STATE OF PRACTICE AND RECENT ADVANCES 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.

At the same time, 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 the 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

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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.

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 things 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 almost exclusively 50 ksi, 2) better investigation and classification of rock which was traditionally reserved for drilled deep foundations, and 3) the routine use of high strain dynamic pile testing for increased QC (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 another level of confidence when pushing the design loads higher and 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.

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 shows 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 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 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 the required capacity due to large system quake.

In many cases, particularly for pipe piles, the pile becomes theoretically undrivable when simultaneously the blow count is at refusal and the driving stresses exceed the allowable limits. In some cases a different hammer may solve the problem. However, in other cases the load may have to be downgraded or the pile section upsized in order to allow the pile to become “drivable”. Figure 2 relates the drivability of a long steel pile (blow count and compressive driving stresses vs capacity) considering variable energy. For this pile, a capacity of 900 kips makes the pile undrivable as the blow count is around 180 bpf and the driving stresses are over 50 ksi. The pile section would have to be increased or the capacity downgraded.

By using a lower capacity of 700 kip for the pile, the transferred energy of 65 kip-ft is too high because the driving stresses are over 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 below 40 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, controls the design. The higher loads have necessitated larger hydraulic hammers and bigger static load frames (see Figures 3 and 4). Two examples of pile load tests for HP14x117 in Boston Argillite are presented in Figures 5 and 6 for soft and hard rock, respectively. The hard rock was able to generate an ultimate capacity of over 1300 kips, as exhibited by the elastic behavior of the load-deformation. The pile displaced a lot during the test since it was 200 feet long with significant rebound, but the permanent set of 0.5 inches was low. In contrast, the soft rock could not generate more than 800 kips in resistance, where the plot shows significant creep at the pile tip.

Figure 7 shows the PDA results for a pile driven through weathered rock on a different project in Boston. Hydraulic or diesel hammers capable of delivering over 100 kip-feet of transferred energy measured by the PDA are commonly used to drive these piles (see left plot of Figure 7). As a result of the significant energy delivered to the piles, they penetrate through the weathered rock with relative ease and at relatively low blow counts of

5 to 6 blows per inch. In this case the weathered rock could not be 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). At the very end of driving, the capacity and stresses jump up due to the pile tip encountering hard rock.

Figure 8 illustrates a case where two static load tests were performed on a single 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 excessively creep at around 445 tons. The pile did not achieve the required ultimate capacity of 960 tons, so 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 the same load as the first test. This demonstrates that the resistance in the rock is limited in this case to around 450 tons, even when redriven with substantially more energy.

EXAMPLES - INNOVATIVE PILE TYPES

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 Boston 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 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 welded over the lower 5 feet of the pile on a slight angle that helps increase both compression and uplift capacity.

The Tapertube piles were selected for a recent project with a sand profile based on their ability to generate more resistance than driven pipe or drilled pipe piles at the same depth. Some photos of the Tapertube piles are presented in Figures 9 and 10. These piles were driven with the same conventional diesel hammer as their counterpart pipe piles. The static load test results in Figures 11a and 11b illustrate the Tapertube pile develops over 20% more capacity than the pipe pile at the same depth, thereby allowing the project to reduce the number of piles (Hamblin et al., 2018).

Spin Fin piles can generate significantly more uplift and compression resistance than the equivalent counterpart pipe piles. Spin Fin photos are shown in Figures 12 and 13. 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 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).

CONCLUSIONS

Driving high capacity H-piles may be unprecedented in an area considering local rock conditions. 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. If a pile cannot be driven to the required capacity due to possibly one or more of the reasons described earlier, then the owner should be willing to carry this risk and allow for a contingency, such as bigger section, reduced load, more piles, etc. This should be factored into the project philosophy.

Spin Fin and Tapertube piles are used more routinely in other parts of the country. If a site consists of medium dense sands and dense soil is fairly deep, these innovative pile types become more economical and technically superior to traditional straight pipe. Once these pile types save a penetration of around 20 to 25 feet or more, they become more cost effective.

