RECENT TRENDS IN DRIVEN PILE FOUNDATIONS IN NEW ENGLAND

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ABSTRACT

Driven pile foundations are a commonly used deep foundation alternative throughout the U.S. and the world. Over the last ten years or so, design stresses for these pile foundations, particularly steel H-piles and pipe, have been trending higher due to detailed evaluation and understanding of soil/rock conditions and increased confidence in quality control and testing methodologies. Design stresses in steel piles have been routinely approaching and/or exceeding 20 ksi, in order to increase the allowable pile loads and enable the driven steel pile industry to compete more efficiently with other deep foundation types such as micro-piles or drilled shafts.

The use of higher stresses has now placed more emphasis on obtaining and classifying rock cores, particularly for the larger HP sections, where design loads of 200 to 300 tons or more are now being used. The pile capacity is now controlled by the rock quality and/or the drivability of the pile rather than the limitations on the structural steel design. As a result, larger hydraulic hammers with more efficient energy transfer and bigger load test frames are necessary for successful installation.

For other applications, in order to avoid driving to rock, which in some case can be over 250 feet deep, other types of innovative steel foundations are being used. These steel foundations have not commonly been used in New England. They are trademarked, fabricated, non-traditional pipe pile sections that improve soil resistance to keep pile penetration lengths relatively short with reasonably high design loads that would not normally be obtainable with standard steel or pipe piles.

This paper discusses the current trends in New England with regard to more frequent use of high capacity H-piles driven to rock and the emerging use of non-traditional pipe piles driven to shallow depths in sandy soils. Design challenges and case history examples are discussed.

INTRODUCTION

The current Massachusetts State Building Code (MSBC) is based on 2015 IBC with its own amendments and overrides. The allowable stresses for driven piles as well as other deep foundation elements in the IBC code are based on factored structural limits of the pile material. They are not based on geotechnical limits. The allowable stress in compression and tension for steel, H-piles and pipe piles based on MSBC Section 1810.3.2.6 is limited to 0.35Fy or 16 ksi. Throughout the most of the last century, a majority of projects used this limitation to establish the design load for the piles and were confirmed with static load testing. Typical design loads of 100 to 150 tons and rated hammer energies of 40 to 70 kip-ft were used to drive these piles based on the 16 ksi stress limit.

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Section 1810.3.2.8 of the Code allows for an increase of the allowable stresses to 0.5Fy or 32 ksi. This increase in allowable stress has to be justified with supporting data that includes performing a geotechnical investigation and a static load test. These two activities are routinely used on almost every project. So why wasn't the higher allowable stress routinely taken advantage of until recently? In our opinion it was based on three major factors: 1) the yield strength of steel was 36 ksi, which now is routinely 50 ksi, 2) better investigation and classification of rock which was traditionally reserved for drilled deep foundations, and 3) the common use of high strain dynamic pile testing for increased quality control (Chernauskas and Paikowsky, 1999).

Allowable stresses for steel are now commonly 20 ksi or higher to enable, for example, an HP14x102 to be driven to a 250 ton design load. The increased use of dynamic pile testing has allowed engineers to observe real time driving behavior in terms of driving stresses, transferred energies, resistances, integrity/damage, and displacements. Numerous piles can be tested around the site to assess variability in the hammer-pile-soil system performance so that the load test pile can be strategically and efficiently selected. This gives an additional level of confidence when pushing the design loads higher.

So what really controls the selection of the pile design load considering the recent use of allowable stresses approaching or exceeding 20 ksi? For HP14x102 or HP14x117 piles with 200 to 300 ton design loads, in many cases it is the rock conditions and drivability (pile geometry and hammer selection) that dictate the allowable load. These considerations are described below for examples using H-piles.

DRIVABILTY CONCERNS

One of the main controlling factors for establishing the allowable load on a high design stress pile is the drivability. Drivability is influenced by several factors, such as soil/rock conditions, pile geometry, and hammer size. The rock conditions will be described in the next section. Pile geometry has a significant effect on the drivability due to changes in length and cross-sectional area. Both of these parameters affect the overall system stiffness and mass. The hammer selection is critical for driving the pile to the required capacity within the allowable driving stresses and at reasonable blow counts.

Figure 1 illustrates the theoretical relationship between pile area (mass) and length (stiffness) with blow count. This figure was created using wave equation techniques, where the same hammer and energy were used to drive the piles. As the pile area increases, the blow count decreases. So for example, by increasing the pile size from an HP14x102 (30.2 square inches) to an HP14x117 (34.6 square inches), the blow count decreases from around 200 blows per foot (bpf) to around 120 bpf. The pile length, soil conditions and energy were held constant to develop the area vs blow count relationship. This is a significant theoretical reduction in blow count by selecting the next larger size of pile and allowing the increased mass and inertial effects to help contribute to its drivability.

In contrast to the area in Figure 1, as the pile length increases, the blow count increases. By increasing the pile length from 50 feet to 100 feet, the blow count increased from around 50 bpf to around 110 bpf. The pile area, soil conditions and energy were held constant to develop the length vs blow count relationship. This is a significant theoretical increase in blow count by doubling the pile length, which creates a less stiff system and reduces the efficiency of driving. It explains why long piles sometimes cannot develop or prove the required capacity as a result of energy losses due to large system quake (high pile elasticity that is further compounded when soil response is included).

A pile becomes theoretically undrivable when, simultaneously, the blow count is at refusal and the driving stresses exceed the allowable driving stress limit. In some cases a different hammer may solve the problem. However, in many cases, the load may have to be downgraded or the pile section upsized in order to allow the pile to become "drivable". This is more common for relatively thin wall pipe piles that rely on the additional load that the concrete can provide for the static case, but is unavailable for the driving case with the steel section alone. An H-pile driven to a relatively high load on shallow, sound rock can also fall into this category.

