AN ALTERNATIVE PILE TECHNOLOGY

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Ductile Iron Piles (DIPs) are a proven, viable alternative to conventional piles, and for the past five years have been successfully used throughout New England. The installation process of DIPs lends itself to both large and small projects. Because of their installation efficiency they provide a substantial cost savings over drilled micro piles. Several New England projects (both end bearing and friction) will be evaluated, focusing on the installation and design procedures.

Background

The driven ductile cast iron pile (DIP) is a simple, fast and highly effective pile system. Over the last 30 years over 10 million feet of this pile system has been installed worldwide predominantly in Europe. DIPs are a pre-fabricated driven pile system utilizing high strength ductile iron pipes which are manufactured using a spun-cast process having outside diameters of 4 5/8" and 6 5/8 inches. The standard pile lengths are 16.4 feet long. The pipes are manufactured with a tapered socket with an internal shoulder for full engagement at the top and a tapered spigot at the bottom. The individual pile sections can be connected with this Plug and Drive® connection to drive a pile of any length.(see figure 1)

Photo 1 Driving a 170 foot long test pile in Boston

The DIP system's inherent advantage is the simplicity of the installation process. The piles are driven to refusal or to a required penetration depth with the use of a high-impact high-frequency hydraulic hammer (ie. Breaker hammer). In most applications the hydraulic hammer is mounted on an appropriately sized excavator (see Photo 2). At the completion of driving the pile, the pile is cut off (see photo 5) and the remaining section of pile is fitted with the appropriate driving shoe and serves as the lead section for the next pile. Being able to reuse the cut off section, yields minimal if any pile shaft waste. The pile is then fitted with an appropriate pile cap plate to receive the load from the superstructure.

Photo 2 DIP driving operation on a flat site

Based on job specific soil conditions, the pile can be installed as an end-bearing (non-externally grouted) pile or a friction (externally grouted) pile. Working loads in the range of 100 kips are possible for the 4 5/8 inch diameter piles and in the range of 200 kips is possible for the 6 5/8 inch piles. Structural pile capacities are designed in accordance to Chapter 18 of the 2009 IBC Code. Table 1 provides a summary of structural pile shaft capacities calculated in accordance with the IBC Code. Where corrosive soils are encountered, a loss of 1/16 inch of wall thickness is an accepted limit for most soil conditions in New England.

No Corrosion

Type	O.D. (in)	Wall Thk. (in)	Pile Capacity (kips)	
			DIP+Grout	w/ #9 Bar
118-7.5	4 ⅝	0.30	114.94	135.79
118-9.0	4 ⅝	0.35	130.91	151.76
118-10.6	4 ⅝	0.42	147.43	168.28
170-9.0	6 ⅝	0.35	210.10	230.95
170-10.6	6 ⅝	0.42	235.36	256.21

Moderate Corrosion (~1.59mm = 1/16" wall loss)

Type	O.D. (in)	Wall Thk. (in)	Pile Capacity (kips)	
			DIP+Grout	w/ #9 Bar
118-7.5	4 ⅝	0.23	94.06	114.91
118-9.0	4 ⅝	0.29	110.03	130.88
118-10.6	4 ⅝	0.35	126.56	147.41
170-9.0	6 ⅝	0.29	179.90	200.75
170-10.6	6 ⅝	0.35	205.15	226.00

Table 1 Pile shaft capacities per IBC 2009

Friction piles are externally grouted and are installed with an oversized patented conical grout shoe attached to the base of the lead pile section.(See Photo 2) As the pile is driven, the annulus created between the pile shaft and the soil is filled simultaneously with a sanded grout bonding the pile shaft to the surrounding soil. Grouted friction piles have external grout diameters of 7 ½, 9 ¾ and 11 ¾ inches. These friction piles can be used as tension elements with the addition of tension reinforcing within the pile. See photo 3 for a snap shot of a typical grouted pile driving operation.

