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Tensile Resistance of Ductile Iron Piles

Tensile loads on foundations are often caused by transient loading conditions from wind or seismic demand or by sustained conditions resulting from lateral pressures on retaining walls or embedded structure buoyancy. Tensile resistance is a common design requirement for deep foundations. This document describes the installation, design and performance of Ductile Iron Piles used for tensile resistance.

Background

As background, the Ductile Iron Pile system (Figure 1) is a low vibration, driven pile system utilizing high strength ductile iron piles manufactured in Europe using a centrifugal- or spun-cast method of fabrication. Modular piles sections have a Plug & Drive manufactured connection using a tapered socket with an internal shoulder for full engagement at one end and a tapered spigot at the other end. This system rapidly connects to form a pile of any length without the effort of dedicated field welding or splicing.

Figure 1: Ductile Iron Pile System

The modular system uses an excavator-mounted, high-frequency hydraulic hammer fitted with a special drive adapter that rapidly advances the pile into the ground using a combination of excavator crowd force and the percussive (ramming) energy from the hammer. Piles develop capacity through either end-bearing on rock or very dense soil (e.g. glacial till) (for compression only) or through friction along the pile-soil interface (compression and tension). The pile-soil interface can be characterized by either 1) the interaction between the roughened Ductile Iron Pile and soil or 2) a grout-to-ground bond zone constructed when an oversized grout shoe is driven while continuously pumping cement grout to fill the annular space.

Piles are available in multiple diameters ranging from 98 mm (3.9 in.) to 170 mm (6.7 in.) and wall thicknesses ranging from 6 mm (0.24 in.) to 13 mm (0.51 in.) allowing for the most efficient and cost-effective designs. For friction piles, grout shoes range in diameter from 150 mm (5.9 in.) to 370 mm (14.6 in.). The variable diameters and wall thicknesses provide piles with working compression capacities ranging from 25 tons to greater than 120 tons. Ductile Iron Piles, outfitted with a center reinforcing bar, can also develop tension resistance. Allowable tension capacity depends on the installation method (dry or wet) and the center bar diameter and range from 10 tons to greater than 75 tons.

Installation

Ductile Iron Piles used for tensile resistance are installed using either a dry method (non-grouted exterior) or a wet method (exterior grouted). Figure 2 provides pile details for the different methods. To maximize tensile resistance through grout-to-ground bonding, the latter method is most used for tensile resistance.

For the **dry installation method**, a drive shoe is placed on the end of the first modular pile section. The modular pile section is driven in the ground. The tapered end of the next pile section is inserted into the bell of the first section and the pile is advanced further. This process is repeated until the pile reaches the design termination depth. A center threadbar designed to resist the full tensile demand is inserted into the pile. For non-grouted (exterior) piles, the bar may be positioned in-place within the interior of the pile followed by placing grout with a tremie tube. Alternatively, the pile interior is filled with grout and then bar is then lowered into the grout.

For the **wet installation method** (exterior-grouted), piles are installed by using an oversized grout shoe at the pile tip that creates an annular space around the pile while driving. Cement grout is continuously pumped through the center of the pile to fill the annular space while advancing the pile. Once the grouted pile reaches the design depth, a high-strength center bar is immediately lowered into the wet grout.

For either installation method, once the pile installation is complete, a tension plate is bolted in place on the high strength center bar sticking up above the pile. The Ductile Iron Pile, center bar and bearing/tension plates are encapsulated in the concrete foundation. For more information on the installation process can be found in the DuroTerra brochure.

Design

Design of Ductile Iron Piles for tension resistance must consider both the structural capacity and the geotechnical capacity. Design approaches rely on applicable building codes and industry-standard methods for determining capacity as described below. The following overview of the design is based on allowable stress design (ASD) principals. The design may also be performed using load resistance factor design (LRFD), but details are not specifically included in this document.