Figure 2 relates the drivability of a long steel H-pile considering three different energies (potentially different strokes or different hammer sizes). The solid lines are capacity versus blow count and the dashed lines are capacity versus compressive driving stress. The pile length, area and soil conditions were held constant for all cases. For this pile using a 65 kip-ft energy, if the desired capacity is 950 kips, the blow count is around 240 bpf (refusal) and the driving stress is around 54 ksi (greater than the typical 45 ksi limit). A larger hammer (more energy) would only increase the driving stresses. A smaller hammer (less energy) would only increase the blow count so that the 950 kip pile capacity could not practically be proven. The pile section would have to be increased or the capacity downgraded.

By using a lower capacity of 700 kips for the pile, the transferred energy of 65 kip-ft is too high because the driving stresses are around 50 ksi, even though the blow count is reasonable at 60 bpf. The transferred energy of 35 kip-ft is too low, because although the driving stresses are 39 ksi, the blow count is around refusal (240 bpf). The hammer that produces 50 kip-ft of transferred energy provides a reasonable blow count of 100 bpf and driving stress of just under 45 ksi. This provides a balance of pile-soil-hammer compatibility.

EXAMPLES - HIGH CAPACITY H-PILES DRIVEN INTO BOSTON ARGILLITE

As the allowable stresses for an H-pile increase and the required load also increases, the condition of the rock, particularly near the pile toe, can control the design. The higher loads have necessitated larger hammers, which frequently are hydraulic hammers, and bigger static load frames. An example of a large static load test reaction frame for over 500 tons test load is presented in Figure 3. A large hydraulic hammer (BSP CG240) with a maximum rated energy of 176 kip-ft is presented in Figure 4. Two examples of static pile load tests for HP14x117 H-piles driven in Boston Argillite on two separate projects are presented in Figures 5 and 6 for hard and soft rock, respectively. The hard rock in Figure 5 was able to develop an ultimate capacity of over 1300 kips, as exhibited by the elastic behavior of the load-deformation. The pile top displaced around 3 inches during the test since the pile was 200 feet long. However, the hard rock yielded low permanent set of less than 0.5 inches and significant rebound. In contrast, the soft rock in Figure 6 could not generate more than 800 kips in resistance, where the static load test indicated significant creep (i.e. pile movement with time under constant load) at the pile tip. It is interesting to note that the load-deformation relationship was completely elastic until the maximum load.

Hydraulic or diesel hammers capable of delivering over 100 kip-feet of transferred energy measured during dynamic pile testing are commonly used to drive these higher capacity piles. Figure 7 shows the dynamic pile testing results for an HP14x102 H-pile driven on another project in Boston. The soil profile consisted of approximately 150 feet of fill/organics and clay over 20 feet of glacial till over weathered Argillite rock. The left side of the plot presents the transferred energy with depth. As a result of the significant energy delivered to the pile, the pile penetrated through the weathered rock with relative ease and at a relatively low blow count of 5 to 6 blows per inch. In this case, the weathered rock could not develop enough point resistance before crushing and thereby was limited to around 800 kips (middle plot of Figure 7). It is also a point of interest to see the compressive stresses at the pile tip slightly decreasing as the pile penetrated deeper into the weathered rock, due to the increase in skin friction as it penetrates (right plot in Figure 7).

Figure 8 illustrates the same project, where two static load tests were performed on a single HP14x102 pile. Prior to the first load test, the pile was driven to refusal using a hammer capable of delivering around 70 to 80 kip-ft of transferred energy. The pile started to creep at around 445 tons and passed through the failure criteria. Since the pile did not achieve the required ultimate capacity of 460 tons, the pile was redriven and a second load test was performed. Around 100 to 105 kip-ft of transferred energy was delivered to the pile as it was redriven to refusal a few inches below where it originally ended. The pile started to creep excessively at approximately 460 tons, which is similar as the first test (few percent difference). This demonstrates that in this case, the resistance in the rock was limited, and did not increase, even when redriven with substantially more energy. The back-calculated crushing strength of the rock was around 15,000 psi based on load transfer and reinforced pile point area.

EXAMPLES - INNOVATIVE PIPE PILE TYPES FOR SANDS

While design loads and allowable stresses for steel H-piles driven to rock have been on the rise, other pile types have been recently used in the New England area to generate more resistance in the soil at shallow depths. Pipe piles traditionally are used for marine locations comprised of medium dense sands. Even with a plate on the bottom, pipe piles are limited in their ability to generate resistance, and in fact, provide minimal increase in resistance as they are driven deeper (particularly in medium dense sands). Two innovative pile types that can increase a pipe pile resistance at shallow depth (typically less than 75 feet) are called Tapertube™ piles and Spin Fin™ piles. These pile types are based on standard pipe sections with modifications over the lower portion to increase the pile resistance. The Tapertube has a tapered section over the lower 25 feet that helps generate increased resistance due to the angle. The Spin Fin has a vertical steel plates (Fins) welded over the lower 5 feet of the pile on a slight angle that helps increase both compression and uplift capacity.

Tapertube piles were selected as a value engineering alternative for a recent DOT project in Providence, Rhode Island with a medium dense to dense sand profile based on their ability to generate more resistance than driven pipe piles or the originally proposed drilled micro-piles at the same or shallower depth. A comprehensive pile load test program was subsequently proposed and executed by GTR to assess pile drivability, load-deformation performance, and to develop site-specific design parameters in two (2) distinct test areas – East and West. One (1) 16-inch OD pipe test pile and one (1) 16-inch OD Tapertube test pile were impact driven at each test area. The test pile program included dynamic pile tests and static compression, tension and lateral load tests on each test pile. Some photos of the Tapertube piles are presented in Figures 9 and 10.