Photo 2 Patented Conical Grout Shoe

Photo 3 Driving Grouted Ductile Iron Piles

When considering skin friction piles, SPT values should exceed 3 blows for cohesive soils and 4 blows for non-cohesive soils. Advancing piles in materials with blow counts exceeding 40 blows in cohesive soils and 60 in non-cohesive soils becomes difficult. The load carrying capacity for skin friction piles should be based on a maximum friction embedment length of 40 to 50 feet. This limitation will prevent a progressive failure developing between the pile shaft and the surrounding soil.

Due to their small diameter, ductile iron piles, like other small diameter piles, have limited capacity to transfer lateral loads. Larger lateral load dissipation can be achieved by passive soil forces on concrete foundation elements or battered piles.

Due to the relatively small diameter of DIPs, buckling of the pile shaft must be considered in soft soil conditions. In soft cohesive soil stratums in excess of 8 to 10 feet and in heterogeneous fill (cavities) with SPT values of less than 2, a buckling check is required. In cohesive soils with an un-drained shear strength of C_u<1.5 psi buckling should be investigated. In non-cohesive soils it can be assumed that lateral support provided by embedment is generally adequate.

For non-grouted DIPs, the minimum spacing between piles can be as little as 3d where "d" is the outside diameter of the pile. For grouted DIPs, the pile spacing should be $3d_c$ where "d_c" is the outside diameter of the externally grouted pile.

Materials

Cast iron, in the form of grey cast iron, has been used for commercial pipeline construction since the 1800's in Europe. Grey cast iron has high resistance to mechanical and chemical influences. However, it is a very brittle material with low impact strength. Through a refined manufacturing process the tensile strength and flexural stiffness are dramatically improved. With the addition of magnesium graphite into the grey cast iron melt, graphite flakes are transformed into spherical graphite nodules. With appropriate heat treatment the embrittlement is prevented yielding a material of increased ductility and strength. Ductile cast iron is composed of 90-95% iron in the form of scrap metal. Table 2 lists selected material properties of ductile cast iron. It should be noted that the high dynamic stresses generated by driving in some cases exceed the service level design stresses. The driving process can be thought of as a quality control measure of wall thickness and crystalline structure of the pile material. The driven depth for end-bearing piles is verified by dropping a tape measure down the pile annulus. Should damage have occurred during the driving process, this quality control measure will define the depth of the pile issue. When installing grouted piles, if the pile has been compromised by driving stresses, an increase in grouting pressures will be observed.

Ductile Cast Iron Material Properties	
Tensile Strength	44 ksi
Compressive Strength	90 ksi
Modulus of Elasticity	24,000 ksi

Table 2 Material Properties

Manufacturing Process

Ductile iron piles are manufactured using a centrifugal spin-casting process. After magnesium is added to the cast iron melt the molten iron is placed into a rotating mold and a pile shaft of nearly uniform thickness is formed. The spigot end is formed with the use of a sand mold. The still glowing pipe is removed from the centrifugal mold and directed to an annealing furnace for a slow and steady cooling to ensure that the tensile and elongation properties are maintained. The annealing process creates a very thick oxide-layer, which provides increased corrosion protection when compared to steel piles. Once the material has properly cooled the manufactured material is sampled and tested for material properties in accordance with factory quality control procedures (ISO 9001). In addition, the wall thickness of each pile is measured at 32 different locations with ultrasonic methods. The pipes are then labeled with date and time of manufacture, then bundled and stored for shipment.

Components

The ductile iron pile system is a very simple pile system. The key element of the system is the Plug and Drive® jointing system. (see Figure 1) The lead end of the 16.4 foot pile is a tapered spigot with the drive end being a tapered socket. This joint system develops the full capacity of the pile shaft in compression and offers a high degree of axial and bending stiffness. It is assumed that the joint has no allowable tension capacity. The manufactured 16.4 foot long elements can be jointed to form a pile of any length. During the driving process the high vertical driving forces cause high hoop stresses to develop in the tapered socket and friction-weld the two pile shafts. Testing of the joint has concluded that the joint system has a strength greater than the pile shaft in compression. In fact, testing revealed that if the tapered spigot was shortened by ¾ inches, the joint still maintained an axial strength higher than the base shaft.

