Foundations for Elevated Ground Mount Solar Panel Photovoltaic (PV) Systems

by

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1.0 SUMMARY

This Report presents the results of research performed to evaluate the behavior of different foundation systems that might be suited for the economic support of elevated ground-mount Photovoltaic (PV) solar panel systems. A large variation exists in suitable foundation types that might be applicable for ground-mount PV systems. The work presented in this Report was conducted between 2012 and 2018 in collaboration with Dr. Stephen Herbert of the Department of Plant and Soil Science at the University of Massachusetts, Amherst, Ma. Field load tests of different foundations were performed at a number of sites in western Massachusetts. The sites included both coarse-grained soils, predominantly sandy materials, and fine-grained soils, predominantly clays and included sites with no water table within the zone of the foundation and with water table near the ground surface.

The work presented in this Report is based largely on field installations and load tests performed on foundations at four principle sites in Amherst, Hadley and South Deerfield, Ma. on property owned by the University of Massachusetts. The evaluation of different types of foundations should consider not only the load bearing characteristics but also the relative economics, consisting of estimated cost of materials, estimated cost of installation and cleanup and other factors, such as subsurface conditions and potential damage to existing ground surface during construction. The advantages and limitations of each type of foundation system are discussed at the end of each section describing the foundations and an overall summary is presented at the end of this Report. The work presented in this Report is predominantly related to fixed PV installations and not tracker type systems.

2.0 INTRODUCTION

Throughout the U.S and other parts of the world, ground-mount PV solar systems have become increasingly popular alternatives to roof mount systems. These systems are being used for large scale commercial installations as well as single home owner installations to provide alternative energy for operating electrical systems. Many PV systems are roof mounted on existing or new structures, however ground-mount systems are also becoming more popular to avoid issues with mounting PV panels to a roof.

Ground-mount PV solar installations are typically one of two types: 1) low-level ground-mount systems that are placed as close to the ground as possible, as illustrated in Figures 1 and 2; and 2) elevated systems that provide sufficient space between the base of the panel frame and the ground surface to allow dual use of the ground beneath the system, as illustrated in Figure 3. Elevated ground-mount systems are attractive in some areas and allow for the ground beneath the system to be used for grazing of livestock, light agricultural activities, such as production of hay and straw and for small plots of vegetables to supplement truck farming activities; in effect, dual land use.

Figure 1. Typical Low-Level Ground-Mount PV Solar System. (Mountain Farms Mall – Hadley, Ma.)

A common misconception among designers of foundations for ground-mount PV systems is that the foundation must be designed to withstand combined loading from compression and bending which produces lateral load in the foundation. In fact, in most cases, the critical design loading is produced by wind which tends to produce uplift forces on the panel array and is accentuated when the panels are elevated and placed closely together. Most systems are constructed of relatively lightweight materials so that axial compression is generally of little concern. In terms of design issues with lateral loads, wind loading is generally directionally random and the performance of the system is not dependent on small lateral movements to remain operational.

Wind loading on the elevated panel system produces uplift forces that must be counteracted by the tension (uplift) capacity of the foundation (McBean 1985; Chevalien & Norton 1979; Shademan & Hangan 2009; Bitsuamlak et al. 2010). The resistance to uplift may be provided by the dead load or weight of the foundation, as in the case of cast-in-place concrete piers, or it may be provided by sliding shear resistance between the soil and the foundations, as with a driven pile, or a combination.

Figure 2. Low-Level Ground-Mount PV Solar System.
(Town of Hadley – Hadley, Ma.)

Figure 3. Typical Elevated Ground-Mount PV Solar Panel Array.
(UMass Agricultural Farm – S. Deerfield, Ma.)

3.0 FOUNDATION ALTERNATIVES FOR ELEVATED GROUND-MOUNT SOLAR PANEL SYSTEMS

Foundation options for ground mount solar systems can be divided into several groups based largely on the method of installation. They fall into 3 basic categories; 1) drilled or excavated cast-in-place or precast concrete pier (drilled shaft) foundations; 2) cast-in-place or precast concrete slabs; and 3) driven, vibrated or screwed steel pile foundations. Groups of driven piles or helical piles may also be used as alternative foundations. At some locations the ground conditions may not allow any of these types of installed foundations, in which case it may be necessary to use precast concrete blocks as a ballasted system to support the panels. The following foundation types are considered in this Report:

1. Straight-Sided Drilled Cast-in-Place Concrete Piers
2. Enlarged Base Drilled Cast-in-Place Concrete Piers
3. Over-Drilled and Backfilled Cast-in-Place Concrete Piers
4. Over-Drilled and Backfilled Precast Concrete Piers
5. Cast-in-Place Footing
6. Precast Concrete Slab
7. Driven/Vibrated Piles
8. Helical Piles
9. Ground Screws
10. Pile or Micropile Groups
11. Precast Ballasted Blocks

Figure 4 illustrates these different groups of foundations. Within each of these groups there are options that can be selected depending on the site specific conditions as will be discussed in the following sections. It is important to note that the various foundation alternatives illustrated in Figure 4 represent more-or-less traditional foundations used throughout the construction industry for support of other types of structures and are not restricted for use in supporting PV systems. They do not represent any unique or proprietary system that is exclusive to the PV solar industry. This means that Contractors should generally be familiar with the requirements for construction for all of the possible foundation systems.

The choice of a particular foundation system that might be used at a site will depend on a number of factors, including:

1. Design Uplift Load Requirements
2. Site Soil Conditions
3. Site Groundwater Conditions
4. Size of the Proposed PV System
5. Site Accessibility
6. Availability of Materials
7. Availability of Contractor
8. Total Cost
9. Construction Schedule
10. Local Building Code Requirements

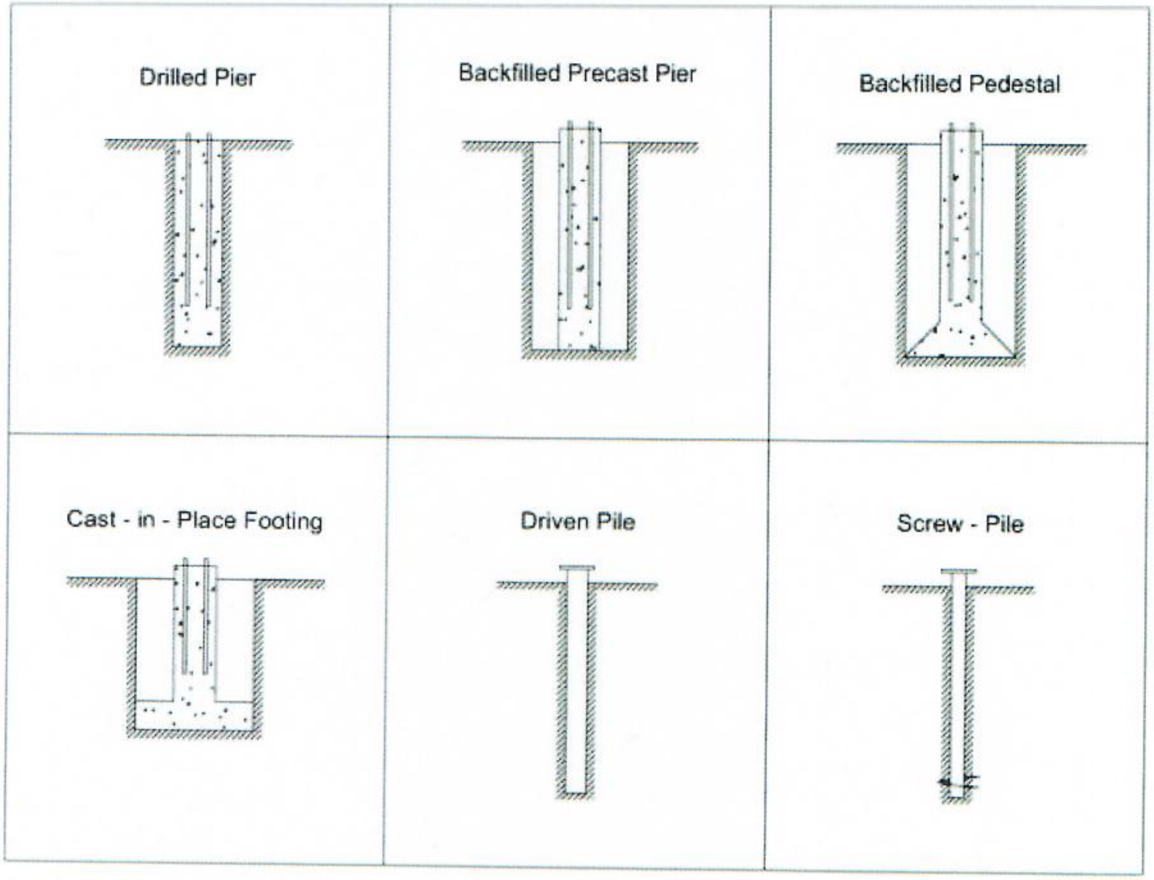

Figure 4. Primary Categories of Typical Ground-Mount Solar Panel Foundations.

3.1 Drilled Cast-In-Place Concrete Piers

A common foundation system that has been in use since the 1940s to support a variety of structures is a drilled cast-in-place concrete pier, which is often referred to as a drilled shaft. As the name implies, the foundation is constructed by drilling an open borehole using a truck or track, or tractor mounted drilling rig, producing a hole on the order of 12 in. to 36 in. in diameter. Larger size piers are available for larger loads. A hole is drilled and the soil removed and must be removed from the site after construction is complete.

Drilled piers are most suited to stiff and very stiff soils that will maintain an open hole while the construction proceeds and for sites where the water table is not close to the ground surface. Lengths of drilled shafts can be greater than 100 ft. but for PV solar systems are typically on the order of 6 to 12 ft. Any soils that have a tendency to cave, such as loose sands or very soft clays are not well suited for shallow drilled piers for supporting solar panels. Figure 5 shows a schematic of a typical PV system supported by a drilled concrete pier.

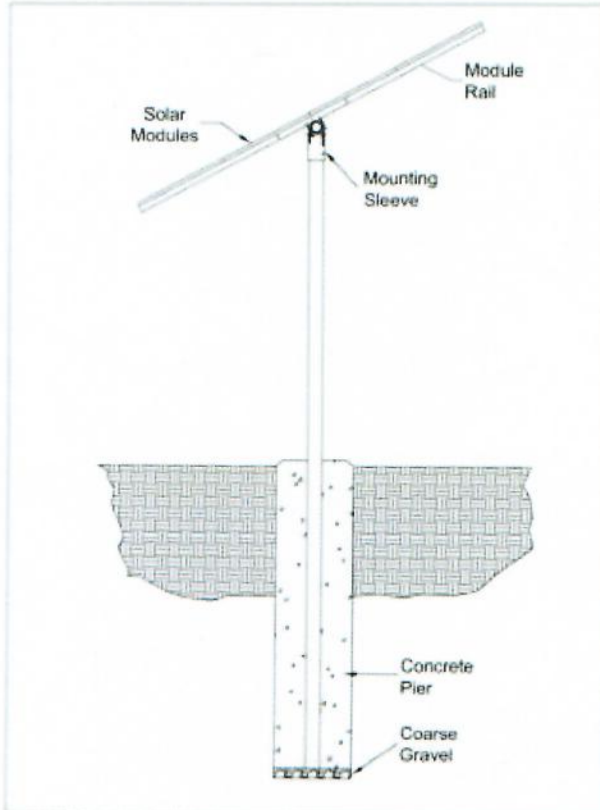

Figure 5. Schematic of Completed Concrete Pier Foundation and Solar Panel.

The construction sequence for a drilled cast-in-place concrete pier foundation consists of the following steps:

1. Drill an open hole of appropriate size;
2. Place a steel reinforcing cage in the hole;
3. Place concrete in the hole;
4. Place a short section of sonotube (construction form) at the ground surface extending about 0.5 to 1 ft. above the ground; and
5. Place a connection plate into the concrete for attaching the solar panel column. Alternatively a steel pipe or steel square tube may be placed by direct embedment in the concrete immediately after or while concrete is placed instead of a mounting bracket as a direct mount for the solar panel frame.

This sequence of construction is shown in Figures 6 to 9.

Figure 6. Excavating a Hole for a Drilled Pier Using a Truck-Mounted Drilling Rig.

Figure 7. Excavating a Hole for a Drilled Pier Using a Truck-Mounted Drilling Rig.

Figure 8. Sonotubes Set For Placing Concrete.

Figure 9. Direct Embedment Mounting Posts Set in Concrete.

Drilled and cast-in-place piers are routinely used to support a number of other types of structures to resist both axial compression and lateral loads. They are relatively easy to install using a specialty rig fitted with different size and type of drilling tools to suit the local ground conditions. For small scale solar installations they appear to be a popular alternative since the depth of drilling is generally shallow (typically less than about 10 ft.). Typical pier diameters range from about 18 in. to 30 in. Figures 10 to 12 show some examples of completed PV installations supported on drilled cast-in-place concrete piers.

Figure 10. Drilled Concrete Pier Supported PV System - Sunderland, Ma.

Drilled pier foundations may be straight sided, drilled with an oversized enlarged base, or they may be constructed an Pressure Injected Footings (PIFs) which is an alternative method of creating an enlarged base as will be described later in this Report.

Figure 11. Drilled Concrete Pier Supported PV System - Putney, Vt.

Figure 12. Drilled Concrete Pier Supported PV System, Leverett, Ma.

3.1.1 Design

As with other types of foundations for PV solar systems the primary design load to be considered is the uplift or pullout load. For drilled cast-in-place concrete piers, uplift capacity is developed from a combination of side resistance developed from the shear strength between the native soil and the sides of the pier shaft and the mass of the pier. The design is dependent on the existing in situ ground conditions. The load capacity of drilled piers is provided by a combination of the mass of the concrete and by the side resistance between the perimeter of the shaft and the surrounding soil.

$$Q_{TOTAL} = Q_{MASS} + Q_{SIDE} \quad [1]$$

where:

Q_{MASS} (lbs.) = 3.14 x $(D/2)^2$ x L X 150 lbs./ft^3
D = Pier Diameter (ft.)
L = Pier Length (ft.)
Q_{SIDE} = 3.14 x D x L x F
F = Unit Side Resistance Between Soil and Concrete (lbs./ft^2)

The influence of soil composition on the uplift behavior of drilled piers is shown in Figure 13 which shows a comparison between the load-displacement behavior of the same diameter and length of shaft in a stiff clay (Hadley) and a silty sand (South Deerfield).