The test piles in the East Area were driven to 2 to 4 blows per inch (bpi) with the same HHK-5A hammer providing 22 to 27 kip-ft of transferred energy. The static load test results for the East Test Area test piles in Figures 11a and 11b illustrate the Tapertube piles developed over 20% capacity than the counterpart pipe pile (TT pile driven 55 feet and pipe pile driven to 70 feet), thereby allowing the project to reduce the number of required piles (Hamblin et al., 2018). Over 18 months was saved on the construction schedule with a cost savings in the vicinity of seven figures compared to the originally proposed micropiles. The use of the Tapertube piles was a first for a large scale infrastructure project in southeastern New England (Hamblin, et al., 2019).

Spin Fin piles can generate significantly more uplift and compression resistance than the equivalent counterpart pipe piles. Spin Fin piles were used on several offshore projects in Southeastern Massachusetts. One of the projects in Oak Bluffs involved a design phase test program to install Spin fin and pipe piles of various sizes. Spin Fin photos are shown in Figures 12 and 13. GTR carried out the test program to evaluate the drivability, compression and tensile capacity of the piles for future design at several ferry terminals. The design philosophy for the deck and wharf structures involved driving 16” Spin Fin piles to a shallow depth of 25 feet for a compression design load of 50 tons and to 35 feet for a tension design load of 30 tons. The pipe piles required penetrations greater than 50 feet to obtain similar results.

Figures 14a and 14b illustrate the cyclic load tests performed on the Spin Fin piles in compression and tension. The tests highlight the ability of these piles to generate significant resistance at shallower depths. Table 1 summarizes the results of the offshore project in Oak Bluffs, where significant pile footage was saved, in addition to the costly and time intensive splices that would have been needed for the pipe piles (Chernauskas et al., 2011). At the Oak Bluffs and Hyannis Terminals, the use of the Spin Fin piles saved over $1,000,000 in installation costs. The Oak Bluffs test program has provided an experience base for future Steamship Authority projects where offshore and/or pier facility structures are planned.

CONCLUSIONS

Allowable loads of 200 to 300 tons are becoming more common for HP14 sizes in Massachusetts. Driving H-piles using high allowable stresses may be unprecedented in certain areas. As the loads become higher, more emphasis must be placed on characterizing the rock. The engineer should obtain good quality rock cores and evaluate the degree of weathering and fracturing. On some of these projects, if the test program reveals that a pile cannot be driven to the required capacity due to drivability or rock limitations, then downsizing the pile capacity, increasing the pile size or increasing the number piles may be required. The project owner should be willing to share this risk and allow for these contingencies with the contractor.

Spin Fin™ and Tapertube™ piles are being more commonly used in Southeastern New England. For medium dense sand profile sites, where fairly deep penetrations are required for pipe piles, Spin Fin and Tapertube piles can be used to develop modest compression and/or tension capacities at shallow depths. When these pile types can reduce penetrations around 20 to 25 feet compared to traditional pipe piles, they become more cost effective. An even more valuable benefit is the acceleration of the construction schedule, due to the quick installation as a result of shallow penetrations and elimination of splices.

REFERENCES

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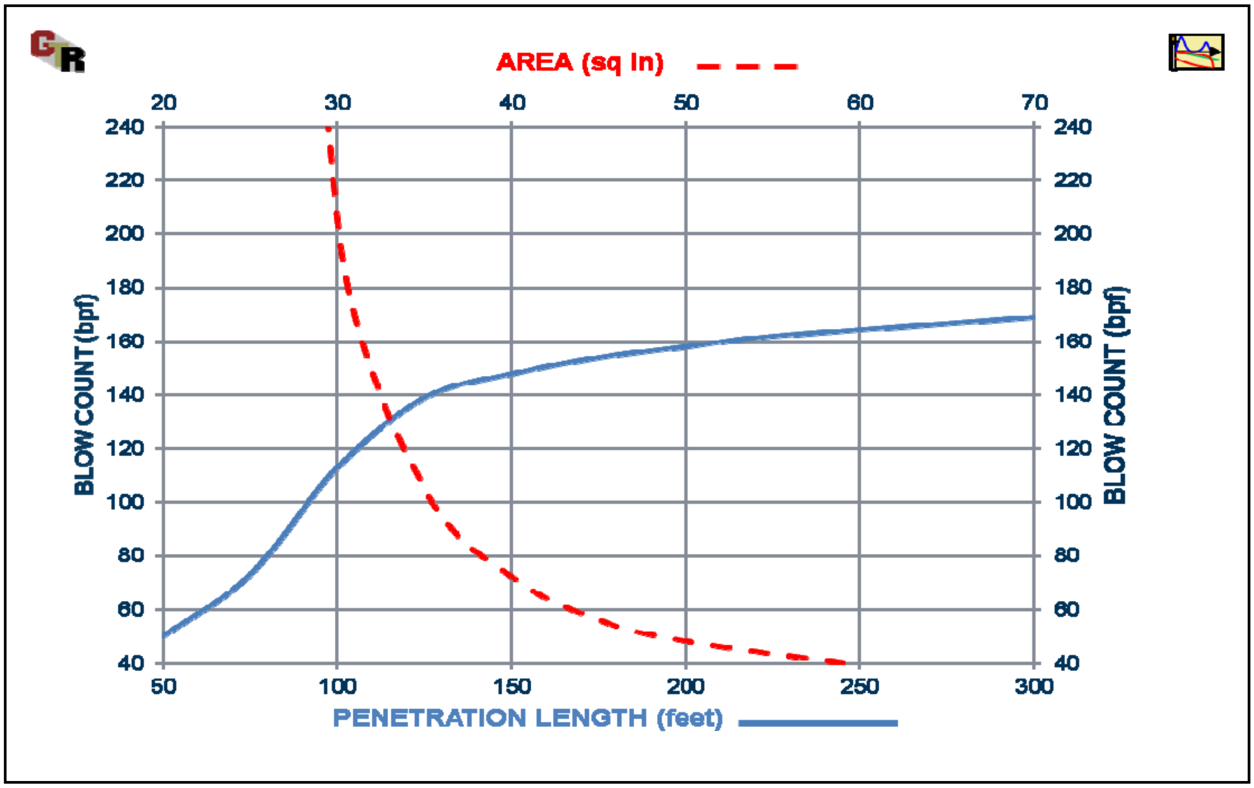