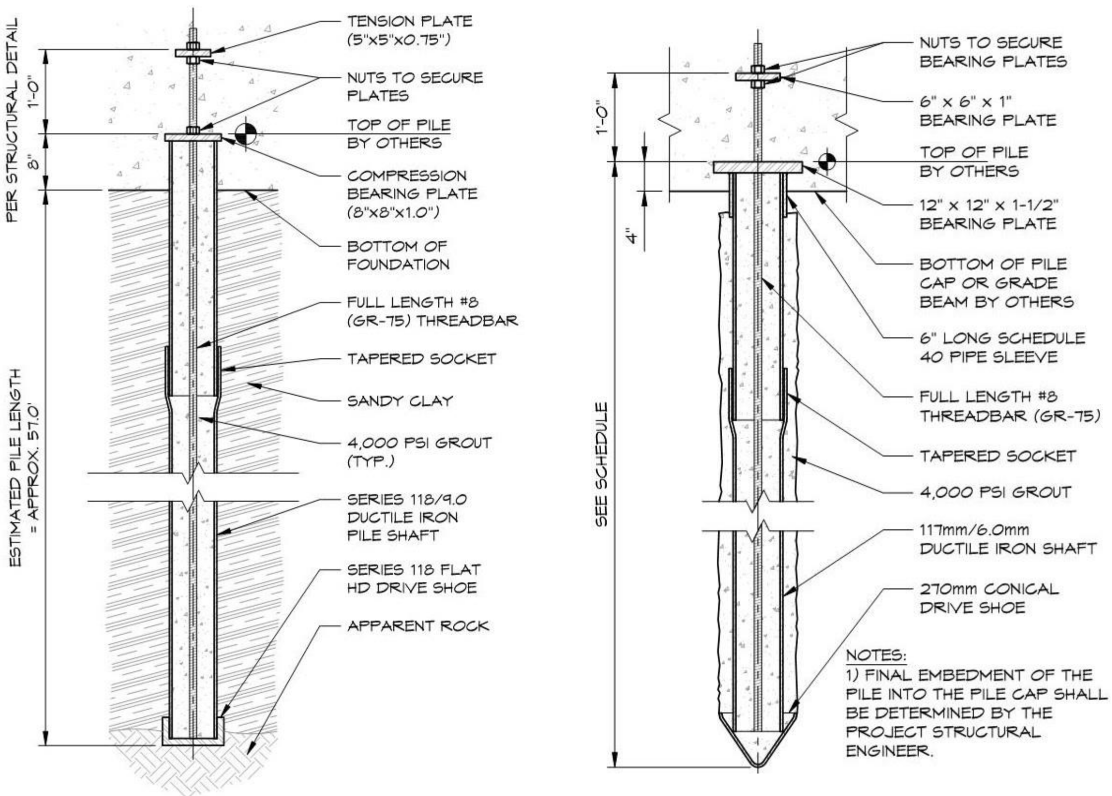

Figure 2: Examples of Ductile Iron Pile Tension Details – a) Non-grouted (Exterior) and b) Grouted (Exterior)

Structural Capacity

As discussed in the DuroTerra Bending and Lateral Resistance Tech Brief, the Plug and Drive connection of the Ductile Iron Piles forms a moment-resisting joint that field welds the modular pile sections during the driving process. Monitoring performed by GRL Engineers, Inc. of an instrumented pile suggests that the Plug and Drive connection fuses during the process of driving (Ryberg 2021). Despite the high integrity of the Plug and Drive joint, the design of the Ductile Iron Pile to resist tension forces typically relies only on the capacity of a high-strength center bar to resist the full tensile load on the pile. The center bar is extended nearly full-length in the center of the pile. The interior grout bonds the bar to the Ductile Iron Pile so the composite pile acts as a single unit.

