

# INNOVATIVE DEEP FOUNDATION SUPPORT USING DUCTILE IRON PILES

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## ABSTRACT

Working in urban environments presents unique logistical challenges beyond those typically associated with traditional geotechnical analysis and design. These challenges often include working on sites with limited access and laydown area, vibration concerns for adjacent structures and the need to work from variable elevations or within excavations. This paper describes the use of the modular Ductile Iron Pile system, a low-vibration driven micropile, on two urban projects on the Johnson & Wales University campus in Providence, Rhode Island. The paper describes the successful implementation of Ductile Iron Piles installed to develop capacity in end-bearing on one site and friction on the second site. Design, installation and quality control operations are all presented as well as the results of full-scale load testing performed on non-production test piles. This paper is of importance to the industry because it describes a versatile option for project teams as an alternative to traditional micropiles, helical piles or other deep foundation systems for urban sites.

## INTRODUCTION

Working in urban environments presents unique logistical challenges beyond those typically associated with traditional geotechnical analysis and design. These challenges often include working on sites with limited access and laydown area, vibration concerns for adjacent structures and the need to work from variable elevations or within excavations. Two recent projects in downtown Providence, Rhode Island faced a combination of problematic soil conditions and urban construction challenges including vibration concerns. Both projects were a part of the Johnson and Wales University campus development. Figure 1 illustrates the urban locations of the sites.



Fig. 1. Project Site Locations

## DUCTILE IRON PILE SYSTEM

For over three decades, European 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 pile system. The modular, small-diameter piles are manufactured in standard 5 m (16.4 ft) lengths to allow ease of installation on constrained sites (Fig. 2). A centrifugal-casting process is used to transform the lamellar graphite cast iron into spheroidal graphite or ductile cast iron to improve the strength and produce superior impact resistance for durability during driving. The DIP material exhibits a design yield stress of 46 ksi and modulus of elasticity of 24,600 ksi (TRM 2015). Each pile is cast with a bell and spigot to form a Plug-and-Drive connection mechanism. 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, thereby eliminating the need for field splicing. The piling material is delivered in compact bundles containing between 8 and 15 pile sections depending on pile diameters. The bundles can be easily transported to congested urban sites and offloaded into limited laydown areas as shown in Figure 3.

### ***Installation***

The system is installed using a medium-sized excavator and a hydraulic breaker hammer. 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 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 class hammers for the range of pile sizes. The hydraulic hammers operate