Figure 1 – Pile Drivability - Area and Length vs Blow Count

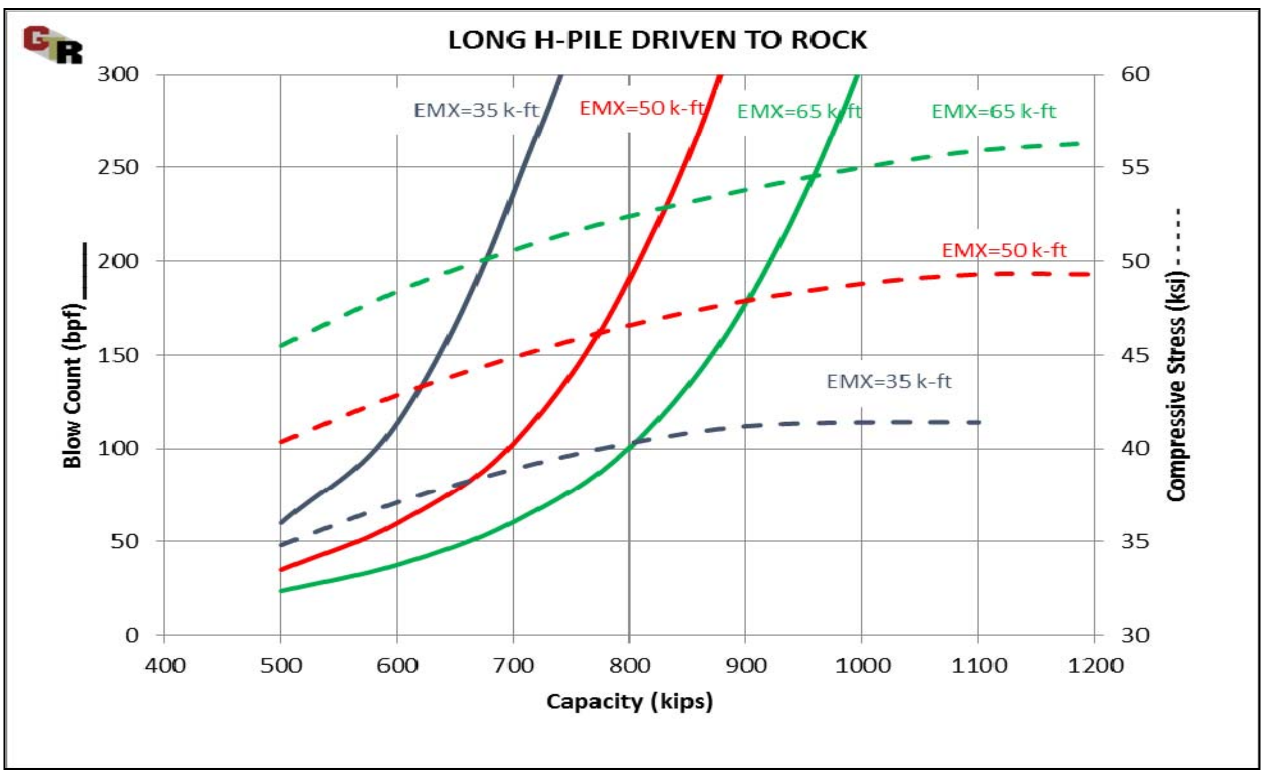


Figure 2 – Pile Drivability – Blow Count and Stress vs Capacity with Energy Variation

Figure 3 – Large Static Load Test Frame For 500 ton Compression Test Load (photo courtesy of GTR)


Figure 4 – Large Hydraulic Hammer – BSP CG240 - 176 kip-ft Rated Energy (photo courtesy of GTR)