From an allowable stress design (ASD) approach, the allowable structural capacity ($P_{all\text{-}tension}$) is a function of the bar size and strength and the allowable stress factor as shown in Equation 1:

$$P_{all-tension} = \mu_t F_{y-bar} A_{bar} \qquad \text{Eq. 1}$$

where $F_{y\text{-}bar}$ typically ranges between 75 and 150 ksi and bar sizes typically range between ¾-inch and 1-$^3/_8$ inches. Larger bar sizes become impractical to install inside the small diameter pile. The IBC lists an allowable stress factor (μ_t) value of 0.6 for micropile bars in tension, which is most applicable to this type of pile (IBC 2018). State and local codes should also be checked for deviation from the IBC. The resulting allowable structural capacity must exceed the tensile demand on the pile.

The tensile load transfer from the Ductile Iron Pile to the pile cap requires the thread bar extend into the pile cap. A tension plate is commonly attached to the top of the bar. The plate width is designed to minimize pile cap concrete stresses, while the plate thickness is designed to provide acceptable bending resistance of the plate. Structural connection details between the Ductile Iron Pile and the pile cap require coordination between the project Structural Engineer of Record (SEOR) and the Ductile Iron Pile designer.

Geotechnical Capacity

The geotechnical capacity of the Ductile Iron Pile in tension considers a cylindrical shearing surface as a function of the pile length and outermost pile diameter. For piles installed using the dry method (non-grouted exterior), the outermost diameter simply refers to the pile outer diameter. For piles installed using the wet method (grouted exterior), the outermost diameter that defines the perimeter cylindrical surface is based on the diameter of the grouting shoe. For instance, a Series 170/9.0 pile size with a 170 mm (6.69 in) diameter pile that uses a 270 mm (10.6 in) grout shoe is designed with a diameter of about 10.6 inches because the exterior grout column defines the shearing surface.

Multiple approaches are commonly used in the industry to evaluate pile capacity depending on the pile type and the soil conditions. Geotechnical capacity for Ductile Iron Piles is often estimated based on a simplified grout-to-ground bond stress approach (Sabatini et al 2005) often used for micropiles or an effective stress method like the Beta Method (Hannigan et al 2016).

<u>Bond Stress Method</u>

For exterior grouted piles, the grout forms a grout-to-ground bond along the length of the pile. This bond engages the soil during loading to provide shaft friction along the pile. The grout-to-ground bond is typically estimated as a bond capacity value (α_{bond}) that is a function of the soil type and stiffness or density. Example bond values for Type B micropiles (pressure grouted) that are most similar to values observed during Ductile Iron Pile load testing are shown in Table 1 (Sabatini et al 2005). Note that while the tabulated bond values are independent of depth or effective stress, experience shows that reduced grout-to-ground values should be considered in situations with shallow friction piles and/or with high groundwater scenarios (low effective stress levels).

Table 1: Grout-to-Ground Bond Values (after Sabatini et al 2005)

Soil Description	Grout-to-Ground Bond Ultimate Strength, psi (kPa)
Silt & Clay (some sand) (soft, medium plastic)	5 – 14 (35 – 95)
Silt & Clay (some sand) (stiff, dense to very dense)	10 – 27.5 (70 – 190)
Sand (some silt) (fine, loose-medium dense	10 – 27.5 (70 – 190)
Sand (some silt, gravel) (fine-coarse, med. - very dense)	17.5 – 52 (120 – 360)
Gravel (some sand) (medium – very dense)	17.5 – 52 (120 – 360)
Glacial Till (silt, sand, gravel) (medium – very dense, cemented)	14 – 45 (95 – 310)

The allowable shaft resistance for an exterior grouted friction Ductile Iron Pile within a uniform soil deposit is estimated as shown in Equation 2:

$$Q_{all} = \frac{\pi D L_b \alpha_{bond}}{FS} \quad \text{Eq. 2}$$

where D is the outermost diameter of the Ductile Iron Pile (including exterior grout column), L_b is the bond length and FS is the factor of safety. The shaft resistance should refer only to soils where competent tensile resistance is anticipated. For instance, very soft soils, organic soils or some undocumented fills may be inappropriate for generating adequate tensile resistance for design and should be neglected from the calculation. In layered soil conditions, the resistance within each competent soil layer corresponding to the length and bond value is calculated and combined to determine the total allowable shaft resistance. Additionally, it is important that variations in strain rates between different soil types be considered when selecting bond values for design.