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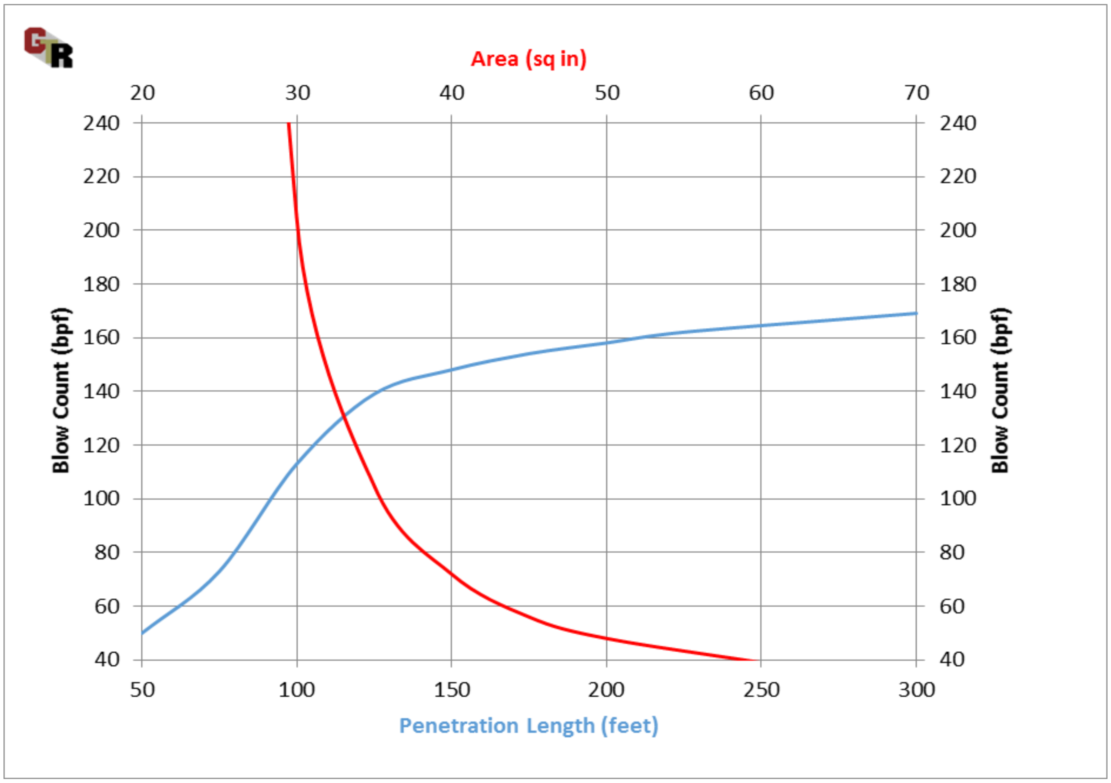