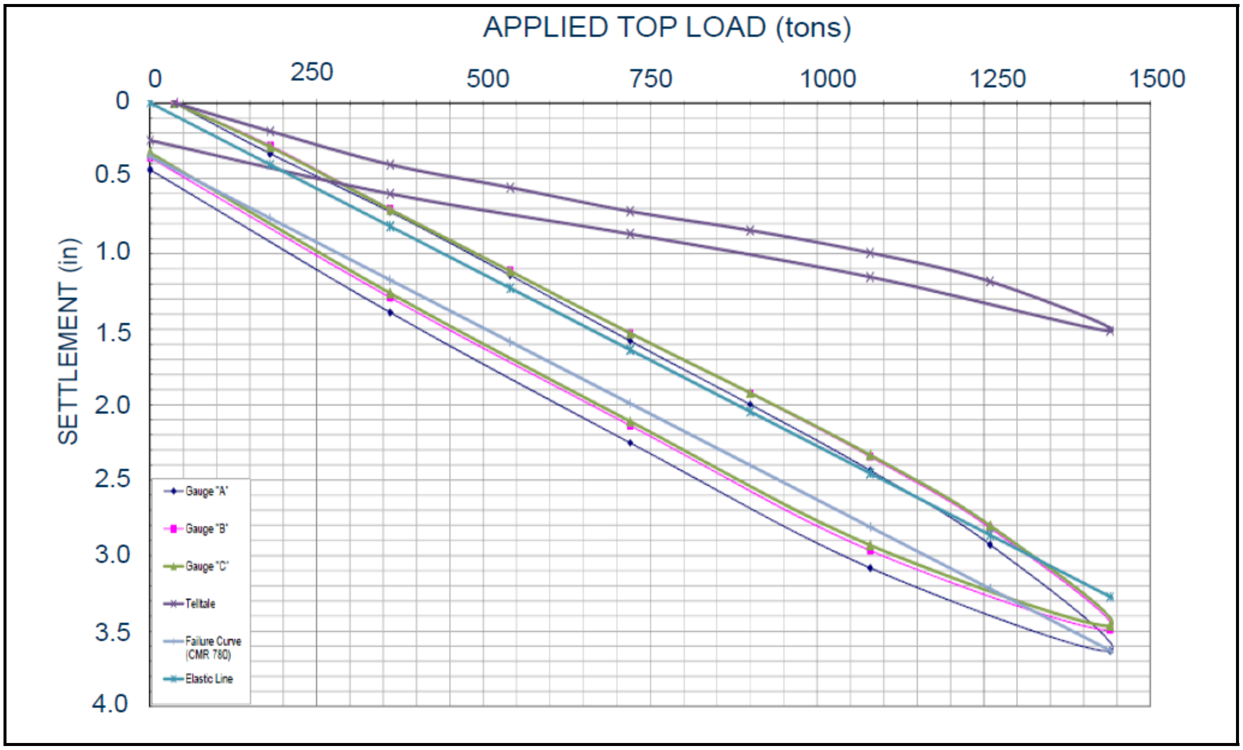


Figure 5 Static Load Test Pile Top Load vs Displacement Hard Boston Argillite (Plot courtesy of Pare Engineering)

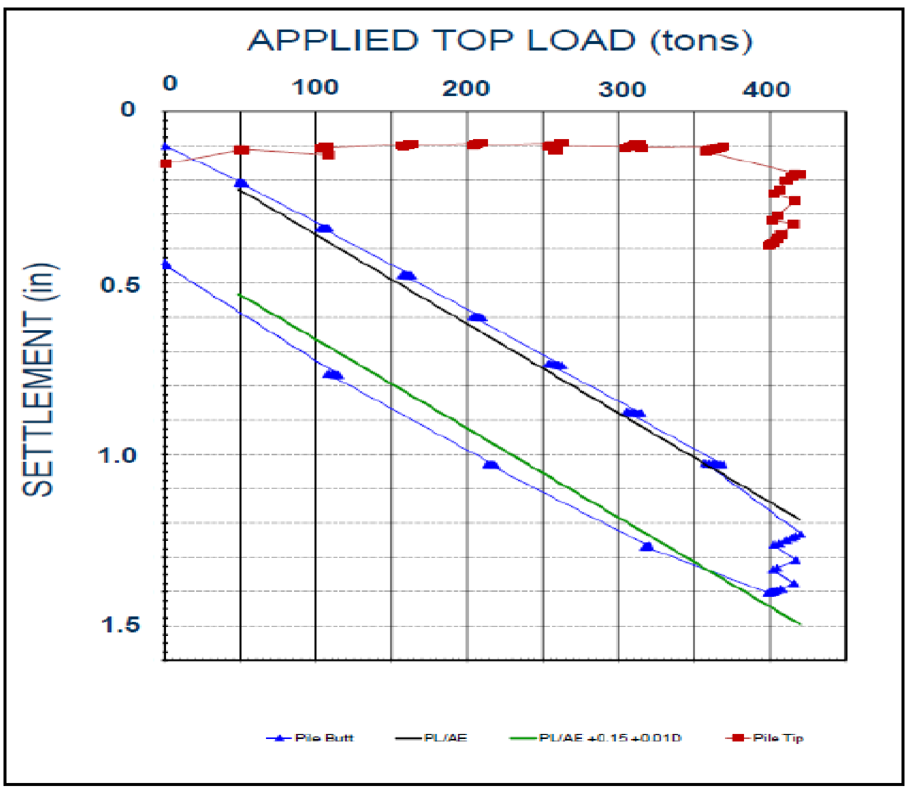


Figure 6 Static Load Test Pile Top Load vs Displacement Soft/Weathered Boston Argillite (Plot courtesy of Haley And Aldrich)