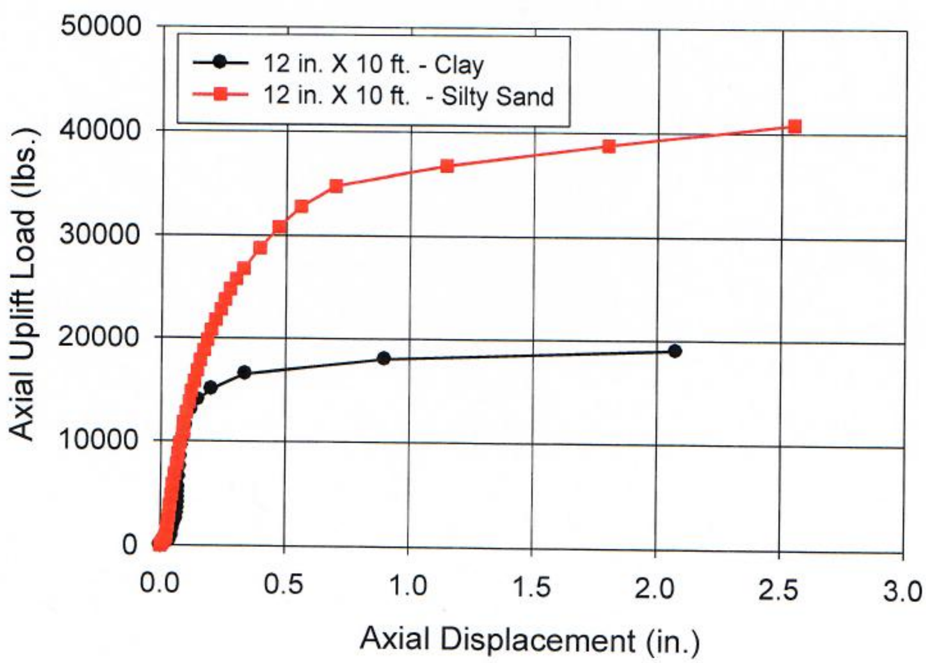

Figure 13. Load-Displacement Behavior of the Same Size and Length Drilled Concrete Piers in Clay and Silty Sand.

These results clearly demonstrate that the load response of a drilled concrete pier depends on the soil type in which it embedded. Clays tend to give lower load capacity. In both cases it also shows that once the side resistance is overcome, the foundation cannot withstand any additional load without large additional uplift displacement.

Figures 14 and 15 show load-displacement behavior of different size drilled concrete piers in Clay and Silty Sand. It can be seen that the diameter and length of the shaft influences the maximum uplift capacity.

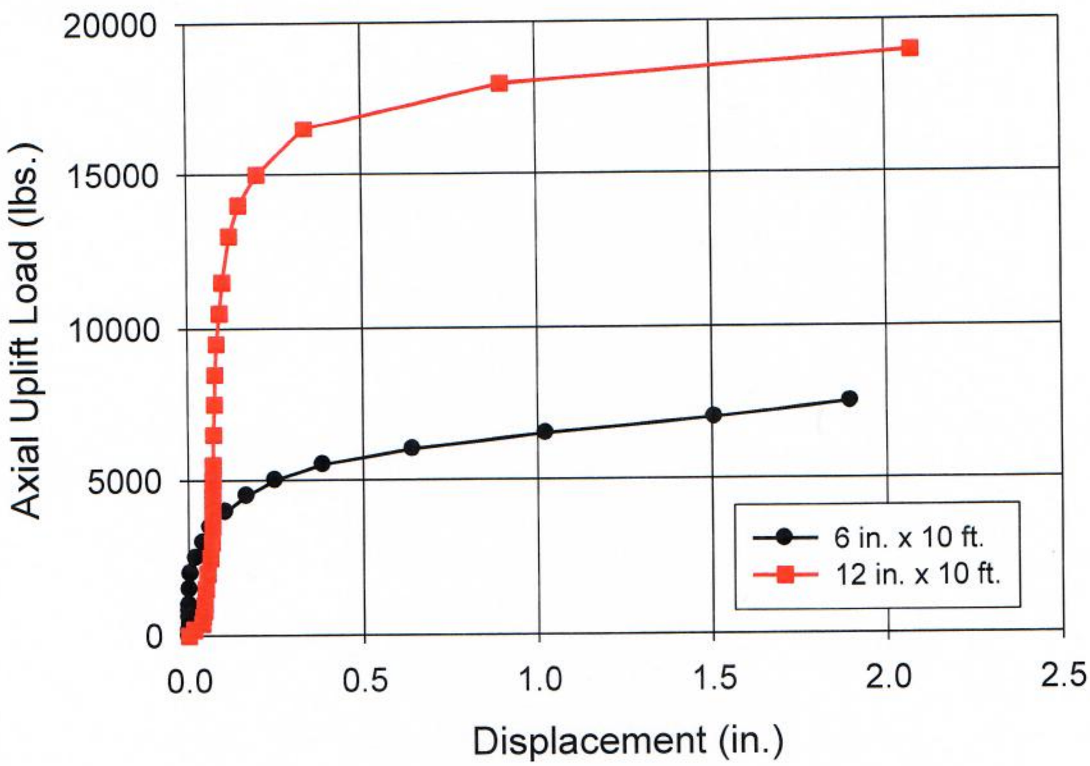

Figure 14. Comparison of the Uplift Behavior of Different Diameter Drilled Concrete Piers of the Same Length in Clay – Hadley, Ma.

Figure 14 shows that the load behavior increases as the diameter of the pier increases since the surface area in contact with the soil along the perimeter of the pier increases as the diameter increases. In some cases, disturbance of the soil from the drilling operation may reduce the available shear strength of the soil that can be mobilized in uplift loading.

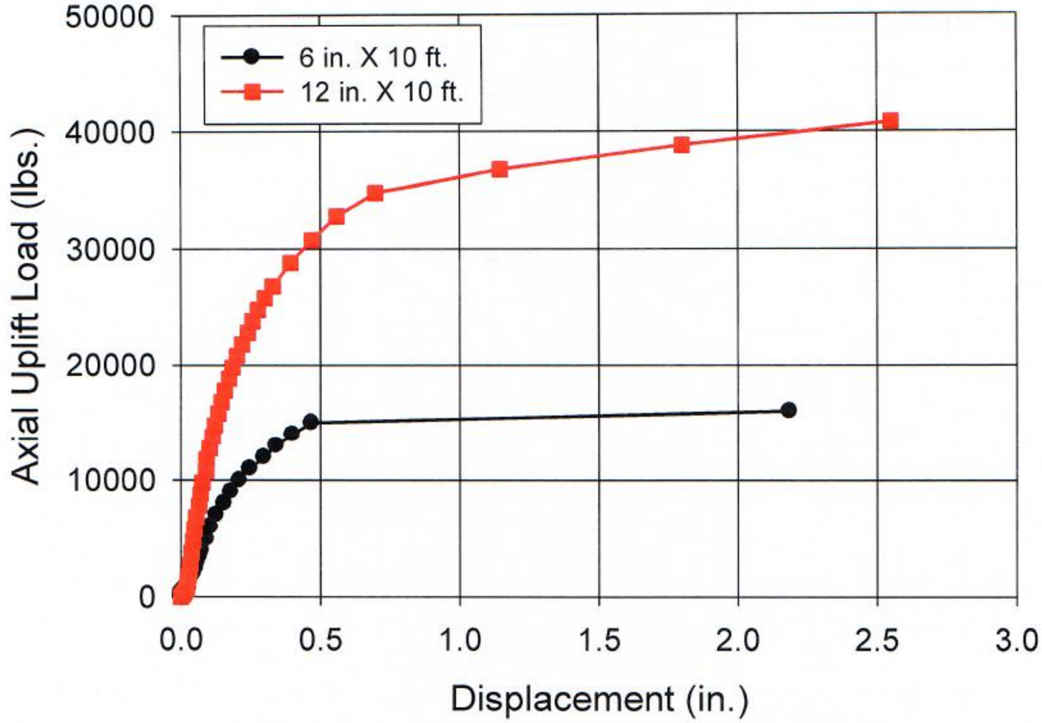

Figure 15. Comparison of the Uplift Behavior of Different Diameter Drilled Concrete Piers of the Same Length in Silty Sand – UMass AgFarm, S. Deerfield, Ma.

Figure 15 shows similar results in the Silty Sand as compared to the Clay from Figure 14. The load behavior increases as the diameter of the pier increases.

3.1.2 Advantages

Drilled cast-in-place concrete piers are relatively simple to construct and can often be drilled using an auger with a suitable extension mounted to ether the front or rear of a tractor, eliminating the need for a specialty contractor, as shown in Figure 16. Alternatively, the hole may be drilled using an auger attachment mounted to a skid steer, as shown in Figure 17 or a compact excavator, as shown in Figure 18. Provided that the hole stays open, concrete can be poured directly into the drilled hole. Typically, a small rebar cage is installed but in many cases a steel pipe or steel H section may be inserted into the concrete to act as a support for the solar panel rack and panels. Only concrete with a minimum compressive strength of 3000 psi (preferably 4000 psi) should be used.

Figure 16. Drilling a Concrete Pier Hole Using an Auger Mounted on a Tractor.

Figure 17. Drilling a Concrete Pier Hole Using an Auger Mounted on a Skid Steer.

Figure 18. Drilling a Concrete Pier Hole Using an Auger Mounted on a Compact Excavator.

3.1.3 Limitations

The principle issues with drilled piers in this application is that the construction can be slow and messy relative to other options and the drilling produces large amounts of soil cuttings that must be disposed of; an added expense to the project. Access to some sites for the drilling equipment and the concrete delivery may be an issue. The placement of the concrete is generally straightforward but often requires a few days for the concrete to harden sufficiently to begin above ground installation. Connection to the support structure may be made by a bolted plate at the top of the pier or in some cases direct embedment of the support from into the concrete is an option. Drilled shafts normally cannot be used in sands that cave.

3.2 Enlarged Base Drilled Cast-in-Place Concrete Piers

An enlarged base drilled cast-in-place pier consists of a straight shaft pier, as previously described, with an enlarged section at the base. A comparison between a straight shaft and two different types of enlarged base drilled piers is shown in Figure 19. Two types of enlarged bases are shown, one that is constructed using a special drilling tool that is capable of enlarging the hole at the bottom and one that is constructed by hammering the concrete to create an enlarged "bulb". The main advantage of using a drilled concrete pier with an enlarged base is that there is usually a very large increase in uplift load capacity provided by the enlarged base in most soils. This means that they may be more appropriate for cases where the near surface soils are weak.

Pedestal style piers or footings are often used as shallow foundations for solar panel systems, transmission towers and other structures to resist uplift loading. In some cases shallow pedestal footings may be constructed as an enlarged base drilled shaft using conventional foundation drilling equipment and a belling tool to create the enlarged base or they may be constructed as a Pressure Injected Footing (PIF) using dynamic impact to create an enlarged end, as shown in Figure 1. These construction techniques create a foundation element in in situ soil that derives capacity from properties of the native host soil.

In many cases, shallow piers or pedestals act as shallow foundations with the embedment ratio H/D_b typically less that about 3. Kulhawy (1985) presented a discussion of the design approach for shallow spread anchors and included anchors constructed in both neat excavations and in overexcavations. In both situations the properties of the backfill, as compared to the host soil, are important and in most cases controls the design.

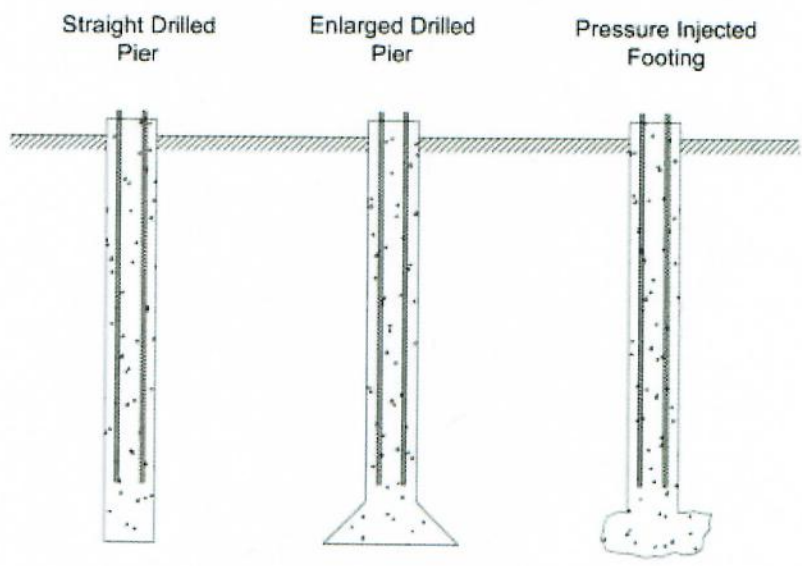

Figure 19. Comparison of Straight Shaft and Enlarged End Drilled Pier Foundations.

If the soils are very strong, for example stiff to very stiff clay, an enlarging tool works well to create a "bell", except that all of the soil cuttings from the drilling of the bell must be removed before placing concrete. A special tool may be required for this operation. Once the bell has been created and cleaned, concrete and steel reinforcement are placed as with a straight shaft drilled pier.

Enlarged base drilled shafts are used to reduce the length of drilling required and shift a portion of the developed load capacity in uplift from the straight sides of the shaft to the enlarged section. Figure 20 shows an idealized schematic of an enlarged base drilled shaft. In some locations an enlarged base is attractive since it can create considerable additional resistance to uplift over a simple straight sided shaft depending on the soil conditions. However, construction can sometimes be difficult and this method is only suited to soils that will not cave as the enlarged base is constructed (generally stiff to medium stiff clays and other hard fine-grained soils).