Figure 1 Plug and Drive® jointing system

The lead pile section is fitted with an appropriate pile driving shoe. For friction piles this is a conical grout point (see photo 2) and for end bearing piles it a standard driving shoe or end plug (see figure 2). In normal unobstructed soils a flat shoe is used to form an end plug. For soils that contain obstructions that are to be penetrated a steel end plug with a rock point may be considered. The piles can also be driven with an open end. When driven in this manner, approximately 4 feet of the lead section will be filled with soil during driving.

Figure 2 Ductile Pile Components

The end plug seals the pile against water and soil infiltration into the pile. With the base sealed, the grout can be used to provide additional structural capacity and limits the corrosion exposure to the outside face of the pile.

For friction grouted piles, a patented conical grout shoe is fitted onto the lead section (see photo 2). The grout shoe is available in 7 ½, 9 ¾ and 11 ¾ inch diameters. The 7 ½ inch grout shoe can be used only with the 4 5/8 inch diameter piles. However, the 9 ¾ inch and 11 ¾ inch shoes can be used with either the 4 5/8 inch or the 6 5/8 inch shafts which simplifies the stocking of points.

Figure 3 Friction Driving Point and End Bearing Driving Adaptors

The friction pile driving process starts by laying the grout shoe at the proper location and inserting the spigot end of the pile shaft into the conical point. The pile is kept in place by a series of keeper plates within the conical grout shoe. As the pile is driven the conical grout shoe creates an annulus between the pile shaft and the surrounding soils. This annulus is immediately and consistently filled with a sand grout that is pumped down the pile shaft and exits within the space between the keeper fins in the conical points. The breaker hammer driving tool is fitted with a special grout box adapter to allow the grout to flow from the grout pump through the driving tool down into the upper most pile shaft (see figure 3 and photo 4). The pumping rate of the grout should slightly exceed the take rate of the grout. In softer soils the grout takes tend to be higher due to outward bulging of the grout column.

Photo 4 Driving Hammer and Grout Adaptor

For lighter loaded piles the load transfer from the foundation to the pile can occur through the pile and inner grout interface. With higher loads an appropriately sized bearing plate is seated onto the pile with a reinforcing bar or other required termination hardware. For tension piles, threaded bar extends beyond the pile cut off.

Photo 5 Cutting of a driven pile with a grinder

In restricted headroom conditions the 16.4 foot long shaft may prove to be too long for installation. Coupler hardware is available that allows the pile to be cut into shorter lengths resulting in reduced headroom requirements for driving. The splice hardware is a friction connection that provides the full flexural resistance of the pile shaft.

Corrosion

The high carbon and silicon content in conjunction with the annealing process allows the nodular ductile cast iron to have a much higher corrosion resistance than structural steel. As with any other pile system, the corrosiveness of the host soils needs to be considered. These corrosion concerns can be eliminated with the use of externally grouted piles, due to the encasement of the pile shaft in grout. For end bearing piles by filling the interior of the pile shaft with grout, corrosion is limited to the exterior surface of the pile shaft.

Grout Materials

For end bearing piles, the pile shaft is filled with either a neat cement grout or sanded grout. The neat cement grout is tremied to the base of the pile. A sanded grout is pumped into the top of the pile. The choice of grout for end bearing piles depends on the size of the job, equipment and crew preferences. Either grout type can fulfill the strength requirements.