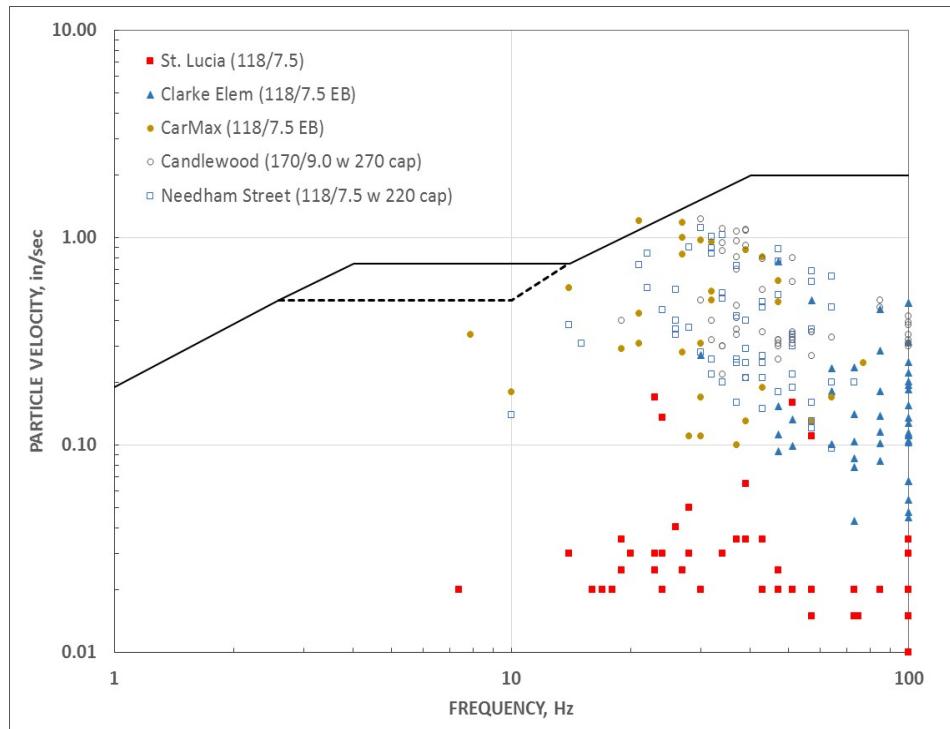


**Fig. 2. Ductile Iron Pile Installation**



**Fig. 3. Ductile Iron Pile Material Stacked On-site**

at frequencies ranging from 250 to 500 cycles per minute, thereby delivering a much higher frequency content than traditional high-impact, low frequency driven piling systems. Vibration measurements recorded on various project sites with different pile diameters, all at distances of less than 1.5 meters (4.9 feet) from installation, are shown in Figure 4. The data has been plotted along with commonly used US Bureau of Mines criteria for limiting vibration levels to avoid damage to residential structures (Siskind et al. 1980). Results of the monitoring in close proximity to the pile installation illustrate that the majority of vibrations are less than 25 mm/second (1 in/s) and nearly all of the readings are below US Bureau of Mines criteria. Vibration levels rapidly decrease with distance from the pile location.



**Fig. 4. Measured Vibrations in Close Proximity to Pile Installation**

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 5-m long pile section is then positioned onto the shoe and driven into the ground using the vertical percussion energy. Using the same excavator, a second 5-m long 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 design length or pile termination criteria is reached. After driving, the pile is cut to elevation and filled with neat cement grout. A center reinforcing bar can be placed into the wet grout, if required by the design.

Alternatively, similar to a micropile, friction DIPs generate compression or tension capacity by creating a grouted bond zone within a competent soil layer. Piles designed for frictional resistance are installed by first installing an oversized conical grout cap over the end of the hollow pile. Conical grout caps range in diameter from 150 mm (5.9 in) up to 370 mm (14.6 in) depending on pile size and design requirements. The pile is then driven into the ground using high-frequency energy while continuously pumping sand cement grout through the interior of the pile. The grout exits through the grout ports in the conical cap, 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 is then inserted if required for compression or tension capacity.

### ***Quality Control***

Field monitoring plays an important role during the installation process. Quality control/quality assurance monitoring is largely focused on pile alignment/verticality and recording penetration rates (length of time to drive 1 meter increments 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. For example, a “set” criteria of less than 1 inch of movement in more than 50 seconds is a commonly used criterion that is confirmed by full-scale load tests. This criterion has developed through decades of experience and testing of the system in Europe and requires compatibility between hammer size (energy) and pile/cap dimensions for driving. For friction piles, the penetration rates are used to determine 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. In the event of very dense driving conditions that preclude the installation of the full bond length, the penetration rates can be used to verify the achievement of the set criteria for compression applications. The compression pile is deemed acceptable by meeting either 1) the required bond length or 2) achieving the set criteria.

Another important part of the quality control process is the performance of a full-scale pile load test on a non-production test pile. Load tests are often performed in compression (ASTM D-1143) or tension (ASTM D-3689) depending on the project requirements. Load testing is typically performed to 200 percent of the design load. Since friction piles rely on a bond zone of grout to soil for resistance to either tension or compression loads, it is typically more economical to test all piles in tension to confirm the grout to soil bond strength. A center bar would then need to be installed in the test pile(s).

### ***Design - Overview***

Both the composite strength of materials (structural capacity) and the geotechnical capacity are considered in the design of Ductile Iron Pile design. . With pile diameters ranging from 98 mm (3.85 in) to 170 mm (6.69 in) and wall thicknesses ranging from 6 mm (0.24 in) to 13 mm (0.51 in), designs can be tailored to achieve maximum efficiency for soil conditions and project loads. The system can also be designed to resist tension loads by inserting a high-strength center bar into grouted piles. Working capacities typically range from 25 tons to greater than 100 tons per pile in compression and 10 to 60 tons per pile in tension.