Figure 1 – Pile Drivability - Area and Length vs Blow Count

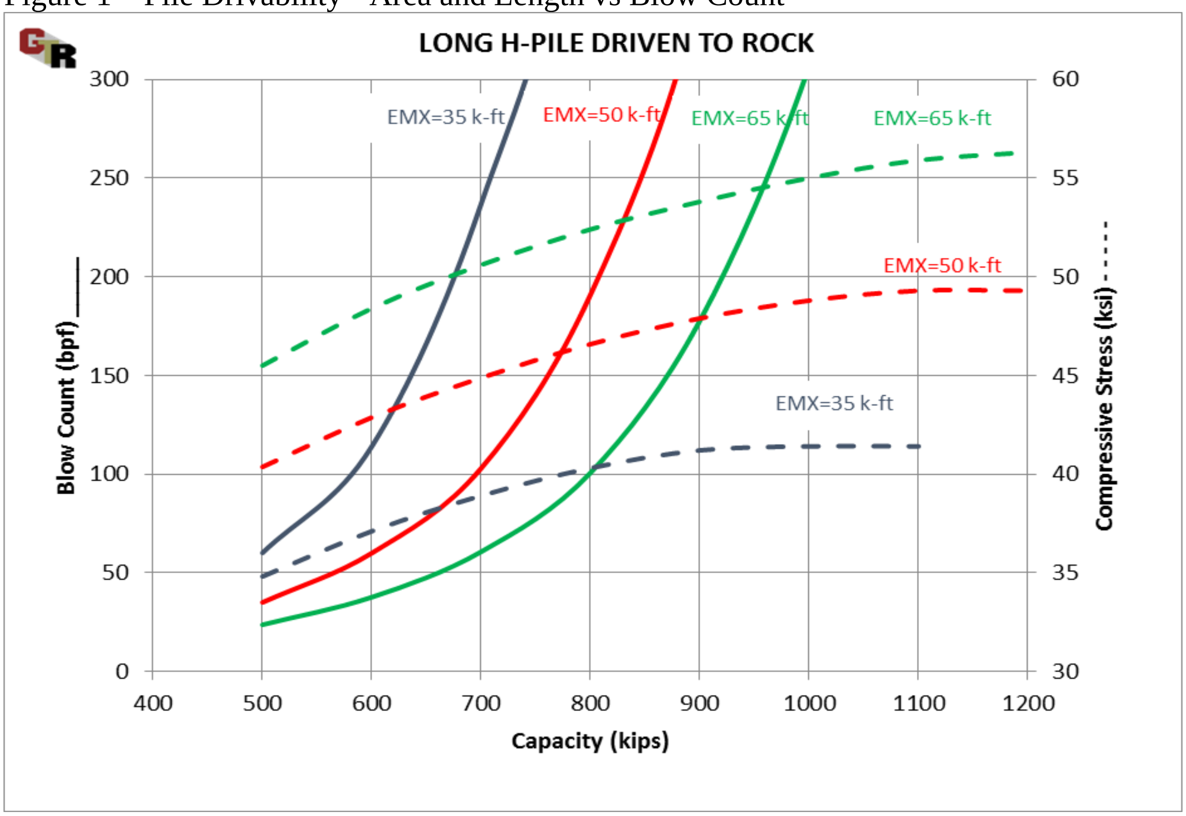


Figure 2 – Pile Drivability – Blow Count and Stress vs Capacity with Energy Variation