If the soil cannot maintain an open hole for even a short period of time, an alternative technique is to drill the hole, place an initial amount of dry very low slump concrete into the hole and then use a drop hammer to drive the concrete down and outward to create a bulb. After the bulb is created, concrete is then placed in the shaft as before. This type of shaft is often called a Pressure Injected Footing, or PIF. Figure 21 shows the general construction sequence used in creating a PIF.

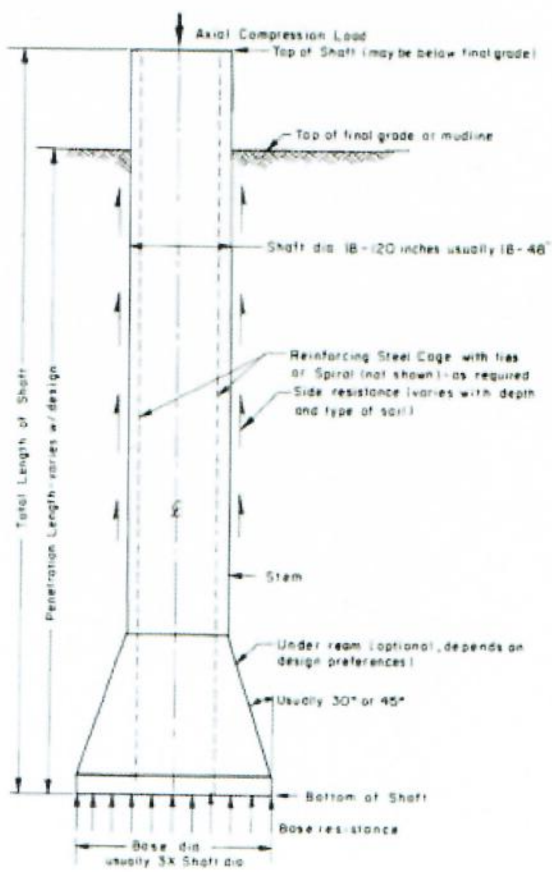

Figure 20. Idealized Enlarged Base Drilled Pier.

3.2.1 Design

The design of enlarged base drilled shafts is performed as:

$$Q_{ult} = Q_S + Q_E + W \qquad [2]$$

where:

Q_{ult} = Ultimate Uplift Load Capacity
Q_S = shaft side resistance
Q_E = bell or bulb "end" bearing
$Q_E = [\pi R_E^2 - \pi R_S 2]q_{ult}$
R_E = radius of the bell
R_S = radius of the shaft

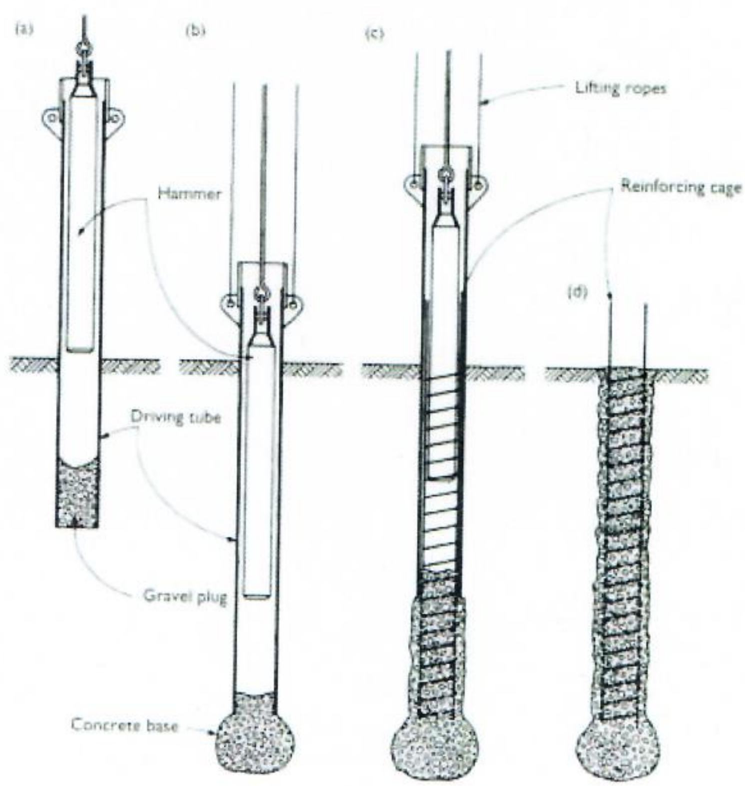

Figure 21. Construction Sequence in Creating a Pressure Injected Footing (PIF)

3.2.2 Advantages

The primary advantage of using an enlarged base drilled pier over a straight sided pier is that the total length can usually be reduced, since the enlarged end develop relatively high load capacity. The actual capacity depends on the specific geometry of the shaft and base.

3.2.3 Limitations

Generally, the limitations of enlarged base drilled piers are similar to those of straight sided drilled pier. In addition, cleaning of the final excavation may be difficult. High groundwater conditions may cause softening and reduction of loaf capacity and difficult construction.

3.3 Over-Drilled and Backfilled Cast-In-Place Concrete Piers

An alternative construction technique to a traditional drilled pier is the create a drilled or excavated cavity and place a pedestal form at the base of the excavation with an attached concrete tube form to create a stem. The space between the stem form and the excavation wall is then backfilled with compacted soil, using either soil native cuttings produced from the excavation or using imported fill, such as granular soil. Recently, it has become cost effective in some areas to use precast pedestal or straight shaft concrete foundations in open excavations, especially for structures such as light poles, overhead signs or small wind generator towers. These different construction options are illustrated in Figure 22.

In situations where the location of a foundation is fixed and the native soil and excavated soil consists of wet clay, the Contractor installing the foundation has a number of options:

1. The Contractor may simply place the excavated soil back into the excavation with minimal compactive effort. This generally would represent a "worst-case" scenario;

2. The Contractor may place the excavated material back into the excavation but attempt to compact the backfill as best as possible. This represents a better operation but "compactibility" may be restricted by the water content of the clay;

3. The Contractor may recognize or be told that the excavated soil is too wet to allow good compaction and may attempt to increase the "compactibility" by drying the soil out before placing it in the excavation and providing good compaction. This assumes that the Contractor has the luxury of allowing the excavated soil to dry, which often may not be the case;

4. The Contractor may remove and replace the wet excavated soil with what seems to be a more appropriate backfill materials, such as a coarse sand, if a supply of material is available; or

5. The Contractor may recognize or be told that the excavated material is too wet and attempt to improve the backfill with an additive such as lime or Portland cement and the place the material back into the excavation for compaction.

Each of these options may have serious implications regarding the behavior of the foundation in uplift. In the present study,

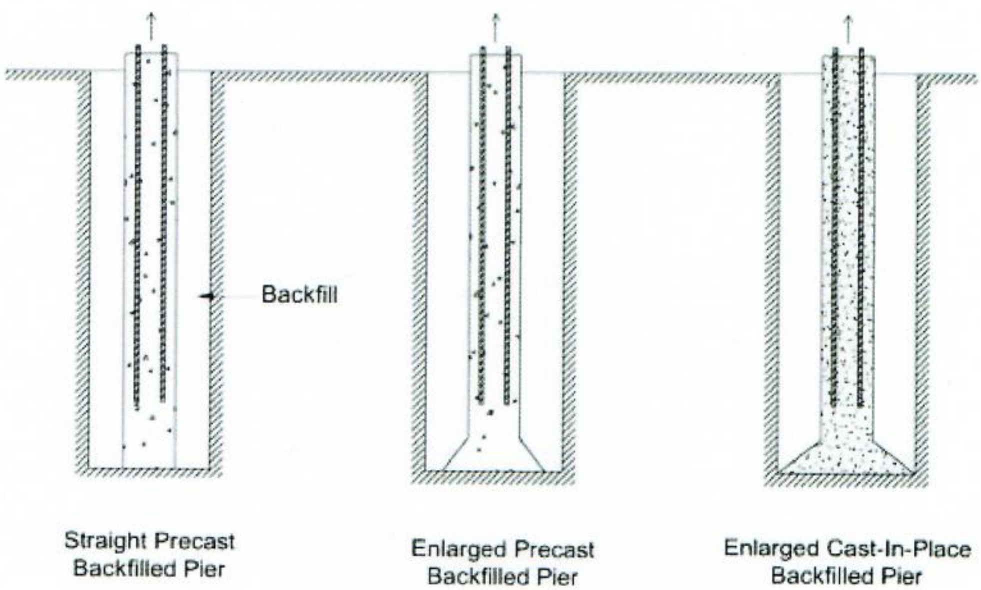

Figure 22. Alternative Construction of Shallow Over-Drilled and Cast-in-Place Pedestal Foundations.

The behavior of excavated enlarged footings also depends on the geometry of the excavation, i.e., whether the excavation is "neat", i.e., the same diameter of the footing base, or whether the excavation is enlarged and has dimensions larger that the footing. The behavior, and therefore the design is also related to the shape of the excavation, i.e., whether the sidewalls are sloped or vertical. A typical construction sequence is shown in Figure 23.

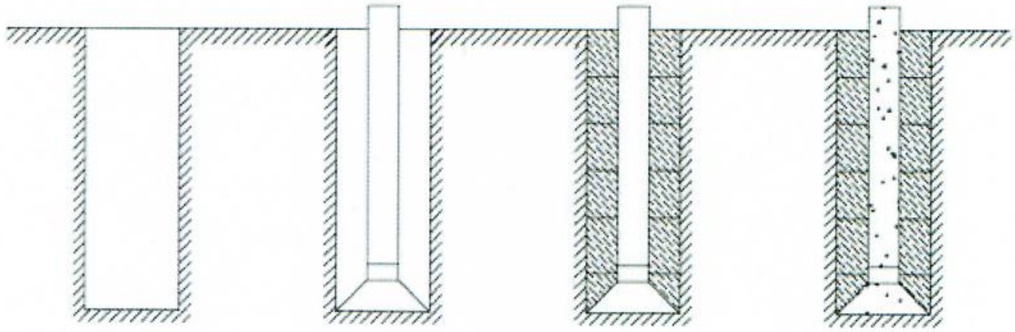

Figure 23. Typical Construction Sequence for Shallow Cast-in-Place Backfilled Pedestal.

Seven uplift tests of full-scale shallow enlarged base overdilled cast-in-place pedestal foundations were performed at a site in Hadley to evaluate the influence of backfill composition and compaction on the uplift behavior. The diameter of the enlarged section was 2 ft. and the diameter of the concrete stem was 16 in. The pedestalled were all constructed to a depth of 5 ft. Figures 24 to 26 show the construction sequence used to create the foundations.

Figure 24. Oversize Drilled Hole for Pedestal Foundation.

Figure 25. Placing an Enlarged Base Form in Ovedrilled Hole for Pedestal Foundation.

Figure 26. Placing and Tamping Backfill Around Overdrilled Pedestal Foundation.

Axial uplift tests were performed on 7 pedestal foundations for different situations as shown in Table 1.

Table 1. Summary of Test Conditions for Overdrilled and Backfilled Pedestal Foundations.

Test No.	Pedestal Number	Backfill Material	Compaction Condition
1	P-1	Native Soil	Worst
2	P-2	Native Soil	Better
3	P-3	Native Soil	Best
4	P-1	Well-Graded Sand	Loose
5	P-1	Well-Graded Sand	Vibrated
6	P-1	Well Graded Sand	Dense
7	P-2	Air-Dried Native Soil	Best

Table 2 gives a summary of the ultimate uplift load for each pedestal, taken as the load corresponding to an uplift displacement of 10% of the base diameter.

Table 2. Summary of Interpreted Ultimate Uplift.

Test No.	Ultimate Capacity (lbs.)
1	7450
2	11550
3	11850
4	9550
5	9150
6	11000
7	15500

Results of load tests using wet native clay backfill for three compaction conditions of poor, better and best are shown in Figure 27. As expected, the backfill with the lowest unit weight, representing poor construction practice, showed the softest response to load and the lowest uplift load capacity. There was no difference however between the better and best compaction efforts, simply because the initial water content of the backfill was too high to allow any significant increase in compacted unit weight with additional compactive effort. The soil would only allow a certain level of compaction to take place before water was being squeezed form the clay.

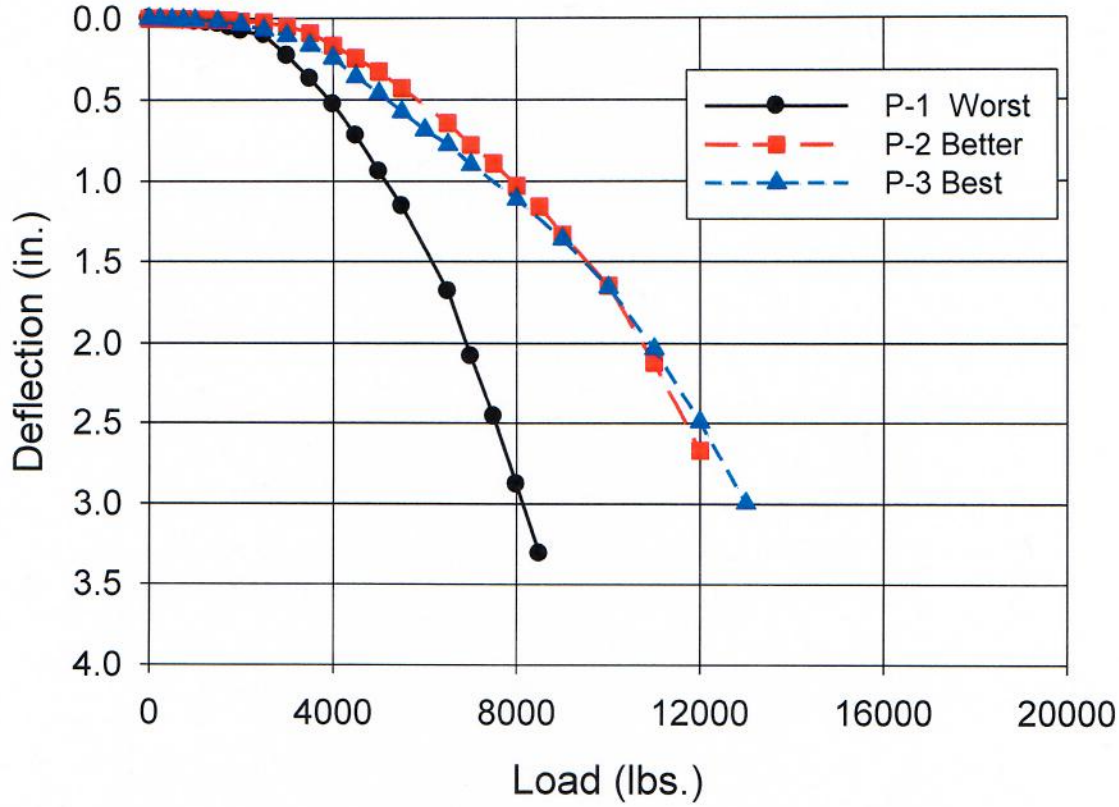

Figure 27. Influence of Compaction Quality on Uplift Behavior of Overdrilled Concrete Pedestal Foundations Using Natural Clay Backfill.