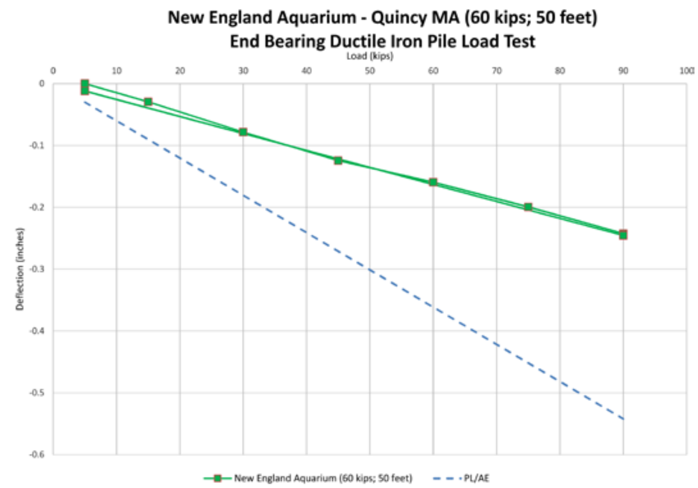
For friction piles a sanded grout is the most suitable. The grout flows through two constriction areas. The first is at the driving point and the second is at the pile grout shoe. Experience indicates that the flowability of grout can be maintained by having grout with a minimum cementitious content of about 900 pounds per cubic yard and a slump in the range of 7 to 8 inches. The shape of local sand aggregate will have an influence on the mix design.

New England Projects

Five local New England projects will be reviewed focusing on the installation and design procedures. For the Nantucket and the Waterbury State Office projects, friction DIPs were used to resist compression and tension loads. On the North Point, Holding Tank, and Johnson and Wales projects end bearing piles were used. We would like to thank the New England geotechnical design Professionals who worked with us on these projects for considering a new pile product, and working with Helical Drilling to carry out these projects. All of these projects included load testing of a sacrificial test pile per ASTM D1143.

Holding Tank Project – Quincy, MA (2009)

The reuse of an existing shipyard manufacturing building as a facility to aid injured sea mammals required the construction of 10 and 12 foot tall above grade tanks within the existing building. Borings revealed that one end of the site was filled land over organics over a clay stratum down to rock at 60 feet. The other end contained fill over a 15 foot deep sand layer underlain by a layer of bedrock. Several pile types were considered and it was decided to use a 60 kip end bearing pile at one end and a 40 kip friction pile seated in the 15 feet deep sand layer to support the new tanks. Due to the available headroom within the former shipyard industrial building a CAT 320 excavator with a Tremac 900 high-frequency vibrating hammer was mobilized and 78 piles were installed. Load tests on both friction and end bearing piles were performed. The results of both load tests are shown in Figure 4. The friction piles were driven to a depth of 30 feet to provide proper friction embedment. The end bearing piles were driven to a depth of 62 feet and a refusal criteria of less than 1 inch of penetration while advancing the pile for the last minute. A pile set criteria of 1 inch for the last minute of driving is a value that has been historically used as the termination criteria for end bearing piles. The design loads on the friction piles was 40 kips and the design loads on the end bearing piles were 60 kips.



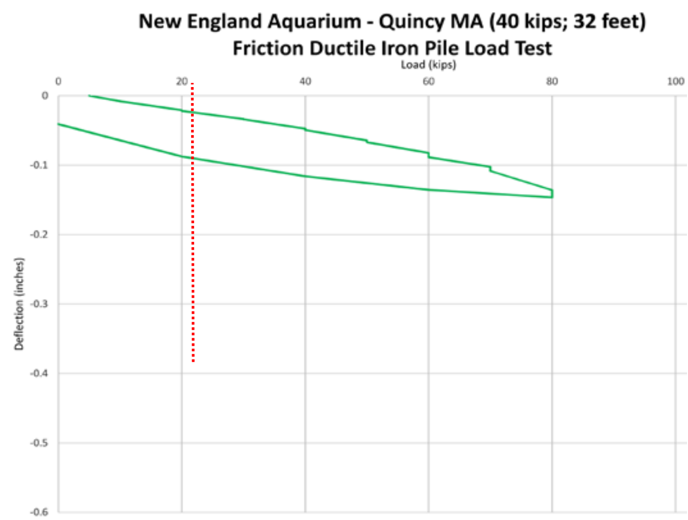


Figure 4 End Bearing and Friction Load Tests for NE Aquarium

North Point Landscape Structures Support – Cambridge, MA (2014)