Figure 3 –Static Load Test Frame For Large Load

Figure 4 – Big Hydraulic Hammer

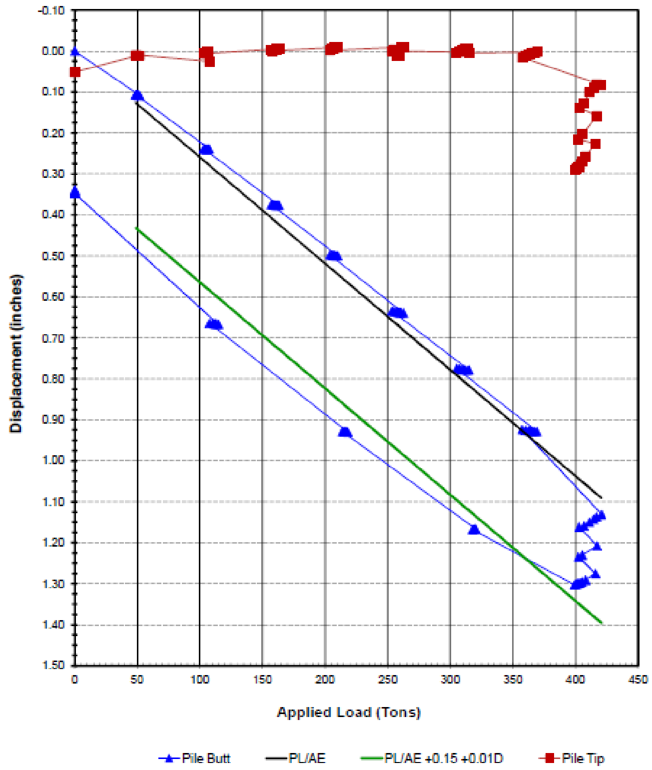


Figure 5 Soft/Weathered Boston Argillite