None of the tests show a plunging failure up to a maximum displacement of about 3 in. While no ground surface heave measurements were taken during the load tests, observations showed no obvious vertical movement of the backfill for the loosest backfill, suggesting that the soil immediately above the base of the foundation compressed. In contrast, the tests with the better and best compaction showed radial cracking in the backfill between the pedestal stem and excavation wall and obvious backfill heave, suggesting that the entire mass of backfill moved upward and the failure occurred between the backfill and the native soil. This is consistent with previous observations of Clements (1960) who found that for no compaction of the backfill, the foundation failed in local shear by compression of the soil immediately above the enlarged base and with no surface movement of the backfill.

Figure 28 shows the results of the load tests performed using the same native soil as backfill but placed and compacted after first air-drying the soil to allow higher compacted unit weight. During the placement of this soil, considerable effort was used to compact the soil to the highest possible unit weight with the hand compaction equipment. The results of this test show a much higher ultimate capacity obtained using the dried native clay as backfill. This test also showed considerable upward movement of the backfill around the stem of the pedestal which suggests that all of the compacted soil around the stem was engaged in resisting uplift as compared to only local soil in the zone just above the enlarged base.

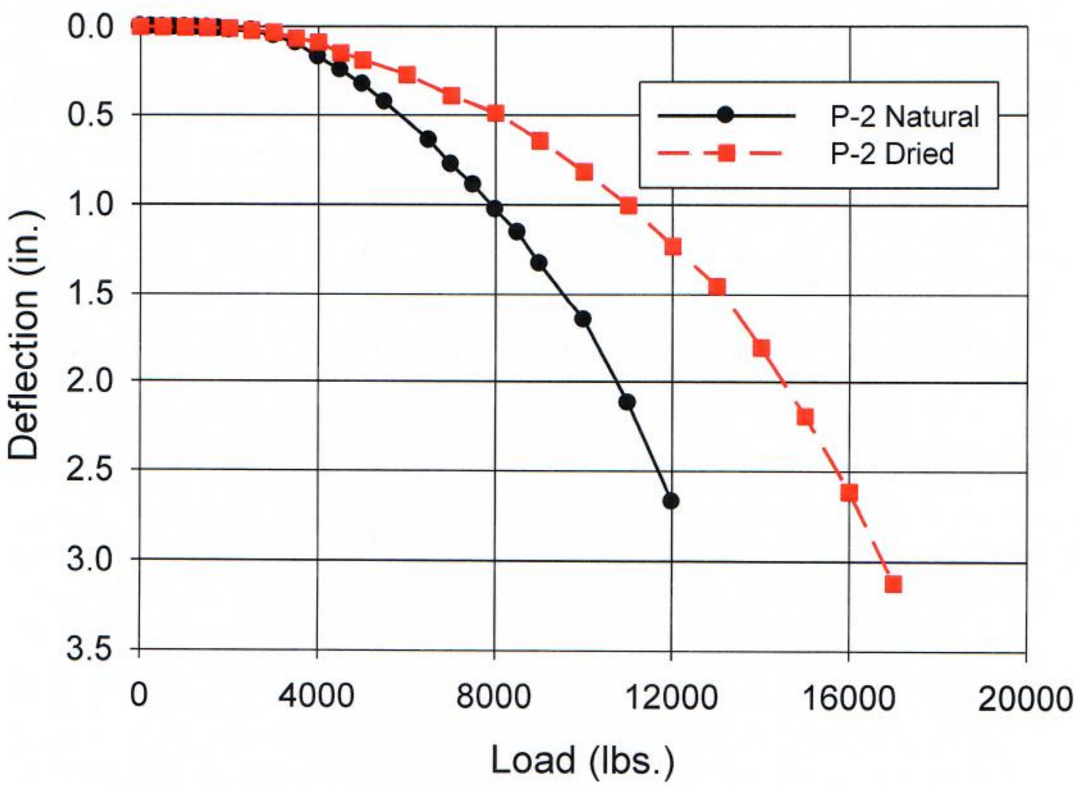

Figure 28. Results of Overdrilled Pedestal Uplift Tests Using Native Wet Clay vs. Native Air-Dried Clay as Backfill.

Another set of load tests was conducted using well-graded medium to coarse sand as backfill. Initially, the sand was placed loose and only lightly tamped. A load test was then performed. After this test, a concrete vibrator was used in the backfill in an attempt to increase the unit weight (e.g., Mirza 1992). After vibration, a second uplift test was performed. Following this test, all of the sand backfill was removed and then replaced but was compacted to a higher unit weight using the hand compaction equipment. Figure 29 shows the results of these three load tests and indicates very little difference between the loose test and the same test after compaction with the vibrator but the dense sand gives a higher capacity.

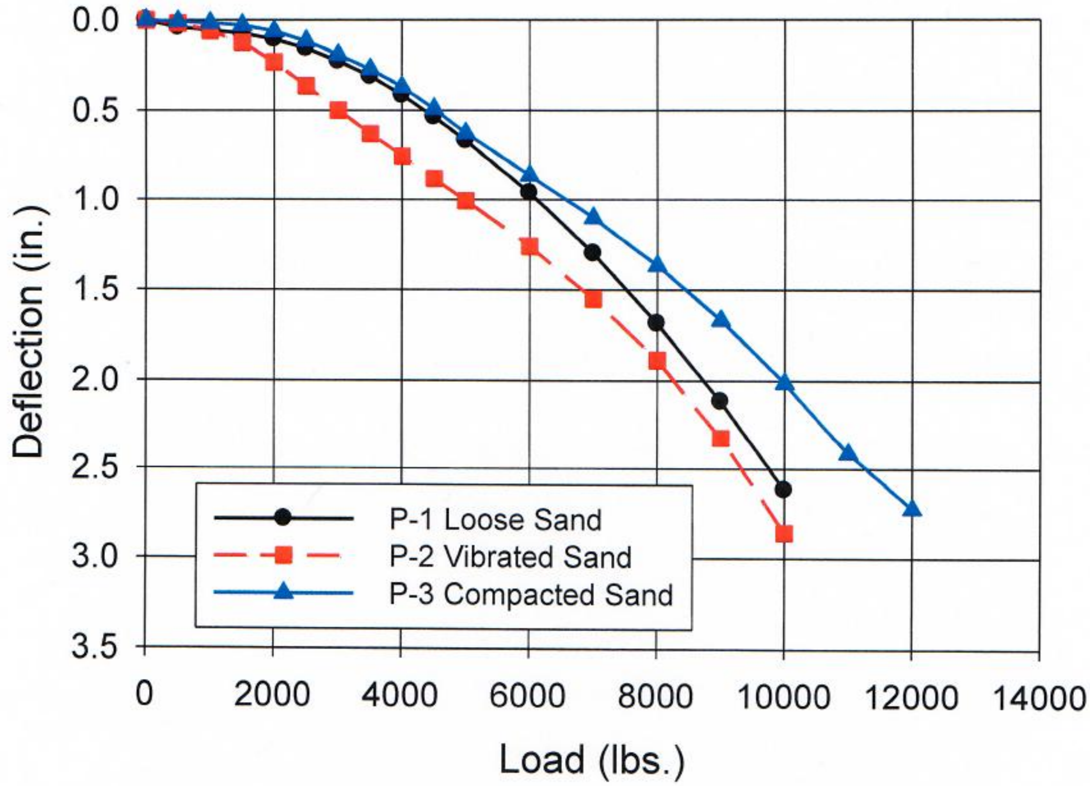

Figure 29. Results of Overdrilled Pedestal Uplift Tests with Sand Backfill.

3.3.1 Design

The design and performance of augered and enlarged base piers or "footings" for transmission towers and other tall structures has previously been reported in some detail (e.g., Balla 1953; Khadilkar & Gogate 1970; Adams & Radhakrishna 1971). In the case of an augered and enlarged base footing, the uplift capacity is derived solely from properties of the native ground into which the footing is placed. This is in sharp contrast to footings that are placed in an excavated or overdrilled hole and then backfilled with native or imported soil.

The influence of backfill composition and compaction on the uplift behavior of excavated and compacted shallow slab and pedestal footings for transmission tower structures has previously been demonstrated (e.g., Clements 1960; Turner 1962; Matsuo 1968; McKenzie 1971; Zmudzinski & Sala 1980) All of these previous investigations have shown that the characteristics of the backfill can have a pronounced influence on the load-displacement behavior and can control the failure mechanism involved in developing load capacity. This was confirmed in the current work and shows that it is important to have a Contractor provide good compaction of the backfill.

3.3.2 Advantages

One of the main advantages to using an overdrilled pedestal foundation is that no special drilling equipment is needed. The hole can be drilled using any piece of equipment or tractor equipped with an auger attachment. The backfill may be selected and the compaction can be controlled so that the backfill around the shaft is strong, even stronger than the native soil.

3.3.3 Limitations

Backfill needs to be of acceptable quality and backfill compaction must be good. Capacity depends on quality of backfill. High groundwater conditions may cause softening and reduction of load capacity and difficult construction.

3.4 Over-Drilled and Backfilled Precast Concrete Piers

An alternative to cast-in place concrete piers is to use a precast factory fabricated concrete foundation. The precast shape can be straight (cylindrical or it may include an enlarged base for added uplift resistance. Figure 30 shows an example of a commercially available precast cylindrical concrete pedestal foundation.

Figure 30. Cylindrical Precast Pedestal Foundation.

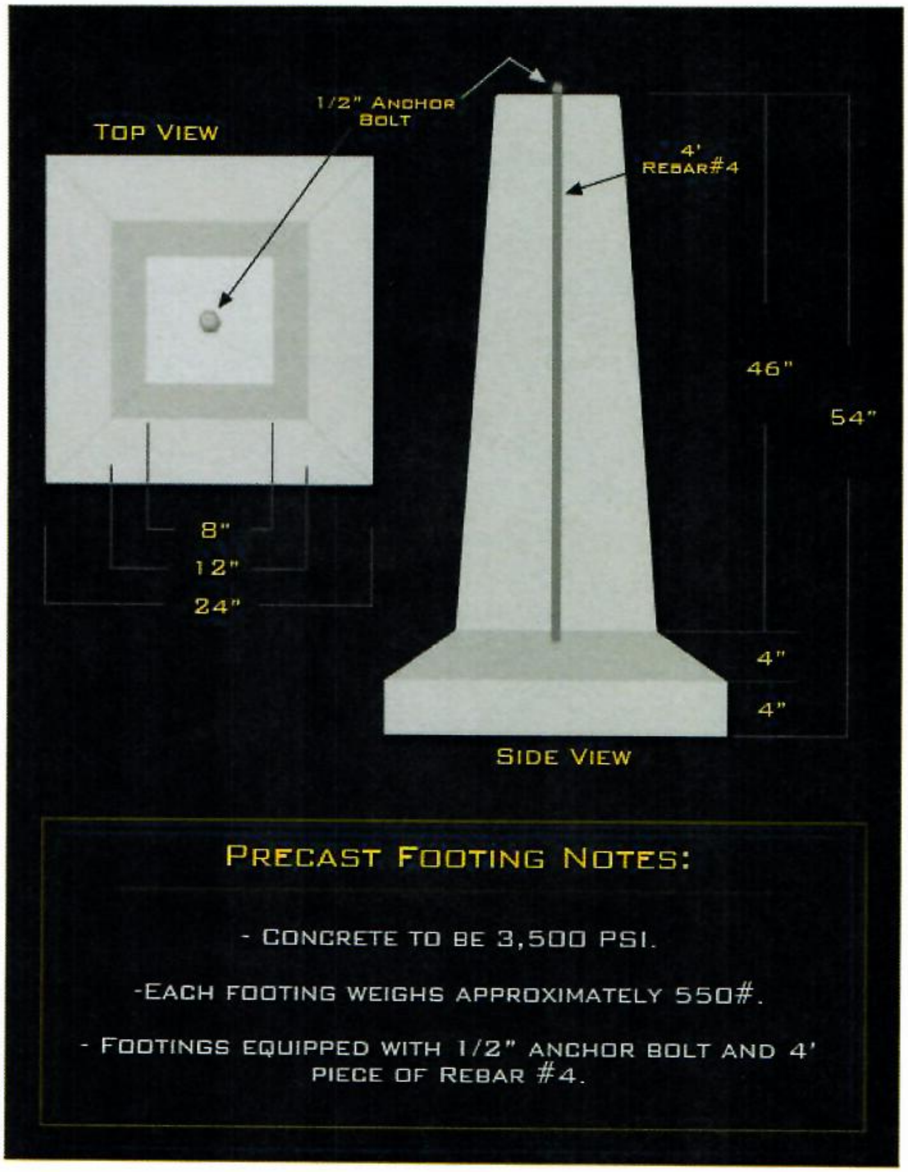

Figure 31. Precast Pyramid Pedestal Foundation.