Construction of a 15 story residential tower in the North Point area required a substantial raise in grade surrounding the residential building which was constructed on H piles bearing on rock. The geotechnical consultant recommended that the concrete landscape structures and sidewalks adjacent to the structure be pile supported to prevent long term settlement of the organics and clay. Numerous solutions were studied, but ductile iron piles were selected based on cost and flexibility of installation schedule. Flexibility of installation allowed additional time for the design of the landscape architecture to be fully developed. The selection of DIPs allowed the construction manager to install the piles within a sloped excavation site while the steel was being erected and long after the main tower foundations were installed. The DIP installation did not require a level working area. The soil strata consisted of variable depths of fill over a substantial depth of organics, up to 70 feet of marine clay underlain by glacial till and bedrock at 110 feet below grade. It was determined that 42 end bearing DIPs would support the design load of 60 kips if seated into the glacial till or bedrock. A load test was run to confirm the required design load with the factor of safety of two. The load test is shown in Figure 5. After the load cycle test was complete, it was decided to reload the pile to 300% and 400% of the design load for R&D purposes. The 400% cycle loaded the pile to 236 kips. This confirmed that the 4 5/8 diameter pile is capable of supporting loads in excess of 100 kips. The results of the reloading are also shown in Figure 5. The piles were driven using a midsized Volvo excavator fitted with a Tremac V1800 hammer to a set criteria of less than 1 inch in the last minute of driving. (see photo 6)

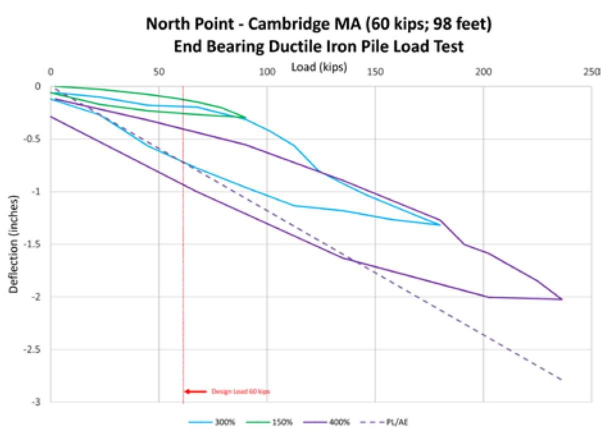


Figure 5 End Bearing Load Test for North Point

Johnson & Wales Center for PA Studies (2013)

Reconstruction of part of the existing structure required 31 pile foundations having a capacity of 70 kips. The project soil borings indicated fill over peat underlain by outwash sands and glacial till at 60 feet. End bearing DIPs were selected to bear on the till or bedrock at 60 feet. Pile driving commenced and the piles were advanced to a depth of 98 feet. Limited borings within the work area combined with a steep slope in the till/bedrock required that the piles be advanced to greater depths at one end of the work area. The crew was able to adjust to the increased depth by simply adding two additional extensions of ductile iron pile. The piles were seated into the till or bedrock using the driving set criteria of less than 1 inch of penetration per minute of driving. The load test for this project is shown on Figure 6. The pile on this project needed to be installed to within a 2 inch horizontal tolerance in the perpendicular direction of the grade beams. This installation criteria was achieved. Shorter friction piles were considered but lack of information about the outwash sands led to the use end bearing piles. The piles were driven with a Cat 320 excavator fitted with a Tremac 900 hammer.