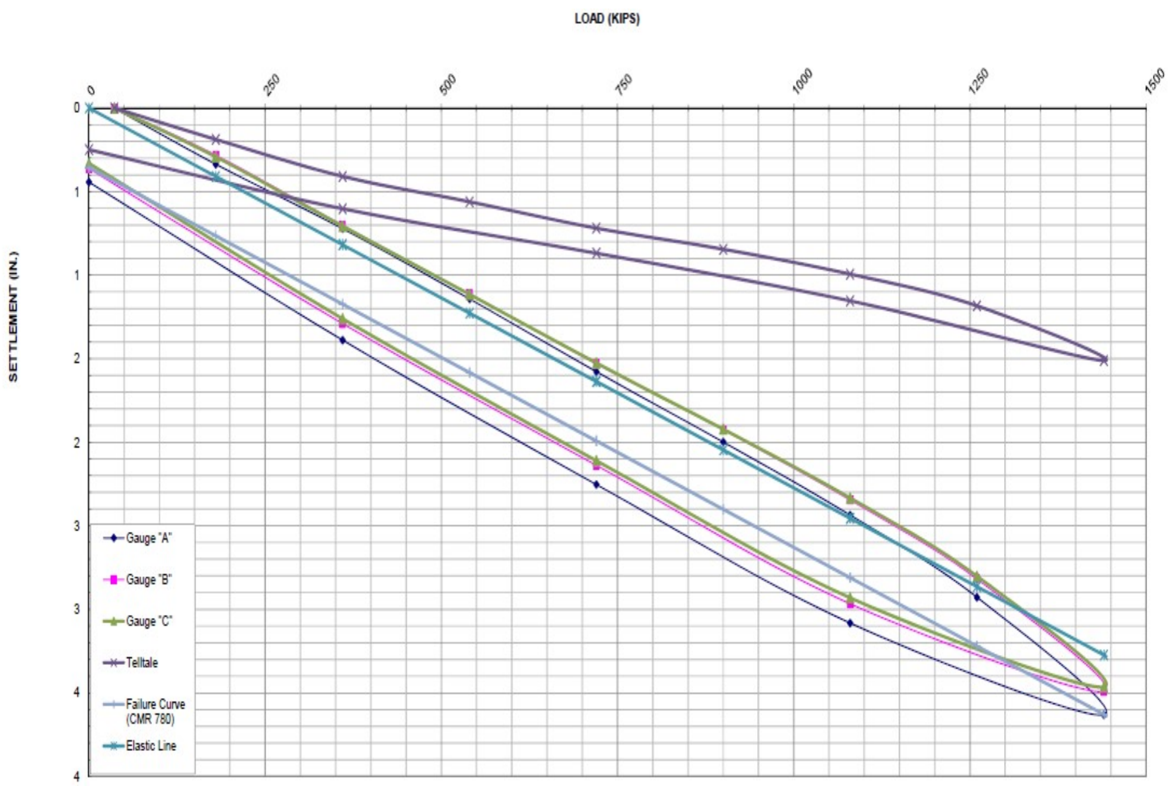


Figure 6 Hard Boston Argillite

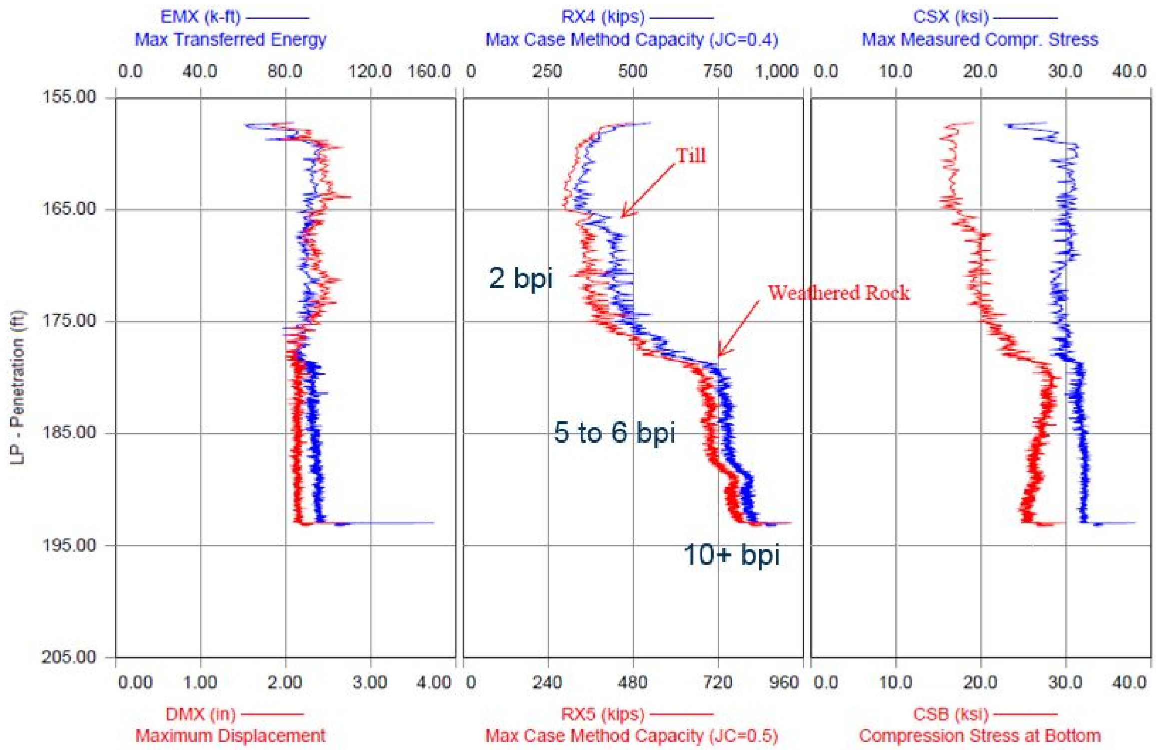


Figure 7 PDA Plots of Driving Through Weathered Argillite

