7 kip (34 kN) allowable lateral load. Additionally, 80 pile locations were identified as also requiring 30 kips (133 kN) of uplift capacity.

One of the initial challenges of pile design was estimating tip depths, both required and achievable, in the highly variable limestone across the site. While it was estimated that most piles would refuse between 45 – 65 ft (13.7 – 19.8 m), the wide of range of depths and degree of weathering meant that there was the possibility of either very early refusal, which would limit the amount of skin friction available for tension piles, or very deep refusal, which would lead to cost and schedule impacts to the project. Since the tension piles, which accounted for almost half of production, would require external grout to achieve skin friction, it was determined that there would be economy to gain by designing all piles to be externally grouted. This mitigated the risk posed by deep refusals, as the piles could be terminated at a prescribed maximum depth which would provide the full compression capacity in friction alone. This also simplified construction and QC by standardizing the driving process for all piles.

Assuming a final set of 1-in/10s and a grout-to-ground bond strength of 5 psi (35 kPa), or approximately 1.6 klf (23 kN/m), for the entire length of pile below 15-ft (4.6 m), it was estimated that an average tip depth of 55-ft (16.8 m) would satisfy all performance requirements. Using an Atlas Copco MB1700 hydraulic breaker, the set criteria of 1-in/10s was somewhat aggressive compared to the general recommendation of 1-in/50s for the size pile being used, however it was justified by the design team's experience driving piles in the local geology, as well as by the use of external grouting. Load testing would be imperative in verifying this assumption, as well as assessing actual grout-to-ground bond. The results obtained from installation and testing of sacrificial test piles prior to production would be used to determine final geotechnical design and installation criteria.

Structural design of piles was performed using International Building Code (IBC) allowable stresses and manufacturer supplied material properties. Utilizing a 4,000 psi (28 MPa) sanded grout, with an average diameter of 8.7 in (221 mm), provides an allowable compression strength 168 kips (747 kN) for an unreinforced pile. However, in uplift piles, a full-length 1-in (25.4 mm) diameter high-strength threaded bar was required to transfer tensile loads across the bell and spigot spliced joints. The threaded bar would also extend into the pile cap and terminate with a 4 in x 4 in (102 mm x 102 mm) plate at the cap upper reinforcement layer to serve as anchorage. Reinforcement for compression piles consisted of only a 10-ft (3 m) hooked #4 bar which served to hold a compression bearing plate in place at the pile head. Details of the pile design are provided in Figure 5.

Both single pile and group lateral analyses were performed using LPILE and DeepFND, respectively. The piles were designed to be embedded just 6-in (152 mm) into the pile cap, below the bottom reinforcement layer, which would generally classify the degree of fixity at the pile cap as a free-head condition. However, as is best practice, the piles were analyzed assuming both free and fixed-head cases and were determined to be acceptable under conditions of up to 75% fixity with a combined axial-bending interaction less than one.

Lateral capacity was governed by the internal bending resistance of the pile and not by displacement. Usually, this means that care must be taken to ensure a splice does not occur within the high moment region below the pile cap. This would be a challenge at the subject site though, as the variable subsurface conditions meant that exact splice locations may be nearly impossible to control. In Ductile Iron Piles however, previously published testing has shown that the bending resistance in the joint created by the bell-and-spigot connection is greater than that of the straight section (DuroTerra, 2022), thus splice location was not a concern for the design.