One type of commercially available precast pedestal foundation is shown in Figure 31. This style of pedestal comes in a fixed geometry so it must be predetermined that the uplift capacity developed will be adequate within the soils in the embedment zone. A simple connection may be provided at the top of the foundation to attach the support structure. The tapered shape of this foundation also reduces frost heave forces in cold climate zones. Construction is performed by excavating (in the case of a square base) or drilling (in the case of a round base) to the foundation depth, removing loose soil at the base of the excavation, placing the foundation and backfilling. The backfill must be properly compacted as with any overdrilled foundation. Figures 32 and 33 show photos of precast pedestals delivered to a site.

Figure 32. Precast Pyramid Pedestal Foundation.

Figure 33. Precast Pyramid Pedestal Foundation.

Another type of precast foundation with an enlarged base is shown in Figure 34. This commercially available foundation is constructed of segmental sections so that the length may be adjusted as needed at any particular site.

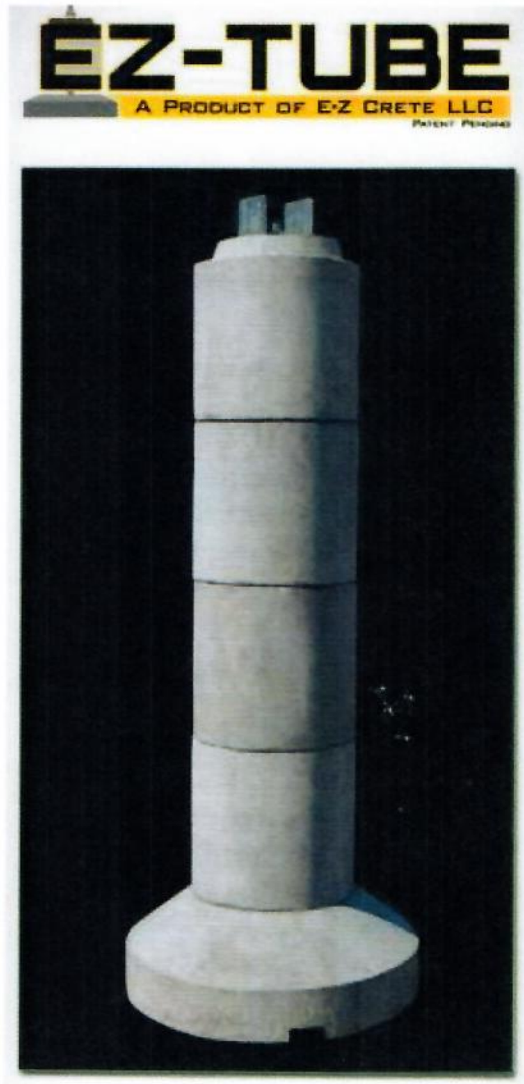

Figure 34. Precast Segmental Enlarged Base Pedestal Foundation.

3.4.1 Advantages

The main advantage to using a precast concrete pedestal foundation is that there is no need to wait for the concrete to harden before above ground construction can proceed. There is much less mess at the site since no concrete trucks are needed. All of the foundations can be delivered at the same time if needed so that the construction schedule can be accelerated.

3.4.2 Limitations

Backfill needs to be of acceptable quality and backfill compaction must be good. Capacity depends on quality of backfill. High groundwater conditions may cause softening and reduction of load capacity and difficult construction.

3.5 Cast-in-Place Footing

Foundations for solar panels can be constructed in a manner similar to constructing a shallow isolated footing to support a tower or bridge or a single column in a building. An excavation is made to the required depth, a steel reinforcing cage is inserted and concrete is placed to create a footing, usually square and typically on the order of 10 to 14 in. thick. A connecting plate is embedded in the concrete so that a steel column can be attached to support the solar panel frame or alternatively a steel pipe or square tube section may be embedded directly in the concrete as it is poured. Typically is a steel tube is used, it will have short (6 in. to 12 in. long) steel "lugs" welded on the sides at the bottom to provide anchoring in the concrete. Figures 35 to 38 show a typical construction sequence of a cast-in-place concrete footing to provide support for a light pole structure, similar to using the foundation to support a solar array.

Figure 35. Placing Concrete in a Form for a Shallow Cast-in-Place Footing.

Figure 36. Steel Reinforcing Cage Placed in Center of Footing for Pedestal.

Figure 37. Completed Pedestal and Stem for Cast-in-Place Footing.

Figure 38. Backfill and Compaction Over the Top of Cast-in-Place Footing.

There are two common construction methods used when constructing a cast-in-place footing; 1) neat excavation; and 2) over-excavation. Neat excavation is preferred since it takes less time, requires removal of less material, eliminates the need for concrete formwork and requires less labor. It also provides for good contact between the concrete and native soil. In a neat excavation the excavation is made to the exact size of the footing so that concrete is placed directly in contact with the adjacent soil. Figure 39 shows an example of a "neat" excavation for a cast-in-place footing. Note that there are no forms used in this construction.

In over-excavation, the excavation is made larger than the dimensions of the footing and wood forms are used to create the footing dimensions. Over-excavation is sometimes used in soils that are unstable and the excavation walls may slough or cave. Neat excavation is applicable to stiffer or drier soils that will remain open over the construction period and not cave into the open excavation. In either case, high groundwater conditions at a site may not allow the excavation to be completed. In both cases, after the concrete is set and the forms (if used) are removed, suitable backfill in s placed in layers about 9 to 12 in. in losses thickness and then compacted over the footing. The compaction should be sufficient to produce a stiff to very stiff soil. If the native soil from the excavation is too wet or contains too many organics, it should not be used as backfill and other suitable soil should be brought in and used for the compacted soil.

Figure 39. Example of a Neat Excavation for a Cast-in-Place Footing.

3.5.1 Design

Cast-in-place footings are a variation of overdrilled and cast-in-place piers but are constructed as a typical shallow foundation with a stem extending to the ground surface to support the structural frame. One advantage of using a cast-in-place footing is that most general contractors can perform the work and no special equipment is needed. The uplift behavior is controlled largely by the quality of the backfill placed over the footing. In most cases there is no excess soil to dispose of unless the excavated soil is considered unsuitable as backfill and other soil needs to be imported to the site. The design for uplift behavior of shallow footings has been discussed extensively by Kulhawy (1985) and Trautmann & Kulhawy (1988).

The uplift capacity of cast-in-place footings depends on the size of the excavation relative to the size of the footing. In a neat excavation the capacity is the combination of the mass of the concrete, the mass of the compacted soil over the top of the footing and the shearing resistance between the compacted soil and the native soil around the perimeter of the excavation.

$$Q_{TOTAL}\ (\text{lbs.}) = Q_{MASS} + Q_{SOIL} + Q_F \qquad [3]$$

where:

Q_{TOTAL} = Total Capacity

Q_{MASS} (lbs.) = L (ft.) x W (ft.) x H (ft.) x 150 lbs./ft.^{3}

Q_{SOIL} (lbs.)= L_E x W_E x H_E x Density

L_E = Length of Excavation (ft.)
W_E = Width of Excavation (ft.)
H_E = Height of Excavation above the Footing (ft.)
D = Density of Compacted Soil (lbs./ ft.^{3})
Q_F (lbs.) = (L + W) x 2 x H x F

3.5.2 Advantages

Cast-in-place concrete footings are a common construction practice and Contractors will be familiar with the construction process. No special equipment is needed to complete the construction.

3.5.3 Limitations

Backfill needs to be of acceptable quality and backfill compaction must be good. Capacity depends on quality of backfill. High groundwater conditions may cause softening and reduction of load capacity and difficult construction.

3.6 Precast Concrete Slab

An alternative to constructing a cast-in-place concrete footing is to obtain precast concrete slabs, similar to using precast concrete pedestal. In effect, this type of foundation acts similarly to a precast pedestal, except that the slab is larger and there is a short stem in the center for attaching a steel post.

3.7 Driven/Vibrated Steel Piles

Driven piles are an attractive foundation alternative for ground mount solar panel systems since the materials are readily available and Contractors are familiar with the technology. For the most part, steel pipe piles and H-Piles are used more than concrete and timber piles that are used for other applications. Driven piles to support ground mount solar systems are typically lighter duty that those used for other structural applications with pipes typically in diameters ranging from 4 to 8 in. in diameter and H-piles typically made from W sections with flanges between 6 and 10 in. A light duty drop hammer can be used to perform the installation and there is little disturbance to the ground so there is only minor cleanup. Figure 40 shows a solar panel installation supported by driven H piles. Note that the pile acts as both the below ground foundation and the above ground panel support structure.

Steel piles may be installed using a simple drop weight hammer or in some cases they may be vibrated into the ground. Figures 41 to 44 show different equipment used to install steel piles.

Figure 40. Driven H Piles Supporting a PV System.

Figure 41. Installing Steel Pipe Piles Using a Tractor Mounted Drop Hammer.

Figure 42. Using a Vibratory Plate Compactor to Install Steel Piles.

Figure 43. Commercial Installation of Steel Piles Using a Special Pile Hammer System.

Figure 44. Installation of Steel Piles Using a Special Pile Hammer System.

3.7.1 Plain Pipe Piles

Plain steel pipe piles are a common foundation system for elevated PV installations. Steel pipe with outside diameters ranging from about 4.5 in. to 10.5 in are typically used. Pipe wall thickness is typically on the order of 0.25 in. (Schedule 40). Most steel pipes for PV installations are installed with an open end. The uplift resistance developed by steel pipe piles depends almost entirely on the side resistance developed between the soil and the wall of the pipe along the exterior perimeter of the pipe. Larger diameter and longer pipes therefore develop higher uplift resistance.

Figure 45 shows results of uplift tests conducted on three different diameters of steel pipe piles at a site in S. Deerfield, Ma. The soils at the site consist of a 3 ft. layer of sandy silt overlying medium sand. The influence of pipe diameter and increased surface area of the pipe is clearly seen, with the larger diameter pipe giving the highest capacity. Similar results for the same size and length piles installed in a stiff clay in Hadley, Ma. are shown in Figure 46.The pipes at both locations were installed using the tractor mounter drop weight hammer previously shown in Figure 41.

One of the interesting trends shown in Figures 45 and 46 is that once the maximum load is achieved, the pile shows very large displacement behavior. That is, once the side resistance between the pile surface and the adjacent soil is overcome by the uplift load, the pile is in complete failure and can resist no further load. This characteristic behavior is very typical for piles in uplift that only rely on pile-to-soil side resistance to uplift load capacity. Of course, this is not a structural failure of the steel pipe but is a soil-to-steel failure. An increase in uplift load capacity can only be realized by using longer piles or larger diameter piles.

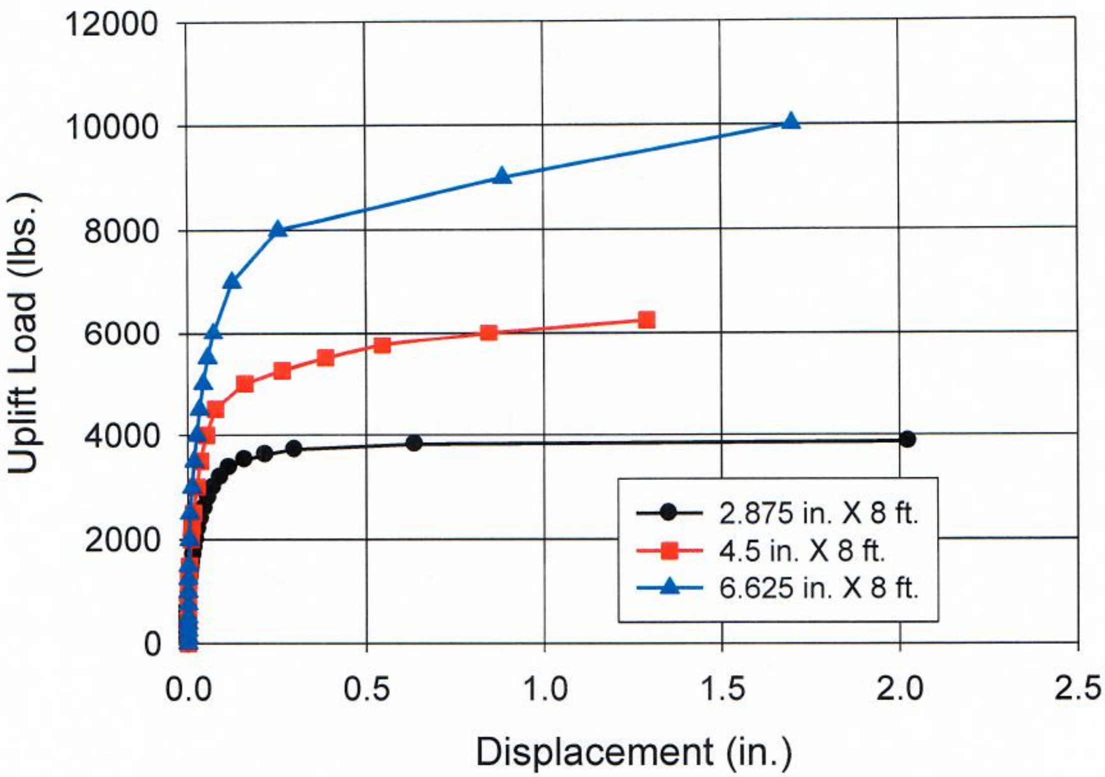

Figure 45. Results of Uplift Tests Performed on Plain Steel Pipe Piles- UMass AgFarm - Deerfield, Ma.