<u>Effective Stress Method</u>

For piles installed using the dry method (non-grouted exterior) or as an alternative to the Bond Stress Method, an Effective Stress Method can be used to estimate the shaft resistance along the pile. This method applies the Beta Method (Hannigan et al 2016) where the frictional resistance is a function of the horizontal effective stress around the pile as well as the interface friction between the pile surface and the soil. Similar to the Bond Stress Method, only competent soil layers should be considered as contributing to the available shaft resistance.

The allowable shaft resistance for a friction Ductile Iron Pile using the Effective Stress Method is estimated as shown in Equation 3:

$$Q_{all} = \frac{\pi D K_s \sigma_v' \tan\delta}{FS} \qquad \text{Eq. 3}$$

where K_s is the coefficient of lateral earth pressure, σ'_v is the average vertical effective stress along the pile shaft and δ is the interface friction angle between the pile surface and the surrounding soil. The expression $K_s \tan \delta$ is often referred to as the Bjerrum-Burland beta coefficient. Values for the beta coefficient range significantly in the literature depending on the ground conditions (soil type, composition, density, etc) as well as the pile type and pile surface characteristics. Displacement piles (i.e. driven piles) typically have higher values than other types of piles (i.e. drilled) due to the volumetric expansion and displacement that occurs during the driving process particularly in densifiable conditions.

A general range of beta coefficient values for various soil conditions vary from 0.15 to 0.90 after Fellenius (Hannigan 2016). Other approaches like those developed by Nordlund indicate higher values of the equivalent beta coefficient values depending on the soil friction angle and amount of volumetric displacement that occurs. Tension load test experience with Ductile Iron Piles suggests the beta coefficient values are often on the higher end of the range, particularly for the non-cohesive soils due to the displacement and corresponding densification that occurs in the surrounding soils during driving combined with the high interface friction between the pile surface and the soil due to the roughened Ductile Iron Pile surface that occurs from the manufacturing process. Unlike a smooth steel pipe pile, the surface of a Ductile Iron Pile is roughened and more comparable to the undulations on a golf ball. Further, the interface friction value for an exterior grouted Ductile Iron Pile where the grout bonds to the surrounding soil should default to the full soil friction angle.

<u>Factors of Safety for Design</u>

A factor of safety of 2.0 is often used when tension load testing is performed. Higher values of 2.5 or 3.0 may be more appropriate when load testing is not performed, and the capacity is determined by analysis only. Note that when loading is due to transient conditions from wind or seismic, IBC Section 1810.3.3.1.5 does allow for a reduced factor of safety of 1.5 when performing load testing or 2.0 when capacity is determined from analysis. These reductions should be used with caution with a complete understanding of the structural load combinations and impacts of the tensile resistance on the design.

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Performance

Full-scale tension load tests performed in general accordance with ASTM D-3689 are routinely used to verify tension resistance for Friction Ductile Iron Piles. Tension tests are also used to conservatively verify the capacity of compression piles developing capacity in only friction (excluding any end-bearing component). These tests provide valuable data to the frictional performance of the system in various soil conditions.

Results of two tension load tests performed on exterior-grouted friction Ductile Iron Piles are shown in Figure 3. Both piles were installed with Series 170 Ductile Iron Piles with a 270 mm (10.6-in) grout shoe to install a 10.5-in diameter grouted displacement pile. Test Pile #1 used a 7.5 mm wall thickness, while Test Pile #2 used a 9 mm wall thickness. Both piles were installed to approximately 32 feet (2 full 5-meter sticks). A 1-¾ inch, Grade 150 ksi threadbar was inserted in each pile to provide the tensile resistance. Soil conditions at these sites were generally loose to dense sand. A 4-ft thick organic layer was encountered between depths of 6 and 10 feet in Test Pile #1. Both piles performed similarly with about 0.5 inch of movement at loads approaching 300 kips.