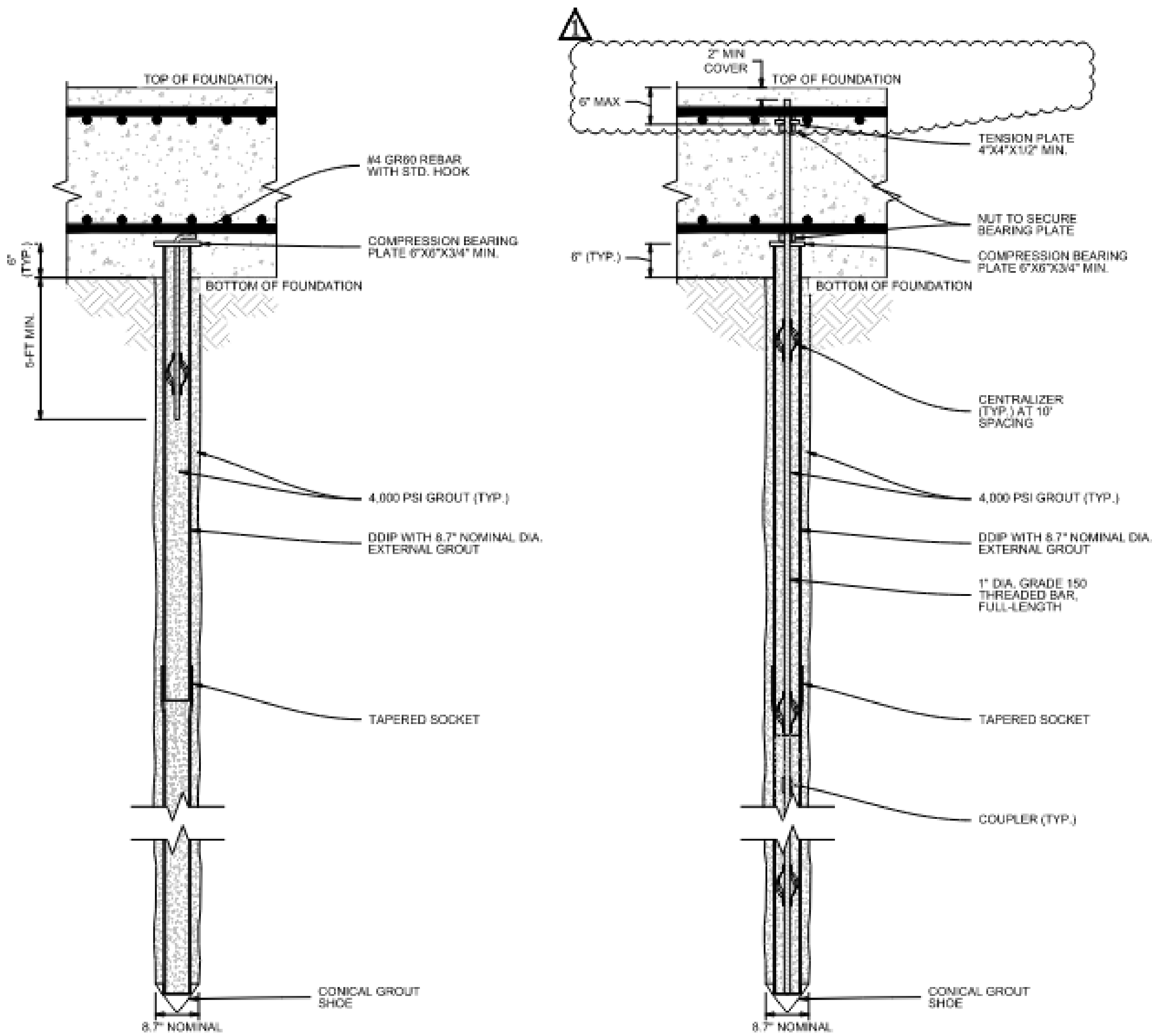

Figure 5. Compression pile (left) and tension pile (right) details

LOAD TESTING

To verify design assumptions and installation methods, a comprehensive test program consisting of compressive (ASTM D1143), tensile (ASTM D3689), and lateral (ASTM D3966) static load testing was implemented. The test program utilized five sacrificial test piles positioned as shown in Fig. 6, wherein the four corner piles would first serve as the subjects for tension and lateral testing, then serve as reaction anchors for compression testing of the center pile. Furthermore, following compression testing, the reaction piles could be used for additional tensile testing to higher loads or to failure.

The concerns associated with variable driving conditions were realized immediately upon installation of test and reaction piles. Several zones were encountered in which the pile would advance on the order of feet, rather than inches, in a single blow, or would advance with only a slight crowd force from the excavator. Piles TP-4 and TP-5 were driven to depths of over 70 ft (21.3 m) and 100 ft (30.5 m), respectively, before achieving the target set, and neither piles TP-1 nor TP-5 produced grout return to surface after exceeding a depth of about 40 ft (12.2 m). Conversely, Pile TP-2 just a few feet away, easily achieved set at less than 50 ft (15.2 m) depth and maintained grout to surface throughout installation.