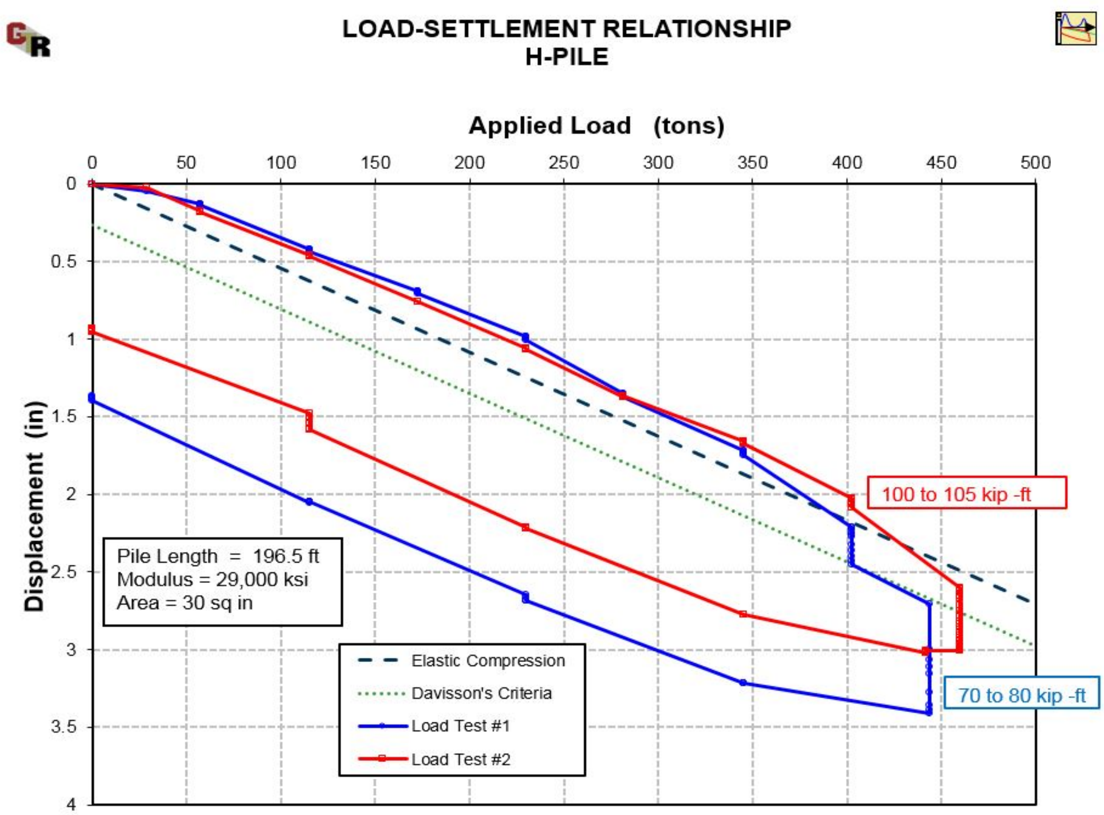
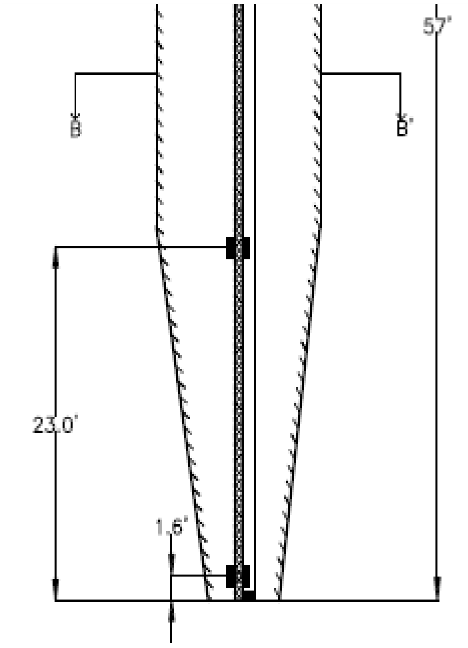


Figure 8 Load Test Results in Weathered Argillite.