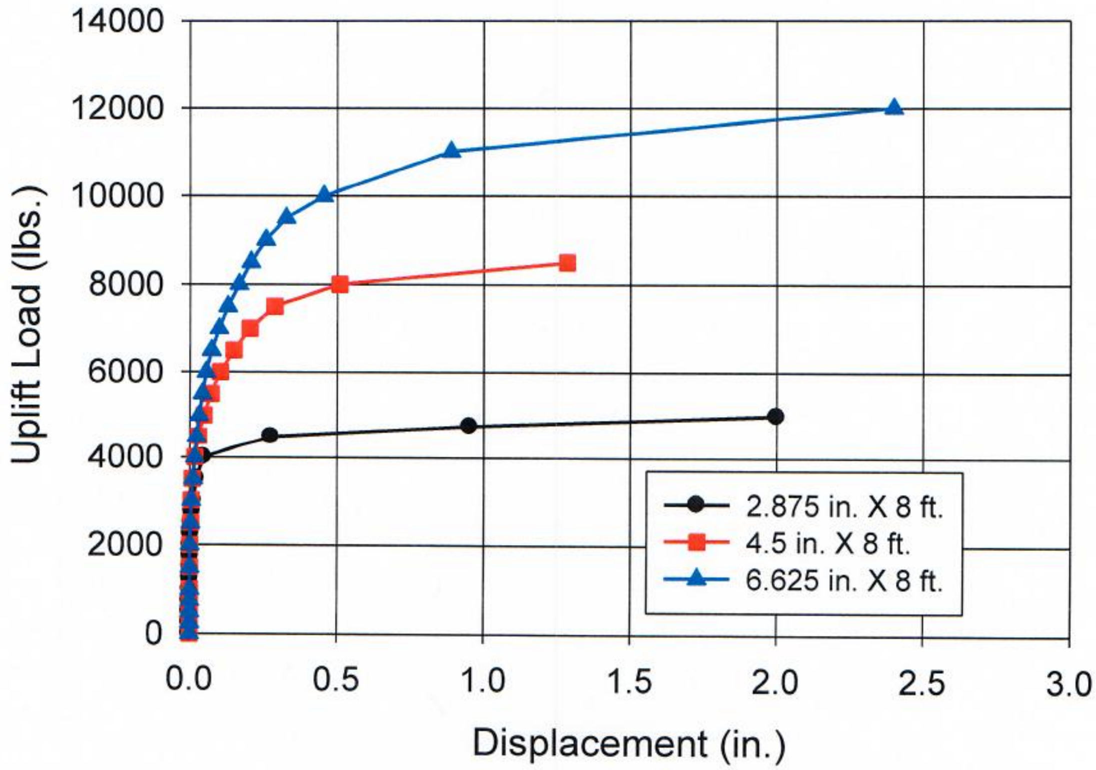

Figure 46. Results of Uplift Tests Performed on Plain Steel Pipe Piles in Stiff Clay – Hadley, Ma.

3.7.2 Fin Piles

Since the uplift capacity of driven piles in most soils depends on the side resistance developed between the soil and pile perimeter, this can be enhanced by attaching additional steel plates or "fins" on the pile to increase the surface area (Lutenegger 2012). This is a simple modification that can even be performed on site if there is a need. In the last 10 years fin piles have been investigated extensively for increasing the lateral stability of driven piles (e.g., Reinert & Newman 2002; Irvine et al. 2003; Songlin 2007; Duhrkop & Grabe 2008; Peng et al. 2010). The use of Spin-Fin Piles to provide increase tension capacity has previously been described by Campbell et al. (1987) and Nottingham & Christopherson (1990).

To demonstrate the influence of adding fins to a plain steel pipe pile, some initial tests were conducted on three sizes of Schedule 40 steel pipe piles. Both plain piles and piles fitted with rectangular shaped steel fins welded to the outside were evaluated. Two different sizes of fins were evaluated and fins were located near the base of the pile and near the top, as shown in Figure 47. Each fin pile was equipped with 4 fins welded at 90^0 around the pipe shaft. All piles were embedded to a depth of 8 ft. (2.4 m) in the silty sand at the UMass AgFarm site in S.

Deerfield, Ma. Table 3 summarizes the geometry of the piles tested. Table 3 also includes the ratio of the total surface area of the pile (pipe + fin) to the surface are of the pipe alone, A_T/A_P.

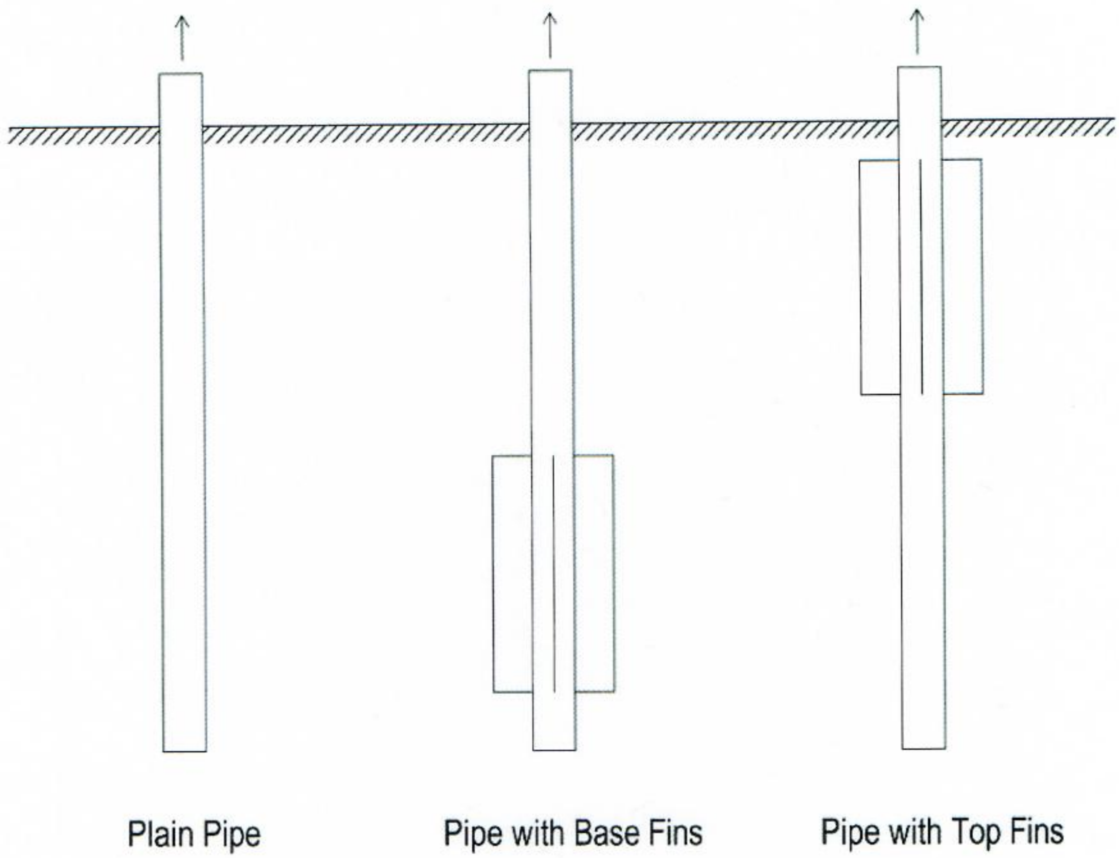

Figure 47. Schematic of Different Plain and Fin Piles Tested.

Table 3. Summary of Initial Fin Piles Tested – UMass AgFarm – S. Deerfield, Ma.

Pile No.	Pile Outside Diameter (in.)	External Shaft Surface Area (in.^{2})	Fin Location	Fin Dimensions (in.)	Fin Surface Area (in.^{2})	A_T/A_P
1	4.5	1357	-	-	-	1.0
2	4.5	1357	Bottom	6 x 36	1728	2.27
3	4.5	1357	Top	6 x 36	1728	2.27
4	4.5	1357	Bottom	6 x 48	2304	2.70
5	4.5	1357	Top	6 x 48	2304	2.70
6	6.625	1998	-	-	-	1.0

7	6.625	1998	Bottom	6 x 36	1728	1.86
8	6.625	1998	Top	6 x 36	1728	1.86
9	6.625	1998	Bottom	6 x 48	2304	2,15
10	6.625	1998	Top	6 x 48	2304	2.15
11	8.625	2601	-	-	-	1.0
12	8.625	2601	Bottom	6 x 36	1728	1.66
13	8.625	2601	Top	6 x 36	1728	1.66
14	8.625	2601	Bottom	8 x 48	3072	2.18
15	8.625	2601	Top	8 x 48	3072	2.18

A typical set of uplift load-displacement curves obtained for the 6 5/8 in. diameter piles is shown in Figures 48 and 49 for bottom fins and top fins, respectively. Similar behavior was observed for all three sizes of piles and can be characterized as follows:

1. Plain piles show an abrupt transition to failure. When failure load is reached, the movement becomes very large and load cannot be maintained by continuous pumping of the hydraulic pump;
2. Piles fitted with fins show a more gradual transition to failure and even though they reach large displacements, the load can still be maintained, even past a displacement of 10% of the pile diameter.
3. In every case but one, piles with fins developed larger capacities than plain piles of the same diameter.

Table 4 gives a summary of the load test results and provides a comparison of the ultimate uplift load capacity of the fin piles as compared to the plain pipe piles, Q_{fin}/Q_{plain},

The increase in total uplift capacity of the fin piles over the plain piles is too great for the piles to be behaving solely by the increase in surface area provided by the fins and an increase in side resistance. This is demonstrated by the unit side resistance values given in Table 4 which are actually lower for most fin piles.

Unit side resistance was obtained from:

$$f_s = (Q_{ult} - W)/A_s \quad [4]$$

where:

f_s = Unit Side Resistance
Q_{ult} = Ultimate Uplift Capacity
W = Mass of Pile
A_s =External Area of Pile (including fins)

Equation 4 assumes that capacity is only developed from side resistance along the pipe shaft and fins.

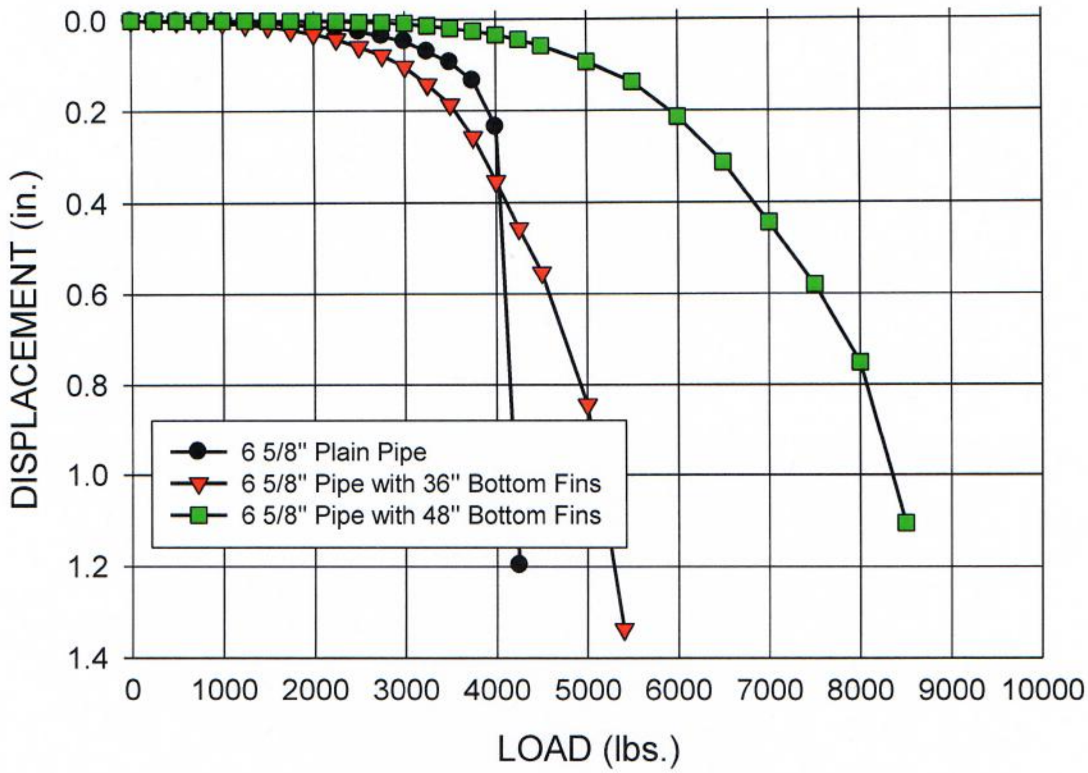

Figure 47. Uplift Load Test Results on 6.625 in. Pipe Piles with Bottom Fins.

Table 4. Measured Driving Resistance, Soil Plug Length and Ultimate Capacity for Fin Piles.

Pile No.	Total Driving Resistance (blows)	Plug Length (in.)	Interpreted Ultimate Capacity (lbs)	Unit Side Resistance (psf)	Q_{fin}/Q_{plain}
1	85	64.0	4100	435	1.0
2	149	56.5	5500	245	1.34
3	78	59.0	6500	289	1.58
4	127	56.0	4600	174	1.12
5	95	55.0	5800	220	1.41
6	93	69.0	4100	295	1.0
7	148	73.0	5200	193	1.27
8	100	72	8000	298	1.95
9	189	72	8200	266	2.0
10	98	72	8100	262	1.98
11	133	37	3700	171	1.0

12	202	64	2700	78	0.73
13	201	81	5500	159	1.49
14	439	82	6100	148	1.65
15	214	67	7500	182	2.03

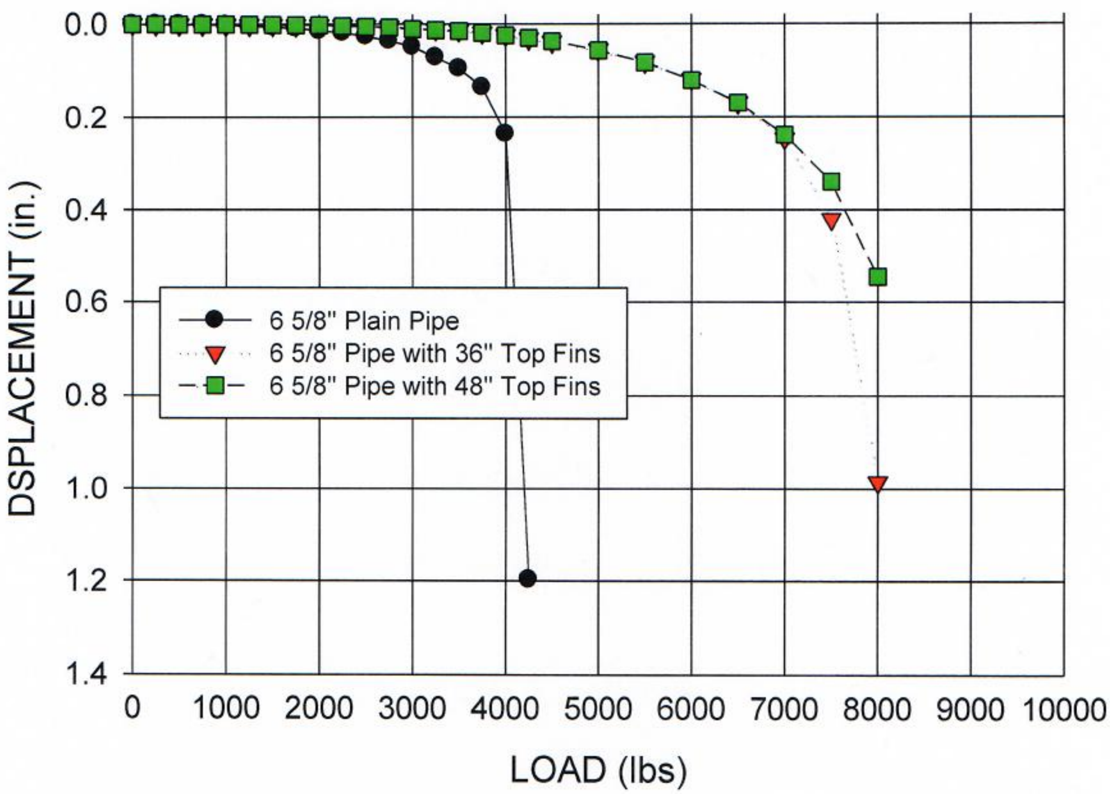

Figure 48. Load Tests Results on 6.625 in. Pipe Piles with Top Fins.