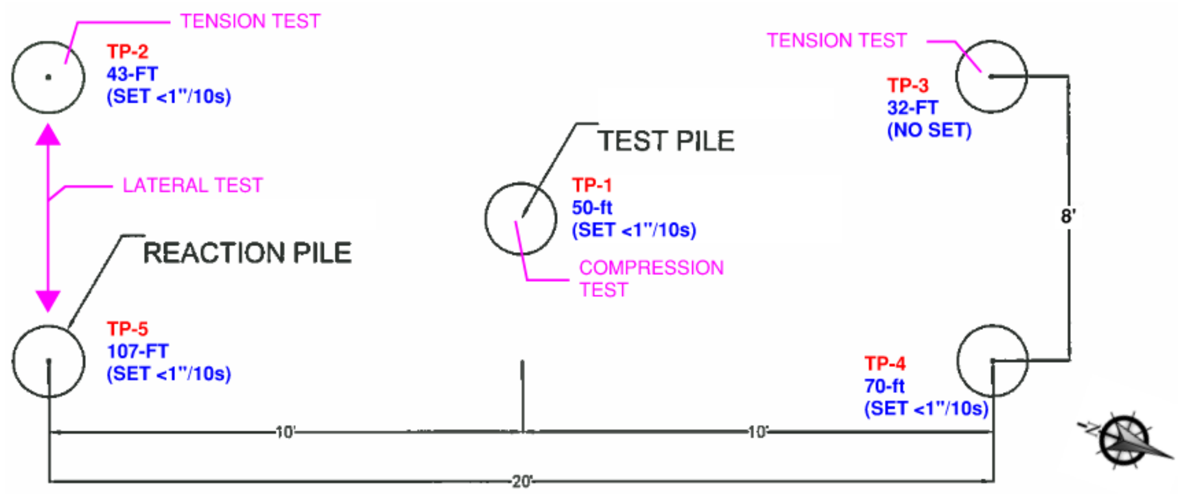

Figure 6. Load test program

A summary of test pile installation and load test results is presented in Table 1. The load-displacement data from all axial testing (both compression and tension) and lateral testing are provided in Figs. 7 and 8, respectively. Compression and tension tests were able to achieve over 4X the respective design loads and were only limited by the safe working capacities of the test equipment. Lateral testing showed that, at ground surface, the piles were able to achieve 2X design load in a free-head condition, with just under 1-in (25.4 mm) displacement as required per IBC standards.