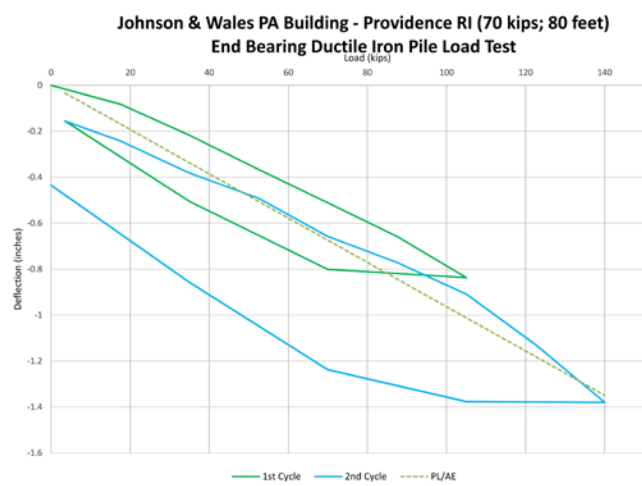


Figure 6 End Bearing Load Test for Johnson & Whales

Photo 6 Driving End Bearing Piles at North Point

Waterbury State Offices – Vermont (2014)

As part of the reconstruction of the Vermont State Offices Complex the woodchip storage pit within the wood fired biomass central plant required 73 hold down tension anchors. The project specifications indicated an 8 inch drilled micro-pile with a tension and compression capacity of 40 kips. The soils were coarse to fine sands with gravels. The soil strata within the pit area varied from loose to very dense sands. To be able to penetrate the very dense sands, the 8 5/8 inch conical grout point was selected. The piles were advanced 32 feet below the pit slab subgrade with the use of a Tremac V1800 Hammer mounted on a Volvo 240 excavator. Within the dense zones, the driving rate was very slow. In the less dense zones the rate of penetration was in the rate of 25 seconds per meter (8 seconds per foot). The design bond strength and embedment length were verified with a tension load test. After the load test was completed, the pile was tested to a load of 120 kips which was the capacity of the high strength tension reinforcing rod. This additional testing confirmed that the use of grouted friction ductile iron piles develops much higher bond stresses when compared to traditional drilled micropiles. The increased bond stresses are due to the displacement nature of the pile construction process. (See Figure 7)

Non-cohesive soils:

Driving Rate	Density	SPT (N60)	Applicable Skin Friction Values (Applied F.S.= 2)		FHWA[1] (Ultimate Strength)		PTI[2] (Ultimate Strength)	
sec/m			kN/m2	psi	kN/m2	psi	kN/m2	psi
Pushed	Very loose	< 4	0	0.0	0	0.0	--	--
5 - 10	Loose	4 - 10	40	5.8	70	10.2	--	--
10 - 20	Med dense	10 - 30	80	11.6	95	13.8	80	11.6
20 - 30	Dense	30 - 50	120	17.4	145	21.0	200	29.0
30+	Very dense	> 50	150	21.8	215	31.2	380	55.1

Cohesive soils:

Driving Rate	Density	SPT (N60)	Applicable Skin Friction Values (Applied F.S.= 2)		FHWA[1] (Ultimate Strength)		PTI[2] (Ultimate Strength)	
sec/m			kN/m2	psi	kN/m2	psi	kN/m2	psi
Pushed	Very soft		0	0.0	0	0.0	--	--
Pushed	Soft	0 - 2	0	0.0	35	5.1	30	4.4
5 - 10	Firm	3 - 8	20	2.9	35	5.1	40	5.8
10 - 15	Stiff	8 - 15	40	5.8	50	7.3	50	7.3
15 - 30	Very stiff	16 - 30	70	10.2	70	10.2	170	24.7
30+	Very hard	> 30	100	14.5	120	17.4	250	36.3

NOTES

1. Type A micropile grout-to-ground bond stress from FHWA "Micropile Design and Construction Guidelines" publication no. FHWA-SA-97-070. June 2011
2. PTI "Recommendations for Prestressed Rock & Soil Anchors" 2004

Figure 7 Bond Stress Comparison Chart

The load test results are shown in Figure 8.