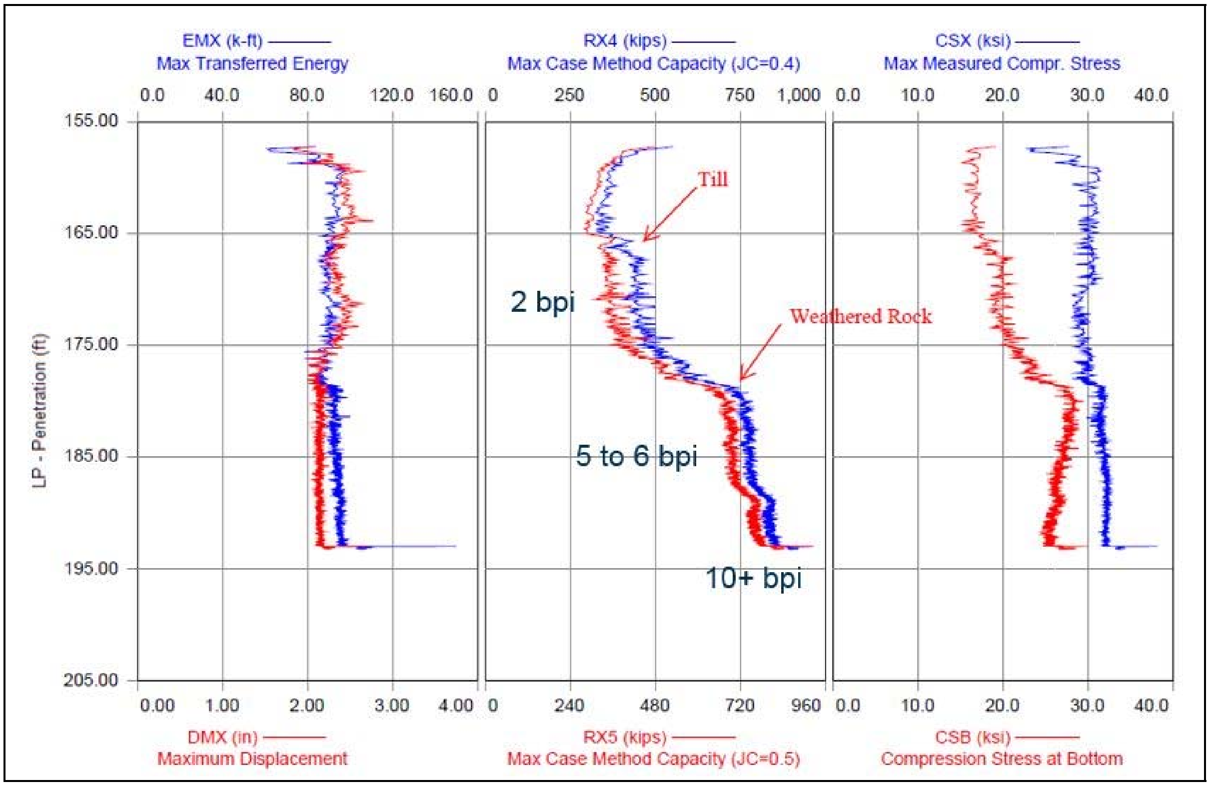


Figure 7 Dynamic Pile Testing Results for HP14x102 Pile Driven in Weathered Argillite

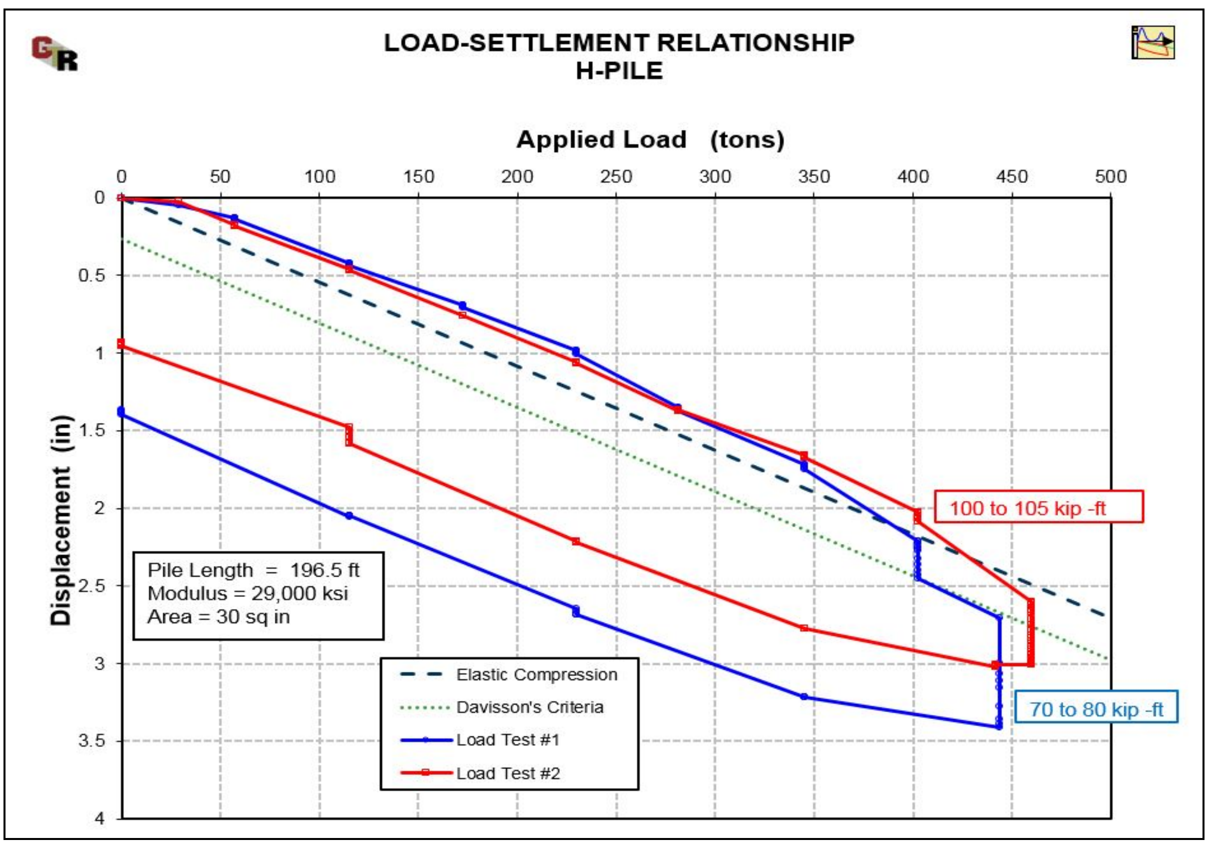
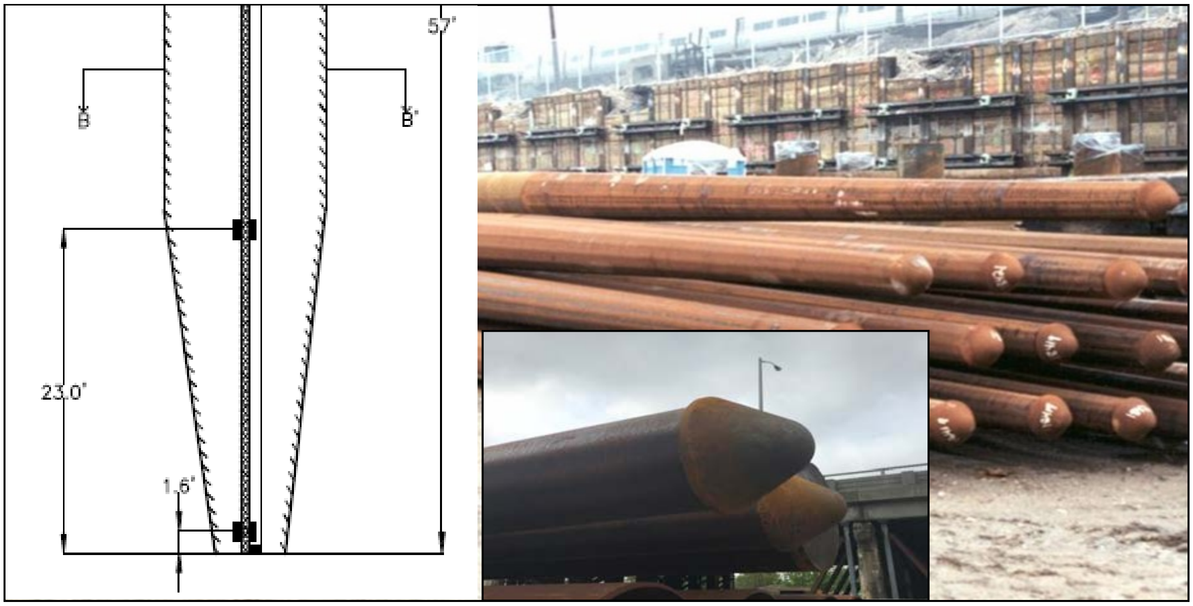