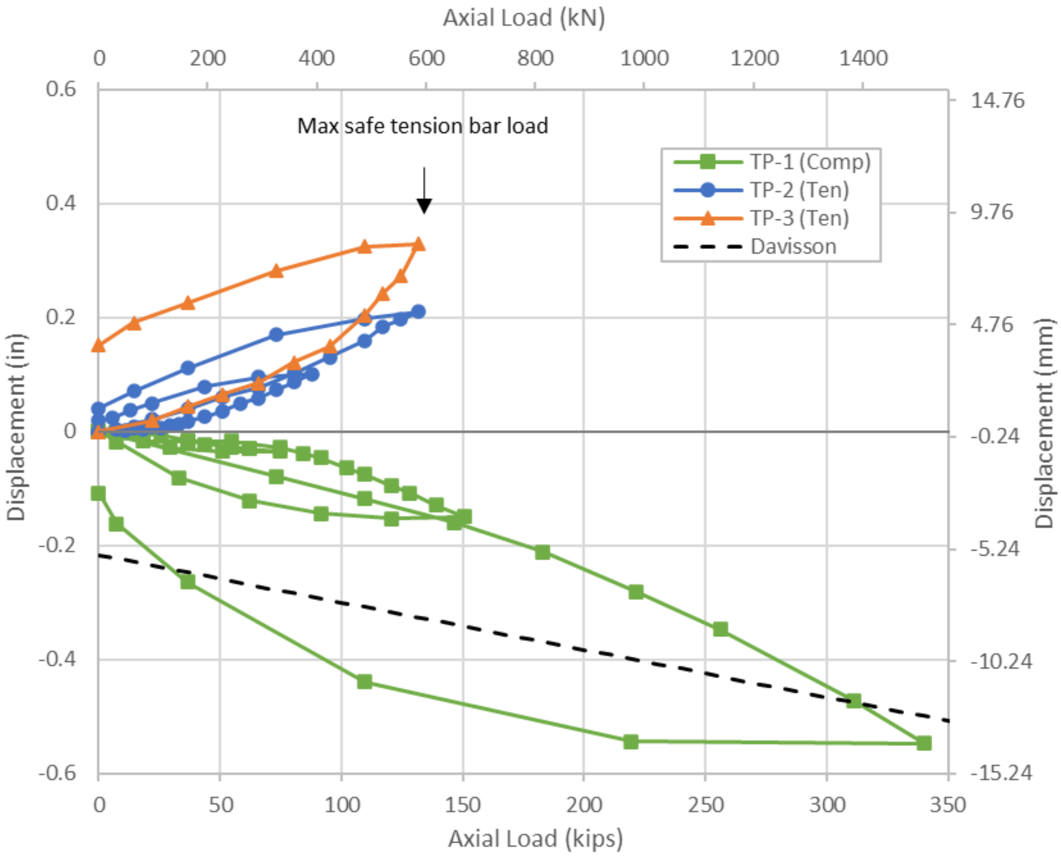

Figure 7. Compression (negative) and tension (positive) displacement vs. load

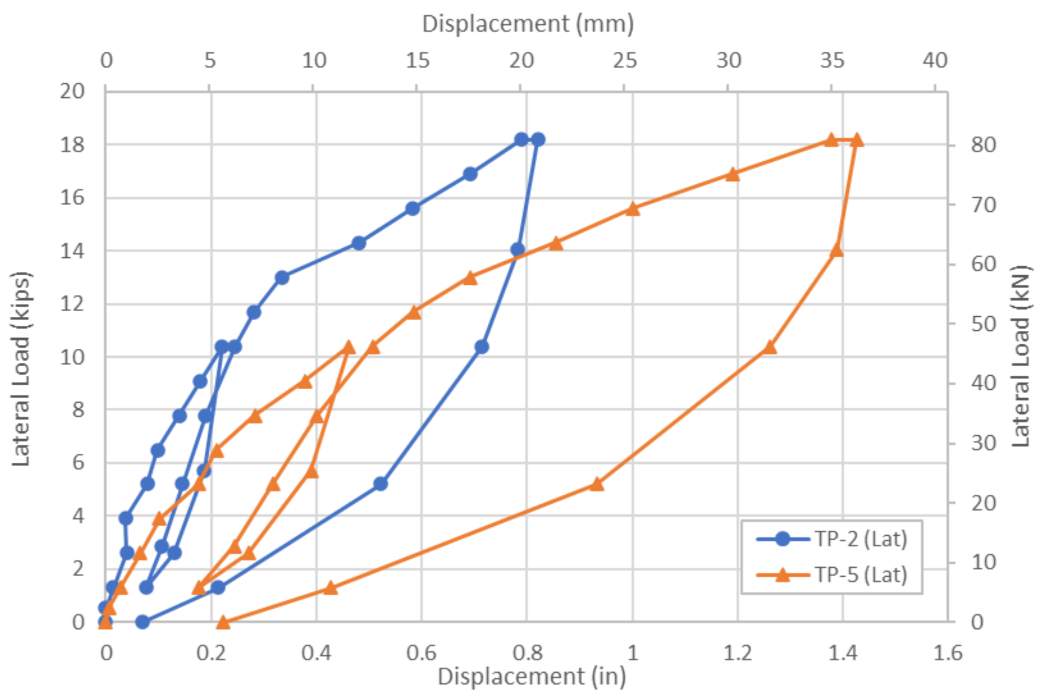

Figure 8. Lateral displacement vs. load

Table 1. Summary of load test results.

Test Pile	**TP-1**	**TP-2**	**TP-3**	**TP-4**	**TP-5**
Length, ft (m)	50 (15.2)	44 (13.4)	34 (10.4)	72 (22.0)	107 (32.6)
Target Set Achieved	Y	Y	N	Y	Y
Grout Volume, CY (m^3)	2.1 (1.61)	0.64 (0.49)	0.45 (0.34)	0.92 (0.70)	1.7 (1.30)
Grout Return	N	Y	Y	Y	N
Load Test	Comp	Ten, Lat	Ten	-	Lat
Compression					
Realized Ult. Compression Load, k (kN)	340 (1512)	-	-	-	-
Displacement at Ultimate, in (mm)	0.15 (3.8)	-	-	-	-
Failure	N	-	-	-	-
Realized Factor of Safety	4.5	-	-	-	-
Tension					
Realized Ult. Tension Load, k (kN)	-	130 (578)	130 (578)	-	-
Displacement at Ultimate, in (mm)	-	0.05 (1.3)	0.08 (2.0)	-	-
Failure	-	N	N	-	-
Realized Factor of Safety	-	4.3	4.3	-	-
Realized Unit Friction, klf (kN/m)	-	3.0 (43.1)	3.8 (55.8)	-	-
Lateral					
Realized Ult. Lateral Load, k (kN)	-	18.2 (81)	-	-	15.6 (69)
Displacement at Ultimate, in (mm)	-	0.58 (14.7)	-	-	0.99 (25)
Failure	-	N	-	-	N
Realized Factor of Safety	-	2.4	-	-	2.0

CONSTRUCTION

As with many construction projects, especially those in urban areas, engineering had to be carefully coordinated with site and schedule constraints for successful execution of the design. Vibration concerns, restricted headroom areas, and a firm completion deadline were all challenges that had to be overcome.