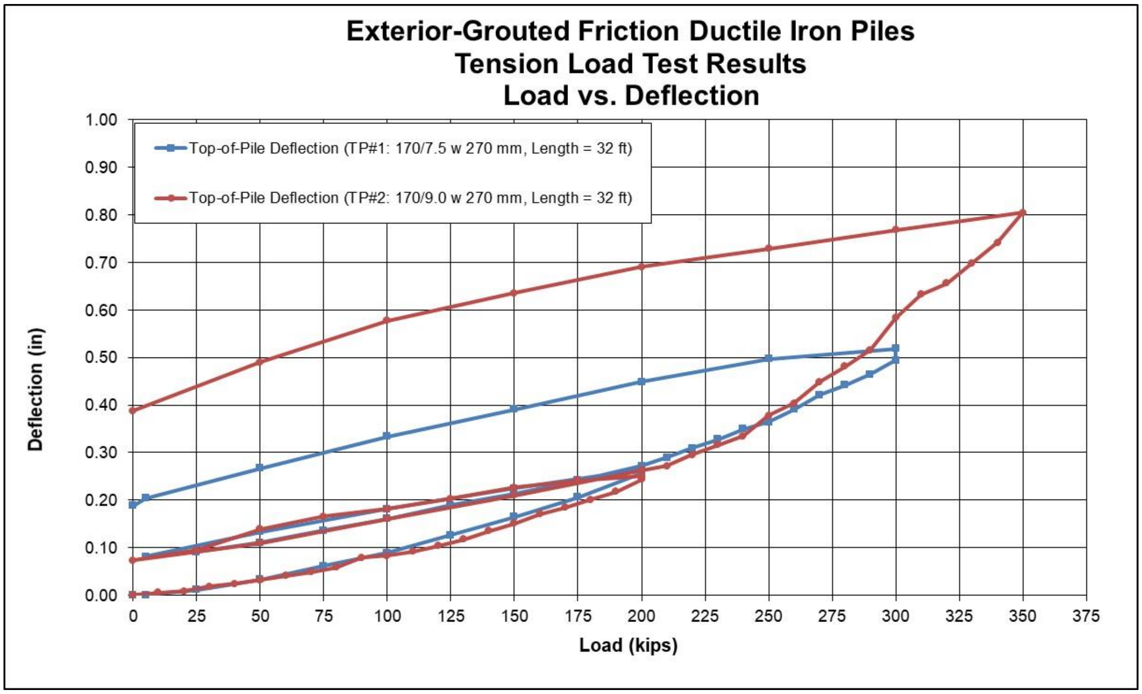

Figure 3: Tension Load Test Results for Exterior-Grouted Friction Ductile Iron Pile

Interpretation of the test results involves review of the elastic elongation of the test pile using strain theory as shown in Equation 4:

$$\delta_{elas-tens} = \frac{P_t L}{EA_{comp}} \quad \text{Eq. 4}$$

where P_t is the average applied load over the length of the pile, L is the pile length over which the load is applied, and EA_{comp} is the axial rigidity of the pile. The data from repeated tension tests, including the tests in Figure 2, confirm that the axial rigidity term is characterized by the properties of the threadbar as well as the contribution of the tensile response of the Ductile Iron Pile material as well.

Conclusions

Ductile Iron Piles are routinely used for support of foundations loaded in compression. Ductile Iron Piles with a high strength threadbar inserted into the center of the grouted pile are also highly effective at resisting tensile loads. This Tech Brief describes the installation, design methods and provides evidence of the superior performance of the system.

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