Figure 8 Static Load Test Results for HP14x102 Pile in Weathered Argillite



Figures 9 and 10 – Tapertube™ Pile Detail and Photos (photo courtesy of GTR)

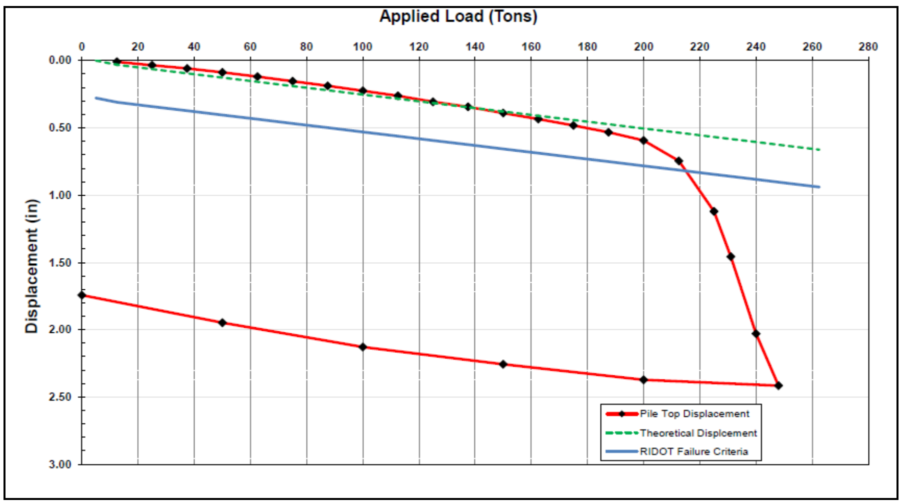


Figure 11a – Tapertube Pile Static Load Test in Providence Sand

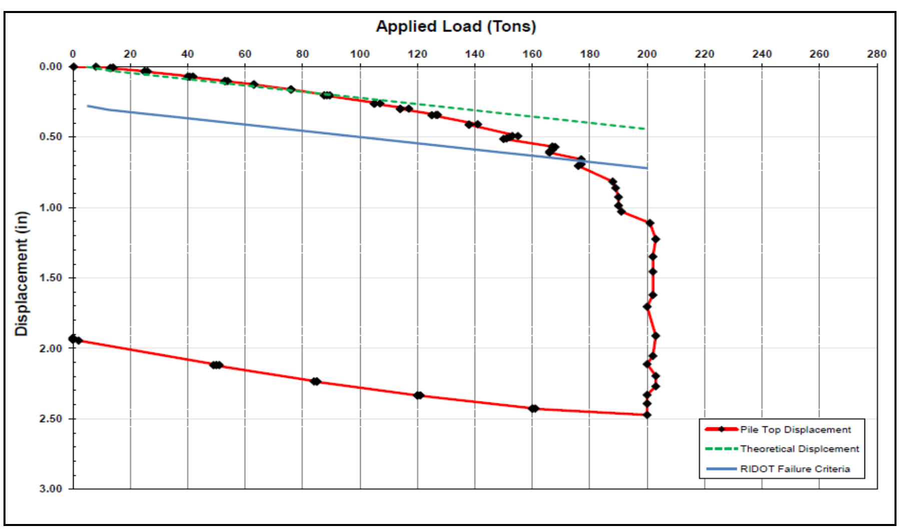
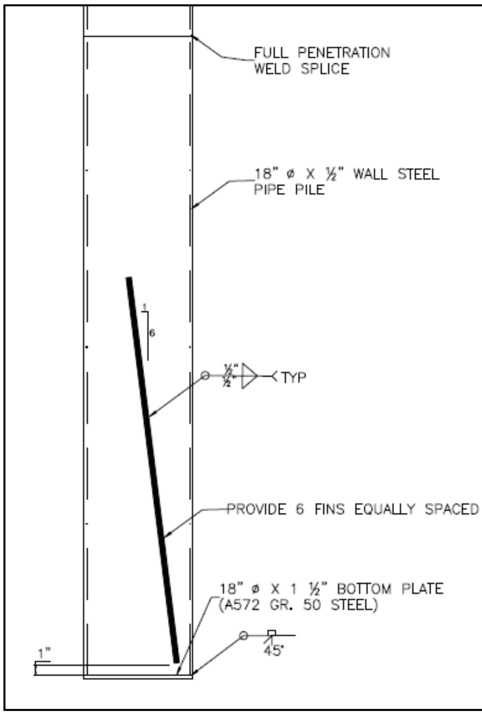
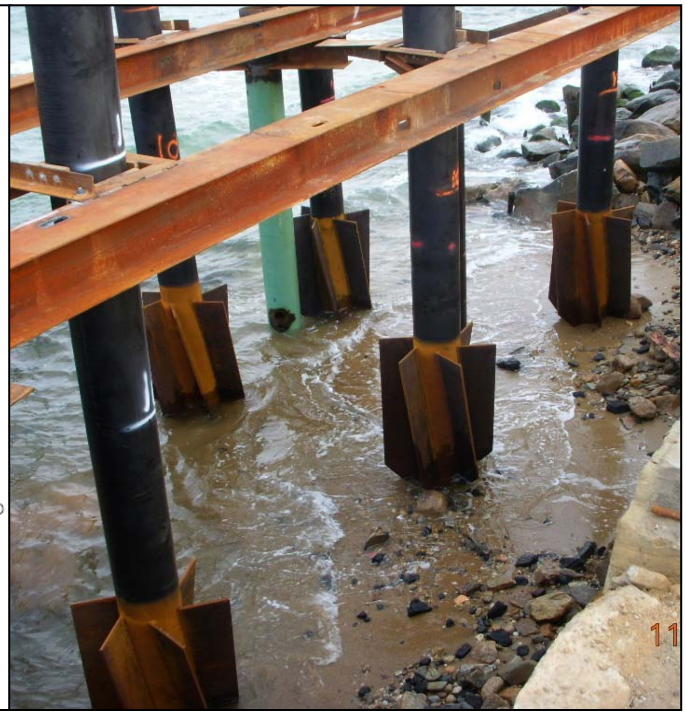
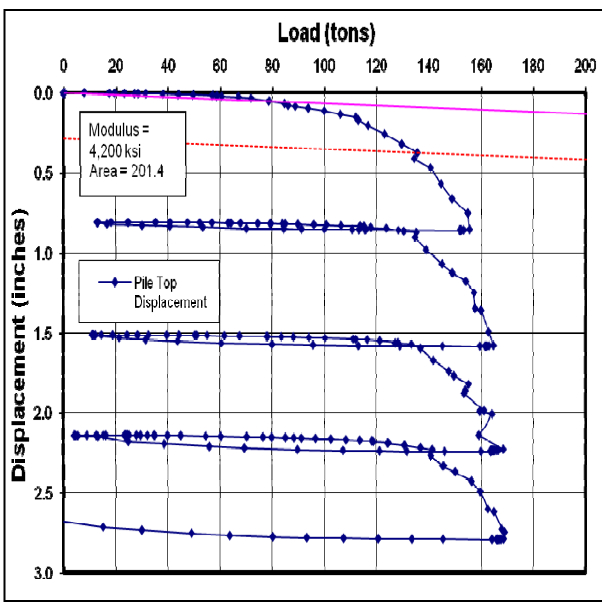


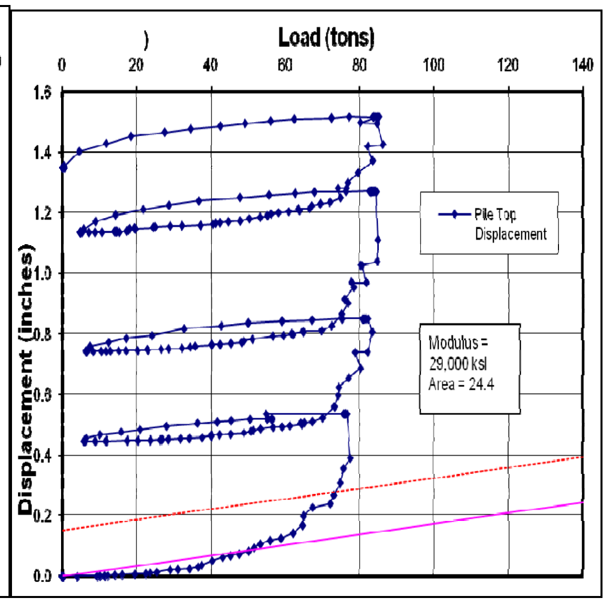
Figure 11b – Pipe Pile Static Load Test in Providence Sand





Figures 12 and 13 – Spin Fin™ Pile Detail and Photo (courtesy of GTR)





(a) **(b)**

Figure 14a - SpinFin Compression Static Cyclic Load Test Results in Cape Cod Marine Sand
Figure 14b - SpinFin Tension Static Cyclic Load Test Results in Cape Cod Marine Sand

Table 1 – Spin Fin vs Pipe Pile Results
Oak Bluffs Design Phase Test Program

	16” Spin Fin® Pile	16” Closed Ended Pipe Pile
Penetration (feet)	25 - 35	50
Ultimate Compression	120 - 155	135 - 155
Ultimate Tension Capacity (tons)	55 - 80	55 - 63