Figures 9 and 10 – Tapertube Piles and Closeup

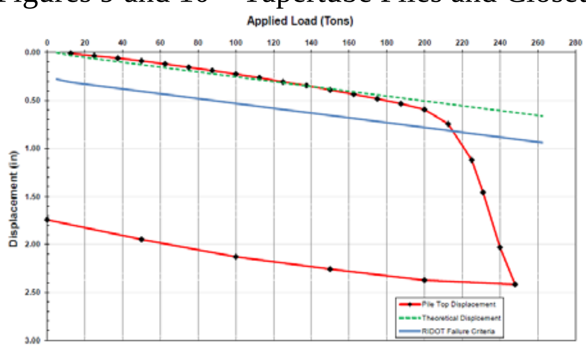


Figure 11a – Tapertube Pile SLT

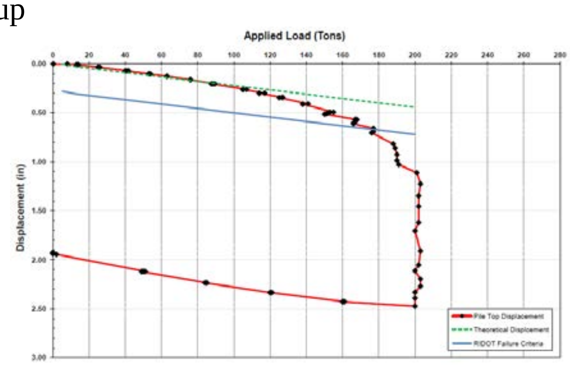
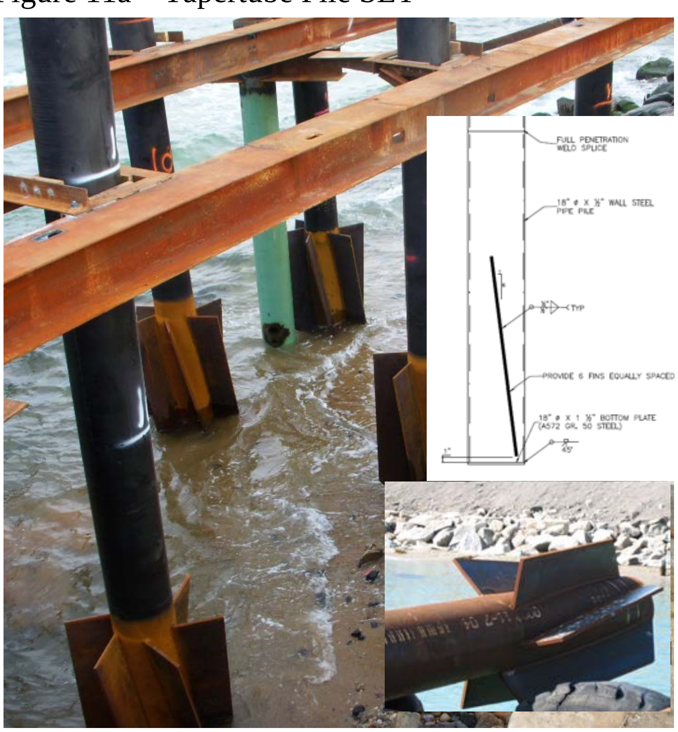
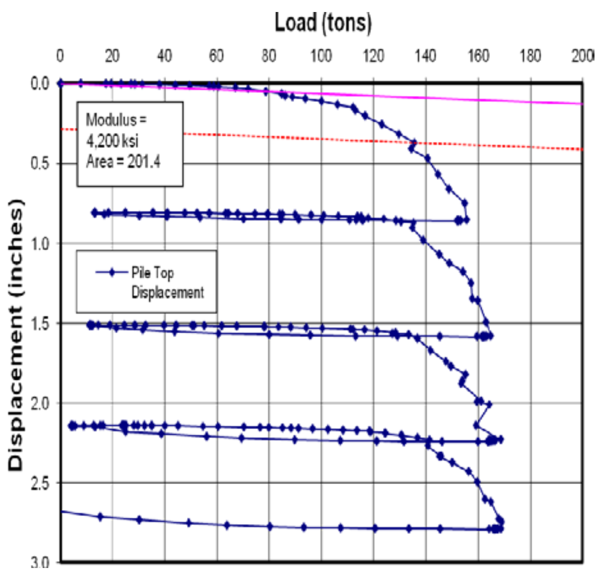


Figure 11b – Pipe Pile SLT



Figures 12 and 13 – Spin Fin Piles and Closup



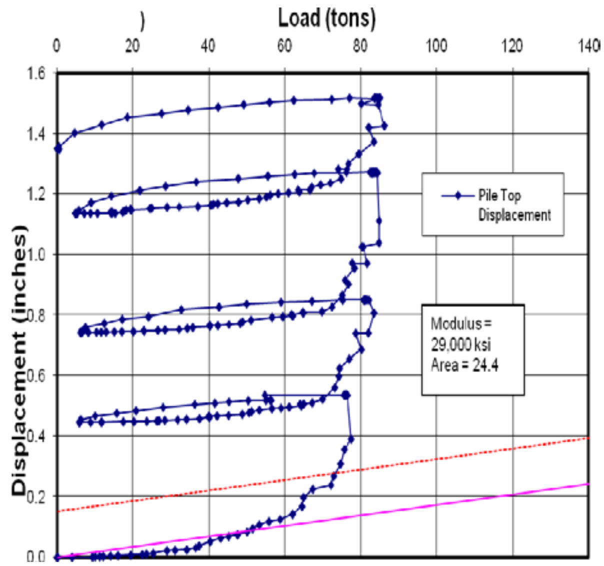


Figure 14a – Spin Fin Static Cyclic Compression Figure 14b - Spin Fin Static Cyclic Tension

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

Table 1 – Spin Fin vs Pipe Pile Results