### ***Design - Structural Compression Capacity***

Ductile Iron Piles derive their structural capacity through contributions of the ductile iron strength, interior grout strength and the addition of a center reinforcing bar (if used). The allowable pile capacity ( $P_{dip}$ ) is calculated as shown in Equation 1 for the separate contributions of strength:

$$P_{dip} = P_{ironpipe} + P_{grout} + P_{bar} \quad [1]$$

where  $P_{ironpipe}$ ,  $P_{grout}$  and  $P_{bar}$  are the allowable load resisted by the ductile iron pipe, the interior grout and the center reinforcing bar (if used), respectively. When grout and another material are to be used simultaneously to support a load in compression, the maximum strain is limited by the grout. In accordance with ACI guidelines, the maximum usable strain at the extreme concrete compression fiber is equal to 0.3 percent (2008). This is the maximum strain that the grout can experience before failure. However, the 46 ksi yield stress of the ductile iron pile is less than the typical values corresponding to the concrete stress

levels at 0.3 percent strain. As a result, the yield stress of the ductile iron is the controlling stress level in the composite capacity calculations.

The allowable load for the ductile iron material is equal to the final pile cross-sectional area ( $A_{dip-final}$ ) multiplied by the yield stress ( $F_{y-di}$ ) as shown in Equation 2.

$$P_{ironpipe} = uF_{y-di}A_{dip-final} \quad [2]$$

The final cross-sectional area of the DIP material may be reduced due to corrosion if applicable. The cross-sectional area for friction piles is equal to the manufactured area with no loss for corrosion since the interior and exterior of the pile is fully grouted. The yield stress reduction factor ( $u$ ) is based on codified values for deep foundations (IBC 2009). Typical factors used for DIP design range from 0.4 to 0.5.

The contribution of the interior grout to the load carrying capacity is calculated as the allowable concrete compressive strength ( $f'_{c-all}$ ) multiplied by the grout area (interior area within the pile) ( $A_{grout}$ ) as shown in Equation 3 where the allowable compressive strength is limited to 0.33 of the ultimate compressive strength. Further limitations of the allowable compressive strength may be appropriate based on local building codes.

$$P_{grout} = f'_{c-all}A_{grout} \quad [3]$$

The allowable load for a center reinforcing bar (if used) is equal to the bar cross-sectional area ( $A_{bar}$ ) multiplied by the yield stress as shown in Equation 4 where a yield stress reduction factor ( $u$ ) of 0.4 is often used. The yield stress selected for design is often the lower of the center reinforcing bar or the ductile iron pile, based on similar design approaches for cased micropiles (FHWA 2005).

$$P_{bar} = uF_{y-bar}A_{bar} \quad [4]$$

The sum of the individual strength components are combined to develop the total allowable load resistance of the ductile iron pile from a structural perspective. The total allowable load resistance must be greater than the applied loading demand. Structural capacity calculations including buckling checks are also used in soft soil conditions to limit design capacities (FHWA 2005).

### ***Design - Structural Tension Capacity***

The allowable structural tension capacity of the Ductile Iron Pile is specifically related to the design of the high-strength center bar installed into the grouted pile. The tensile capacity is estimated as the product of the yield stress of the reinforcing bar ( $F_{y-bar}$ ), a stress reduction multiplier ( $u$ ) of 0.60 (IBC 2009) and the cross-sectional area of the center bar ( $A_{bar}$ ) as shown in Equation 5.

$$P_{all-tension} = uF_{y-all}A_{bar} \quad [5]$$

### ***Design - Geotechnical Capacity***

The geotechnical capacity for Ductile Iron Piles is generated through either end-bearing on competent materials or friction along a grouted bond zone. For end-bearing piles, consideration of the type of bearing material and specific “set” criteria will influence the geotechnical capacity for design. Design approaches are empirical and based on significant load test experience. Results have shown DIPs driven with a hammer of suitable energy for the specific pile size and bearing on very dense soils (i.e. glacial till) or competent rock will typically achieve the required geotechnical capacity to match the allowable structural capacity with a set criteria of less than 25 mm (1 in) of movement in 50 seconds.

The geotechnical capacity for friction piles is a function of the grouted DIP diameter (related to the selected grout end cap used during installation), the estimated bond length ( $L_b$ ) and the allowable soil skin friction along the DIP ( $\alpha_{bond}$ ) as shown in Equation 6. The bond length is estimated based on the anticipated penetration of the DIP into a layer capable of providing side resistance. The allowable skin friction values for design are determined based on the bond zone soil conditions. These bond zone friction values are consistent with accepted industry standards used for Micropiles – Type B (pressure grouted) as a result of the displacement installation and pressurized grouting operation (FHWA 2005). A check of the bond between the Ductile Iron Pile and the grout may also be performed, but is not typically a controlling factor.