A model for the tension behavior of Spin-Fin piles was suggested by Nottingham & Christopherson (1990) as shown in Figure 49 and assumes that soil is lodged between the fins and creates an end-bearing component to uplift capacity just above the fins. It appears that piles with fins located at the bottom likely developed some component of "end bearing" at the top of the fins. However, it might be unlikely that this could occur for piles with fins at the top simply because of the shallow depth of the fins. Top fins on these piles were placed so that after driving the top of the fin would only be about 0.5 ft. below ground surface.

Following this initial set of tests, another series of fin piles was conducted at three sites. In this case only pipes with a diameter of 4.5 in. were investigated. Figure 50 shows the shape of the fins tested. The number of fins was varied as shown in Figure 51. Only bottom fins were considered based on the results obtained from the initial series of fin pile tests.

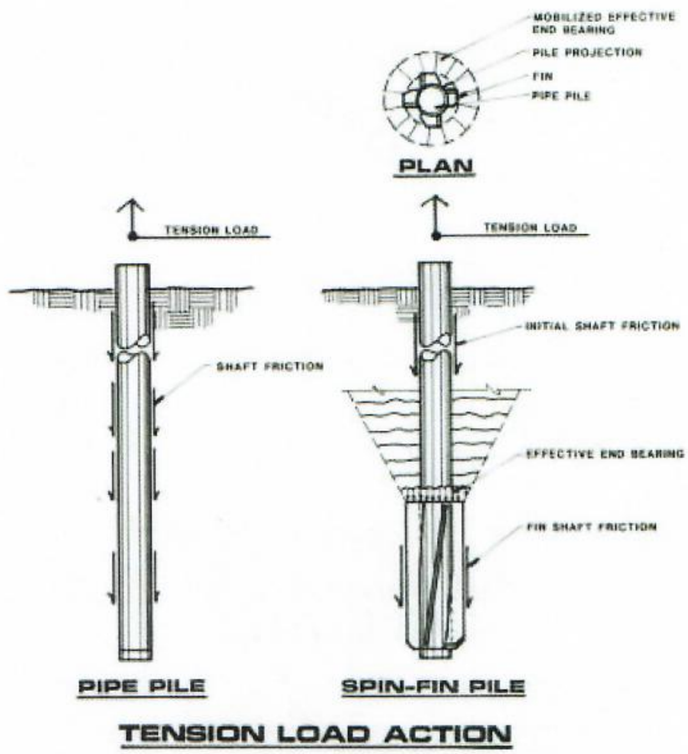

Figure 49. Assumed Uplift Behavior of Spin-Fin Piles.

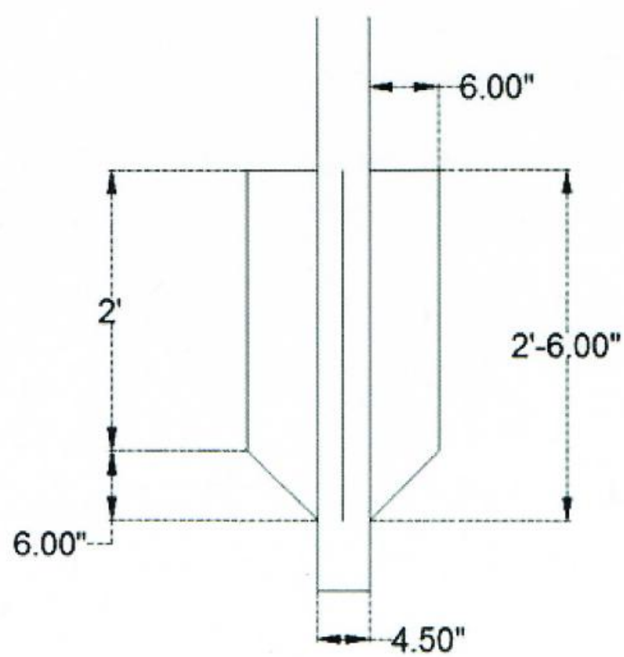

Figure 50. Shape of Pile Fins Investigated in Second Series of Fin Pile Tests.

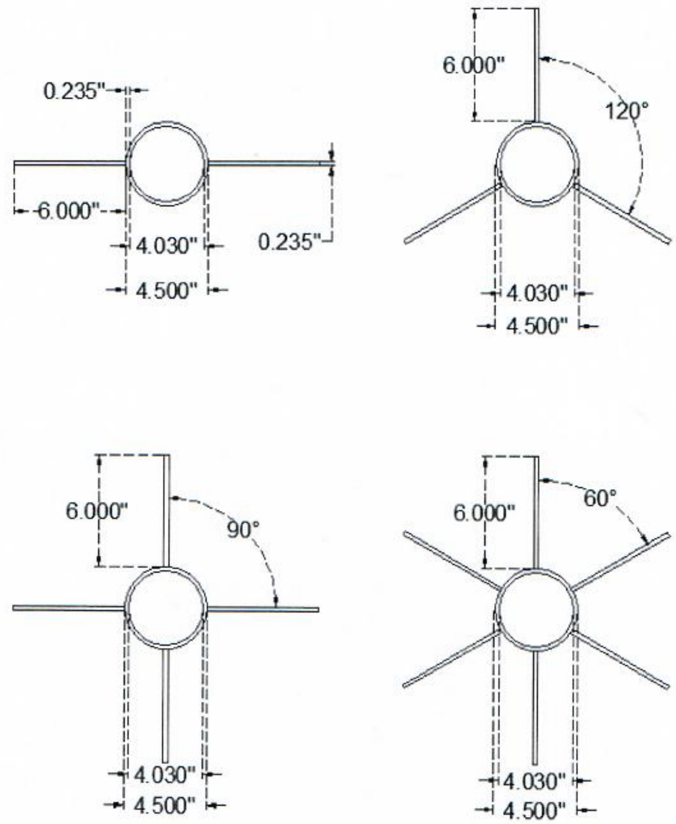

Figure 51. Location of Fins on Pipe Piles; 2, 3 4 and 6 Fins.

Uplift tests on these fin piles were performed at three sites:

Site-1 UMass AgFarm - Silty Fine Sand

The site is located in South Deerfield, Ma. at the University of Massachusetts Agricultural Experiment Station. The soils consist of about 7 ft. of tan to light brown moist low plasticity to non-plastic silt and sandy silt overlying alluvial uniform fine to medium sand. The sand extends to a depth of about 16 ft. A standpipe piezometer placed at a depth of 10 ft. indicated no water table during pile installation or load testing.

Site-2 UMass Horse Farm - Medium Soft Clay

The site is located in Hadley, Ma. at the UMass Hadley "Horse Farm" on Maple Street. The soil to a depth of 4 feet beneath the ground surface consists of a stiff gray sandy clay crust overlying a soft gray low plasticity silty clay that extends to a depth of at least 30 ft. The groundwater table fluctuates seasonally at the site due to changes in precipitation. During the time of the tests the water table was located at a depth of about 2 ft.

Site-3 Franklin Tech School - Clean Uniform Sand

The site is located in Turners Falls, Ma. at the Franklin Technical Regional High School. The soil at the site consists of a uniform fine to medium poorly graded sand to a depth of 12 feet below the ground surface. The groundwater table depth was not measured, however, it should be noted that no wet samples were collected during any explorations.

At the first two sites, the piles were installed by driving, using the tractor mounted drop hammer previously shown. At the third site, piles were installed using a vibratory hammer to evaluate the efficiency of installing piles quickly. Therefore no driving record is available for this site. During installation at the two UMass sites, the number of hammer drops to install each pile was recorded. Figures 52 and 53 show the influence of adding fins on the effort required to install the piles. Figure 54 shows how the number of hammer drops increases with surface area of the pile + fins.

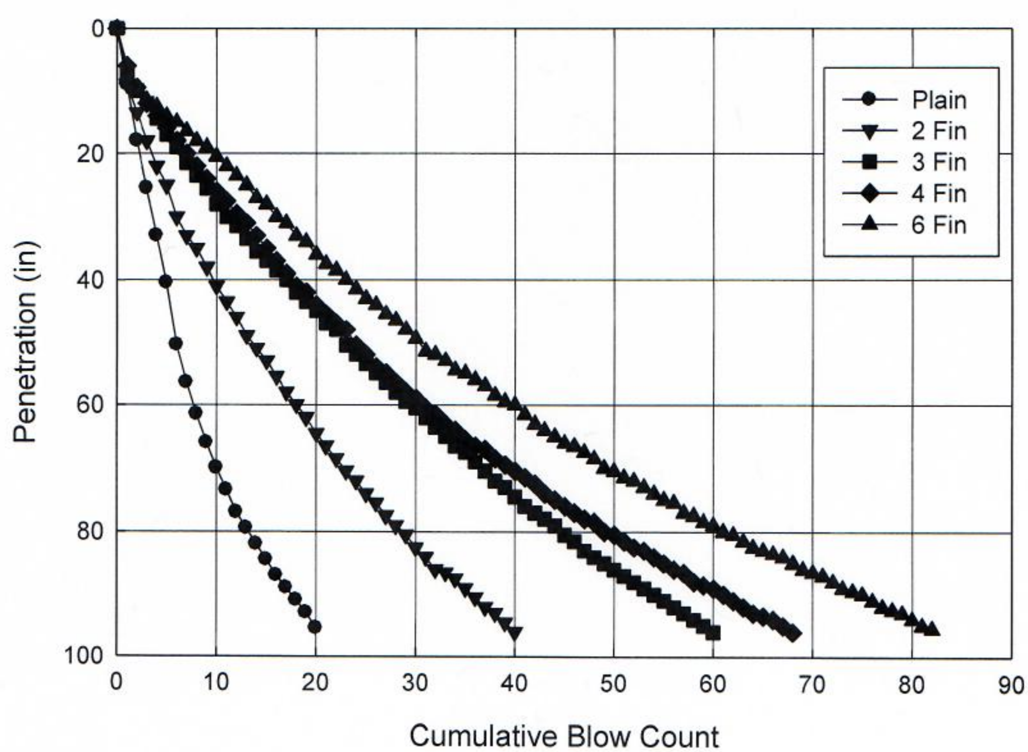

Figure 52. Summary of Fin Pile Installation Driving Records: UMass AgFarm Site.

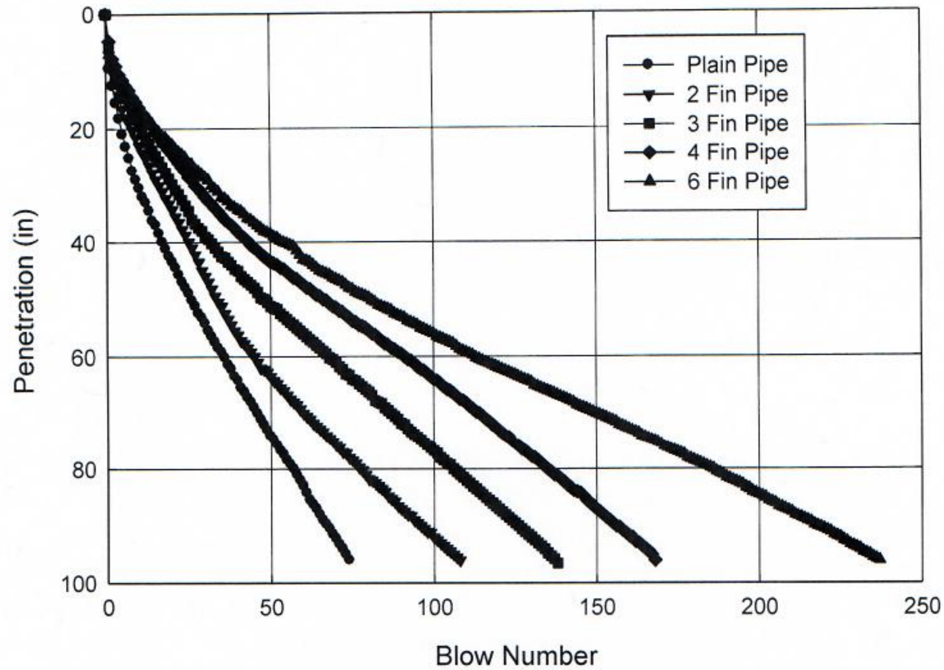

Figure 53. Summary of Installation Driving Records: UMass Horse Farm Site.