Piles driven on the west side of McKinley Drive were to be installed immediately adjacent to a fully operational hospital, meaning that vibration and noise levels typically associated with other driven pile types were not acceptable. Driven Ductile Iron Piles, however, produce comparatively minimal vibrations and noise, therefore all piles were able to be successfully installed with no impacts to the existing hospital throughout the entire project. Vibration and sound monitoring instrumentation setup along the edge of the building recorded maximum Peak Particle Velocities reaching 0.295 in/sec (7.49 mm/s) at 25 Hz, or just 24% of acceptable threshold values, at distances as close as 25-ft from pile driving. (Fig. 9)

The newly constructed treatment facility on the east side of McKinley Drive also posed a challenge. The upper floors created an overhang which limited the usable headroom to just 30 ft (9.1 m) at six pile locations. A common obstacle for piles installed with a crane or fixed-mast rig, often requiring design revisions, the headroom constraint was not an issue for the medium-sized excavator and the piles were easily able to be installed as designed.

The ability to drive piles using only a medium-sized excavator also had direct benefits to schedule constraints. When it came time to re-mobilize from the east side of McKinley Drive to the west side, it provided for a very quick transition, as the excavator could simply be loaded onto a lowboy trailer and driven to the other side of the highway. Not requiring complete breakdown and setup of a crane or other large rig allowed for minimum downtime and practically immediate continuation of production work.

Furthermore, while no production piles required depths of 100+ feet (30+ m) to achieve set, as in the case of Test Pile TP-5, the variable bedrock did result in tip depths ranging anywhere from 32 ft (9.8 m) to 70 ft (21.3 m), meaning that for any given pile the possible number and locations of required splices is unknown. With Ductile Iron Piles however, unplanned splices can be performed quickly and easily without sacrificing capacity, drivability, or excessive time. Additionally, the cut-off lengths at the end of driving each pile were used as the lead section to start the next pile, making efficient use of materials.

With production rates as high as 20 piles per day, the deep foundation scope was able to be completed within budget and schedule. Overall, the efficiencies realized by using DIP allowed for an early completion, reducing the originally anticipated duration by more than 25%.

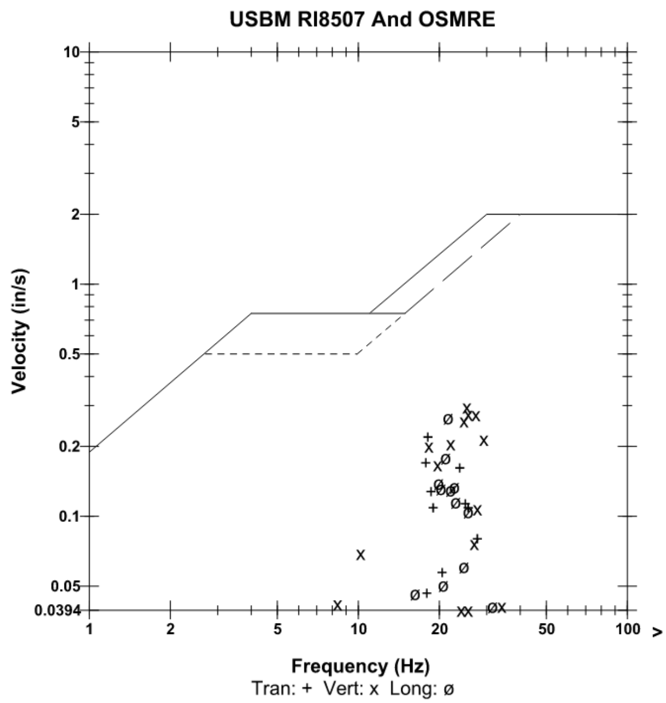

Figure 9. Recorded vibrations during pile driving.

CONCLUSION

Driven Ductile Iron Piles provided a deep foundation option that met the owner's needs for a solution that would support the heavy structural loads of the new bridge with a low-vibration piling system that would have minimal impact on the adjacent structures and roadways. The Ductile Iron Pile system was designed to accommodate complex loading conditions, and the construction methods were adaptable to variable bedrock strengths and elevations encountered at the site. Construction sequencing met the owner's need for flexibility to construct deep foundations on both sides of the busy roadway and installations were performed quickly and safely. Full scale compression, tension, and lateral pile load testing verified capacities up to and beyond design requirements, as well as helped add to the growing database of achievable pile performance with DIP in variable soil conditions.

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