$$P_{friction} = \pi d_{endcap} L_b \alpha_{bond} \quad [6]$$

## GEOLOGY AND SUBSURFACE CONDITIONS

According to the 1956 Surficial Geology Map of the Providence Quadrangle, Rhode Island, the surficial soils in the vicinity of the site are comprised of artificial fill underlain by glacial outwash plains consisting of sorted sand, stratified silts and local deposits of gravel. The underlying bedrock is the Rhode Island Formation and is described as greywacke, conglomerate, sandstone, shale and meta-anthracite based on the 1959 Bedrock Geology Map of the Providence Quadrangle, Rhode Island. (GZA 2014)

The results of explorations performed at the actual project sites indicated the subsurface conditions were relatively similar at both sites. A layer of loose sand and silt “urban” fill containing construction debris (brick, concrete, asphalt, wood) was encountered up to 3.4 m (11 ft). The fill was underlain by up to 1.8 m (6 ft) of soft peat at some locations. More competent soils including medium dense to dense outwash sand and silt were then encountered and extended to 17.7 m (58 ft) followed by glacial till. Groundwater generally ranged between 4.6 m and 7 m (15 and 23 ft) below grade.

## PROJECT 1 - JOHNSON & WALES UNIVERSITY - PHYSICIAN ASSISTANT STUDIES

Johnson & Wales University’s new Center for Physician Assistant Health Sciences School (Fig. 5) was the first-of-its-kind facility in Rhode Island. The 1,672 m<sup>2</sup> (18,000 ft<sup>2</sup>) facility houses lecture halls, active learning classrooms, labs and study spaces. The structure was constructed through renovations, modernization and addition to an existing 2-story building. Design loads were up to 150 kips for columns and 35 kips/foot on exterior walls.



**Fig. 5. Photo of Center for Physician Assistant Studies**

### *Geotechnical Design Approach*

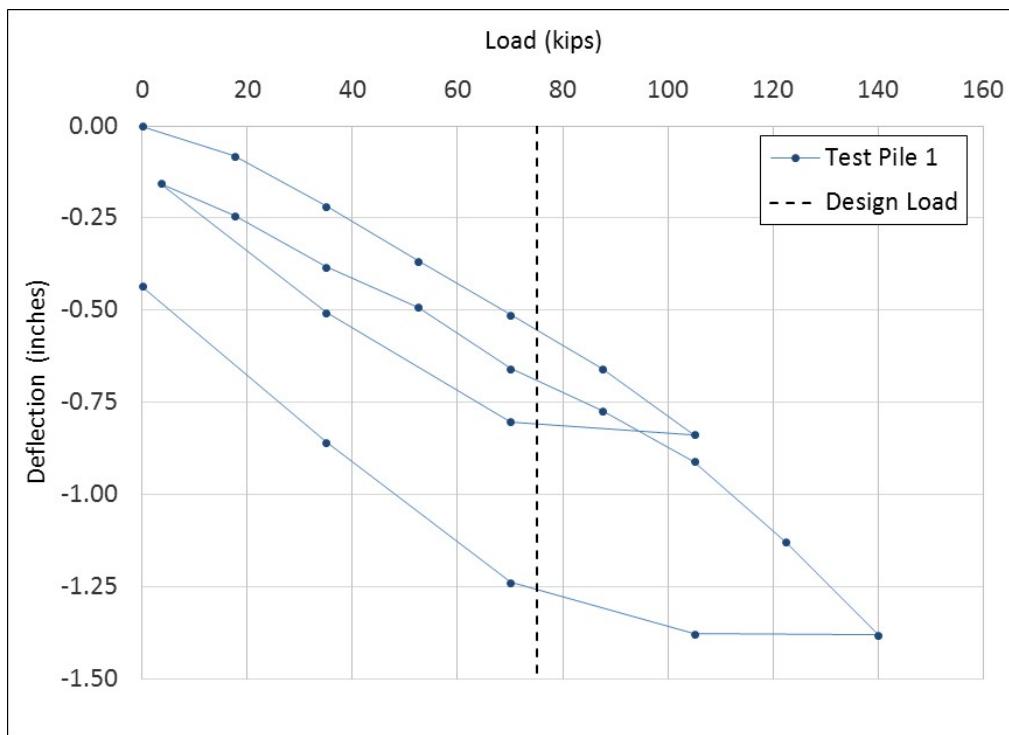
Geotechnical challenges for the project included the problematic urban fill and organic soil, but also included the practicality of construction of the new building addition while tying into the existing structure supported on shallow spread footings. Initial foundation support options under consideration were ground improvement, micropiles, compaction grout columns and helical piles which were also required for

underpinning of the existing building. Concerns with access limitations and vibrations generated from ground improvement and cost and speed of micropiles steered the project team towards a combination of compaction grouting for new construction and helical piles for underpinning of the existing building walls. The design team also considered and ultimately selected a Ductile Iron Pile solution to provide support for the new building foundations in concert with the planned helical anchors for existing building support where limited head-room restricted access with the on-site DIP installation equipment.