Photo 7 Site Conditions in Waterbury Vermont

Photo 8 Driving Friction Piles in Waterbury Vermont. Note the variation in grades.

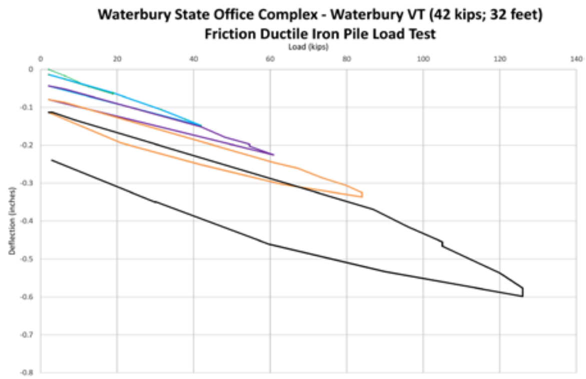


Figure 8 Load Test Results Waterbury Vermont

<u>Derrymore Street Residence, Nantucket, MA (2014)</u>

During the initial civil engineering evaluation of a residential lot on the island of Nantucket, it was discovered that the entire lot was underlain by an 18 foot deep layer of fill and organics over medium dense sands. Several types of piles were considered before selecting friction ductile iron piles having a capacity of 50 kips to support the two story residence and the kidney shaped pool. After overcoming transportation logistics to get materials and equipment to the island, the crew patiently backed up the tractor trailer 1.5 miles to access the site. Once there a test pile was installed to a depth of 32 feet below the working grade. The bond length of 14 feet was used to develop a working bond stress of 11 psi. A load test confirmed the design load with a factor of safety of two. The results of the load test are shown in Figure 8. Seventy four friction piles were installed in five days. The penetration rates with the sands in the bond zone closely matched the historically predicted driving rates.

Photos 7, 8, and 9 Derrymore Street House Site

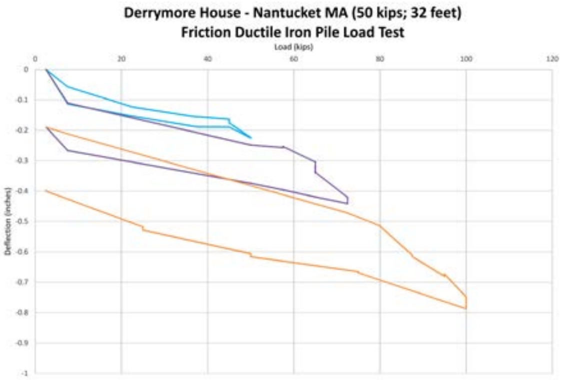


Figure 9 Load Test Results Derrymore House

Conclusion

Ductile iron piles have been successfully used on a range of projects throughout the extremes of the New England area. Designs in accordance with the 2009 IBC have proved to be competitive with drilled micropiles and various other types of end bearing piles. DIPs have been designed in both friction and end bearing applications. The DIP pile system provided both financial, as well as operational benefits to completed projects. For example, mobilizing an excavator to Nantucket versus a pile driving rig provided substantial logistical and driving advantages. When soil conditions differ, adding pile length proves to be an easy task. We believe that the DIP system will have great advantages on tight sites with deep depths to the end bearing stratum. The DIP system also will be used very efficiently on projects where loads can be shed in friction at shallower depths. Both end bearing and friction DIPs prove to be cost-effective, adaptable to varied site grading conditions and the simplicity of mobilization. The virtually vibration free driving process allows the installation of piles adjacent to existing structures. Peak particle velocity values recorded during the installation of DIPs typically range from .04 to .06 inches per second. This compares well to precast concrete piles which exhibit values of .2 to .4 inches per second.[3]

References

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3. The Driven Ductile Cast Iron Pile – A Practical Handbook, January 2007. Bauer Speziltiefbau GmbH
4. International Building Code, 2009