### ***Installation and Testing***

Ductile Iron Piles were designed for an allowable capacity of 70 kips using a 118/7.5 series pile (118 mm diameter with 7.5 mm wall thickness). The ductile iron piles were designed to develop capacity in end-bearing. The low-vibration piles were driven through the fill, peat and outwash sand/silt to terminate in the competent glacial till. Piles lengths ranged from about 20 to 27 meters (65 to 90 ft) to terminate in the sloping glacial till, a feature attributed to the nearby Providence River. The range of pile lengths was adjusted by simply adding additional pile sections. The piles were seated into the till or bedrock using a driving set criteria of less than 25 mm (1 in) of penetration per minute of driving. Prior to completion, the DIPs were then filled with grout during installation to further enhance the structural capacity and reduce the potential for corrosion on the pile interior.

Prior to installation of the production piles, a full-scale load test was performed to confirm the design capacity and performance. Results of the load test are shown in Figure 6. At the design load of 70 kips, deformation of the end-bearing DIP was limited to 12 mm (0.5 in). The deflection increased to 35 mm (1.38 in) at the 200% design load of 140 kips. A net deflection of 11.4 mm (0.45 in) was recorded upon rebound. The test results confirmed the performance of the system on the site.

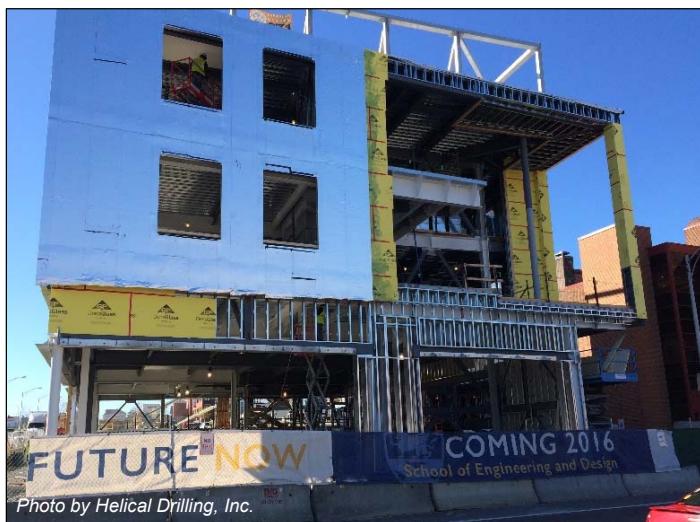


**Fig. 6. End-Bearing Ductile Iron Pile Load Test Results**

## PROJECT 2 – JOHNSON & WALES UNIVERSITY – SCHOOL OF ENGINEERING & DESIGN

Johnson & Wales University's construction of a new \$40M, 3-story School of Engineering & Design building (Fig. 7.) consisted of a 6,596 m<sup>2</sup> (71,000 ft<sup>2</sup>) academic facility including classrooms, offices and labs. The project footprint was bordered on two sides by immediately adjacent Johnson & Wales buildings, requiring a solution to support foundations without adverse impacts to the existing buildings. Column loads in these locations of the building ranged from 150 to 600 kips with up to 45 kips of uplift (tension) at select locations.

### *Geotechnical Design Approach*



**Fig. 7. Photo of School of Engineering and Design**

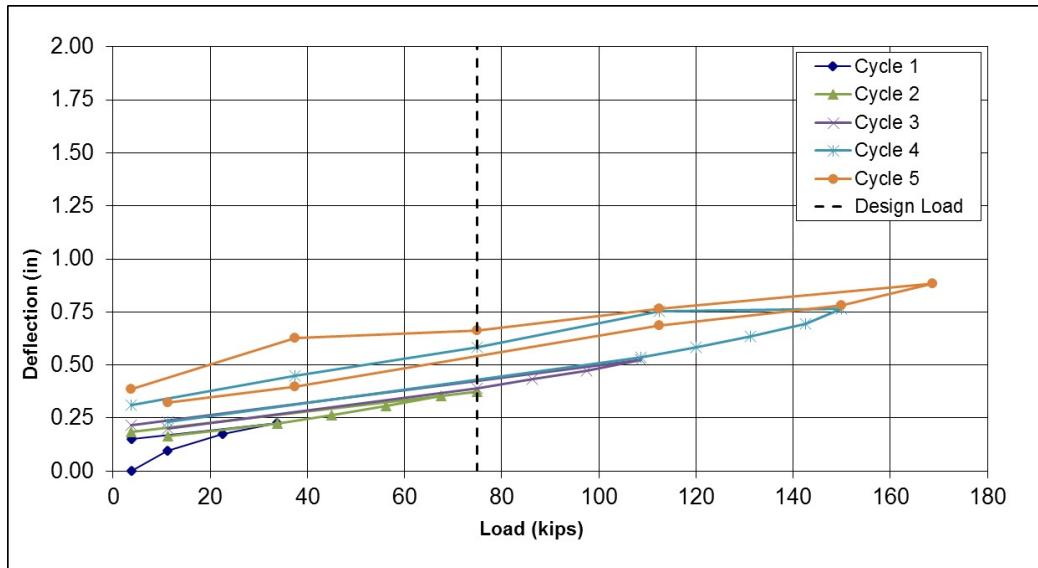
While the urban fill was present at the site, the organic deposit found at the previous project was not encountered. A cost-effective solution was developed to found the majority of the structure on improved ground with a design bearing pressures of 4,000 psf following Rapid Impact Compaction, a form of dynamic densification. However, vibrations generated during densification were expected (and confirmed) to be far too great to support building foundations immediately adjacent to the existing structures. With successful implementation of the Ductile Iron Pile system on the nearby Physician Assistant Studies building, the design team specified the system for the heavily-loaded, vibration-sensitive areas of the project to provide a cost-effective alternate to traditional solutions like drilled micropiles.

### *Installation and Testing*

The use of 37.5 ton Ductile Iron Piles consisting of a Series 118/7.5 pile (118 mm diameter pile with a 7.5 mm wall thickness) was specified as part of the design. Although the design approach on the Physician Assistant Studies building required piles be installed through the fill and sands to terminate in end-bearing on dense glacial till or rock, a more cost-effective solution was designed using a friction DIP to develop both compression and tension resistance. Piles were designed with an oversized 220 mm (8.7 in) diameter conical grout cap to develop capacity with a friction bond zone in the sand underlying the fill. Pile lengths of 7.6 m (25 ft) were designed to resist the applied working loads of 75 kips. A #9 threaded, center reinforcing bar with a yield strength of 75 ksi was installed in certain piles to provide resistance to the tension loads.

A full-scale tension load test (ASTM D3689) was performed to confirm the design capacity and performance with the 4.9 m (16 ft) bond length. Results of the test are provided in Figure 8. At the design load of 75 kips (Cycle 2), upward deflection of the friction DIP was limited to 9.7 mm (0.38 in). The test was performed up to 200% of the design load (150 kips) where the total deflection was 19.3 mm (0.76 in) (Cycle 4). Following the final unloading, the test was reloaded to evaluate performance up to 225% (Cycle 5) where movement was still limited to only 22.3 mm (0.88 in) and maintained load resistance. Net deflection after unloading the pile was limited to under 10 mm (0.4 in).

The ductile iron piles were rapidly installed to extend through the fill and terminate in the sand. A minimum pile bond length of 4.9 m (16 ft) was maintained in the sand. A total of 66 piles were installed in 3 working days. Vibration levels were observed to be so minimal during installation that monitoring was discontinued.



**Fig. 8. Friction Ductile Iron Pile Tension Load Test Results**

## CONCLUSIONS

Ductile Iron Piles were successfully installed at two urban project locations in the Northeast United States where the benefits of the low-vibration, modular driven micropile system were realized. Selection of the system was made based on cost-effectiveness, low vibrations, rapid installation rate and the ability to work at constrained sites. The versatile pile system was designed to develop capacity in end-bearing on one site and through friction resistance for both compression and tension loads on the other site. Design approaches described herein and modeled after other traditional piles (driven piles, micropiles) provided an acceptable method for predicting capacity and performance. Load testing performed at both sites confirmed the design approach and anticipated capacities.

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