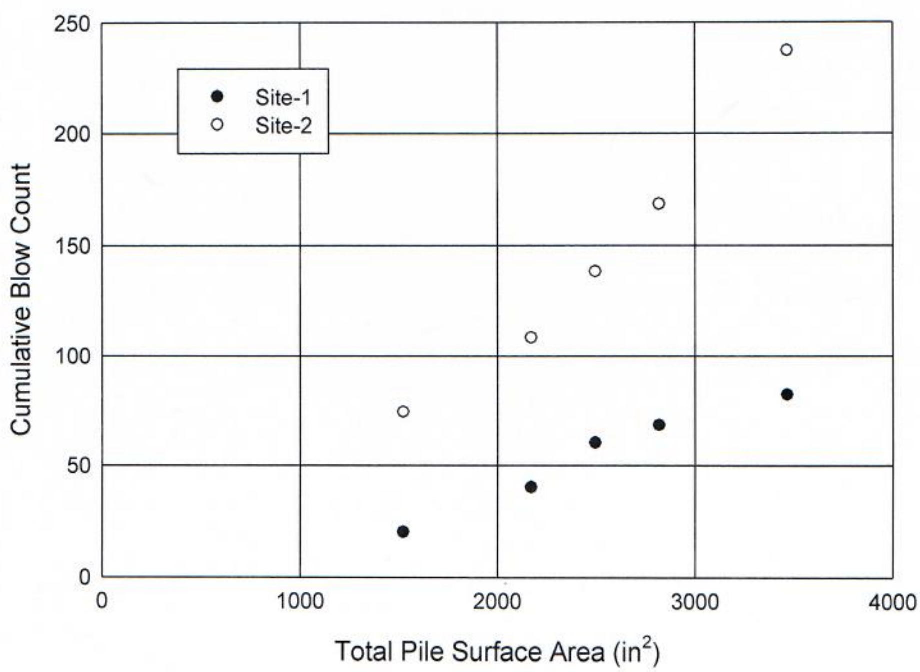

Figure 54. Influence of Total Pile Surface Area on Driving of Fin Piles.

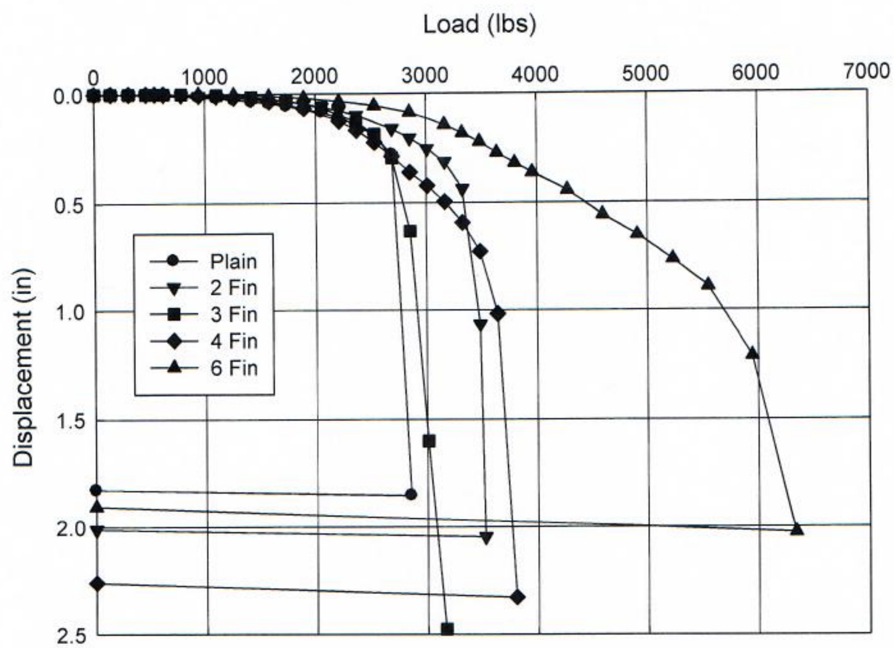

Figure 55. Results of Uplift Load Tests – Fin Piles UMass AgFarm Site.

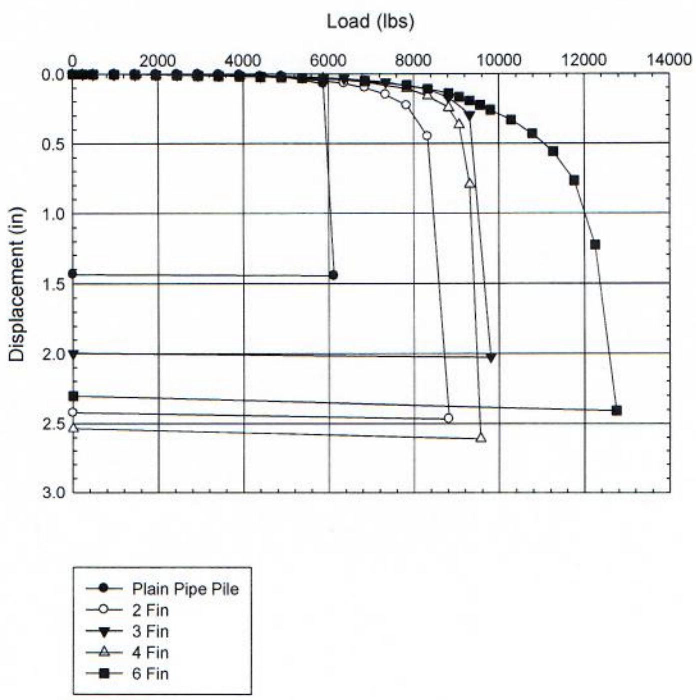

Figure 56. Results of Uplift Load Tests on Fin Piles – UMass Horse Farm Site.

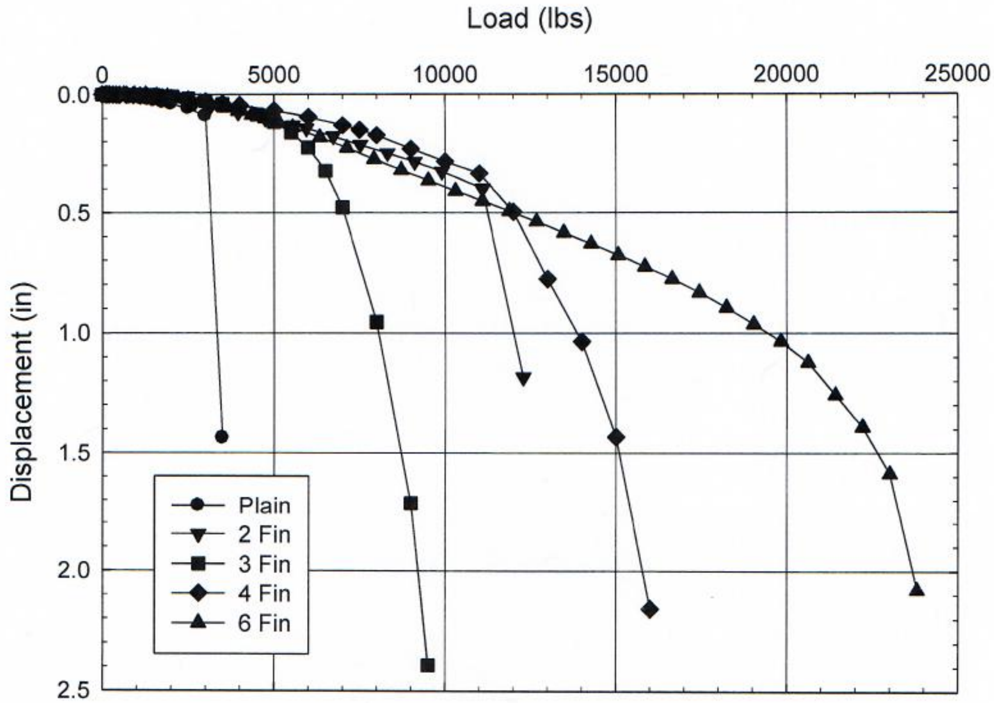

Figure 57. Results of Uplift Load Tests on Fin Piles –Franklin Tech Site.

Table 5. Results of Uplift Load Tests and Back-Calculated Unit Side Resistance.

Number of Fins	Site-1		Site-2		Site-3	
	Q_{ult} (lbs)	$f_{s,bc}$ (psf)	Q_{ult} (lbs)	$f_{s,bc}$ (psf)	Q_{ult} (lbs)	$f_{s,bc}$ (psf)
0	2700	286	6000	637	3250	345
2	3500	251	8550	614	11700	840
3	3150	195	9400	581	9250	572
4	3700	201	9400	510	15500	841
6	6200	270	12600	550	23500	1025

Results of uplift load tests for all three sites are shown in Figures 55 to 57. Results of the tests are summarized in Table 5. Figure 58 shows a trend of increasing uplift load capacity with increasing number of fins, which is related to the increase in surface area. In summary, these results clearly show that one simple method in increasing the uplift load capacity of a driven pipe pile is to add fins at the bottom. Figure 59 shows a photo of the 6 fin pile extracted from the ground after the completion of load testing at the UMass Horse Farm Site and shows the soil lodged in between the fins.

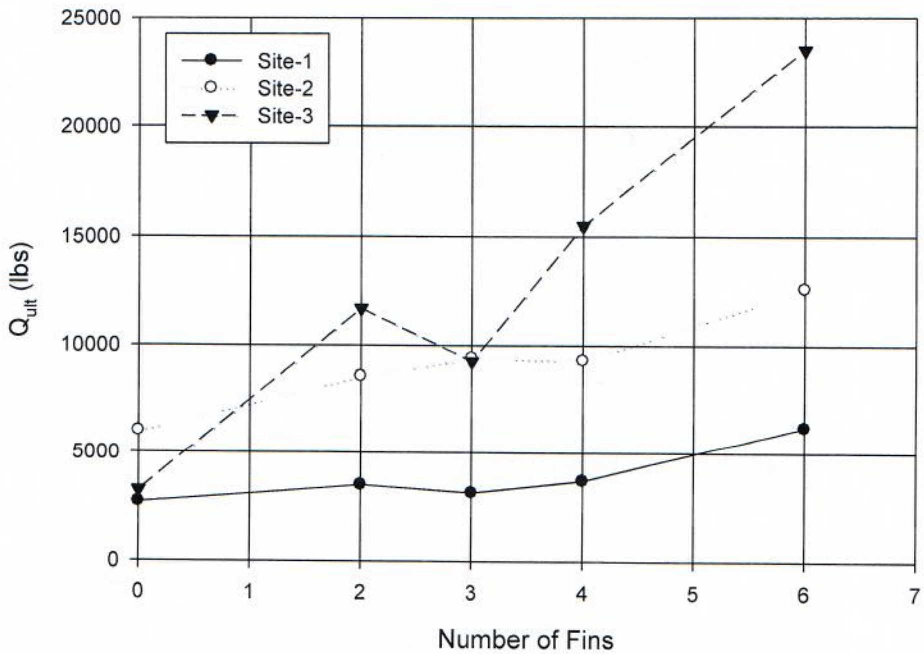

Figure 58. Increase in Ultimate Uplift Load Capacity with Increasing Number of Fins.

Figure 59. Soil Lodged Between Fins of 6 Fin Pile Extracted from UMass Horse Farm Site.

3.7.3 Steel H-Piles

H-Piles for supporting PV solar systems usually consist of standard structural steel sections obtained from a local steel supplier. Installation may be by driving using an impact drop hammer or by vibration. For driving, a tractor with a small post driving attachment mounted on the rear can be used as previously noted. Deep foundations consisting of driven H-Piles are a relatively common alternative to other types of driven piles, such as precast concrete or steel pipe piles. They are structurally sound and can be driven in a wide range of materials from very soft clay to very dense sand. If bedrock or a very strong layer is very deep, driven H-piles may be used as "friction" piles and derive support between the soil and pile shaft with only a small portion of load taken in end bearing.

Whereas a large amount of research has been conducted on the behavior and development of shaft resistance of driven open and closed end pipe piles, it appears that the development of side resistance of H-Piles is not fully understood. In order to investigate the development of side resistance of H-Piles, as an alternative for supporting PV systems, a series of uplift load tests were performed in natural soils at several sites. The tests consisted of tests on H-Piles of three different sizes. The structural sections used in the current investigation actually represent full size piles being used to resist uplift of small elevated solar panel installations. The owner desired to determine the smallest section that would meet the required load conditions and therefore be the most economical. Table 6 summarizes the sites. The piles were installed at all sites using the tractor mounted drop weight previously shown.

Table 6. Test Site Conditions for Uplift Tests on Steel H-Piles.

Site	Name	Soil Conditions
Site-1	UMass-AF-Solar	Silty Sand
Site-2	UMass-AF-GT	Silty Sand
Site-3	UMass-Taylor	Sand/Clay
Site-4	FR. Tech.	Uniform Med. Sand
Site-5	UMass-DOE	Med. Stiff Clay
Site-6	UMass-Horse Farm	Med. Stiff Clay

The behavior of steel H-Piles, like all piles, is related to the specific geometry of the piles, in this case the dimensions (length and thickness) of the web and flanges. During driving by dynamic penetration, soil may become wedged between the flanges but this is unlikely as the dynamic energy that produces incremental penetration will likely overcome this tendency. This means that the driving resistance will be related to the interface behavior between the pile and the soil. This produces the general perception that H-Piles are "small-displacement" piles and may generally advance fully unplugged.

By contrast, during quasi-static loading, as in an axial load test, the rate of load application is relatively slow, as compared to installation. Any soil that becomes lodged within the flanges may make the pile act as a fully plugged pile during loading. This causes the behavior of the pile to be influenced by both the interaction between the soil and the pile surface (along the outside of the flanges) and the behavior of the soil itself (on the inside between the flanges).

Figures 60 and 61 show results of installation driving records in terms of cummulative number of hammer drops versus pile penetration depth for Site-3 and Site-6. These records show that driving resistance increases progressively with the size or increase in surface area of the piles. That is, larger pile sections require more hammer drops to drive the piles. In one case the piles were driven to a depth of 10 ft. below ground surface and in the other case the piles were driven to a depth of 8 ft.

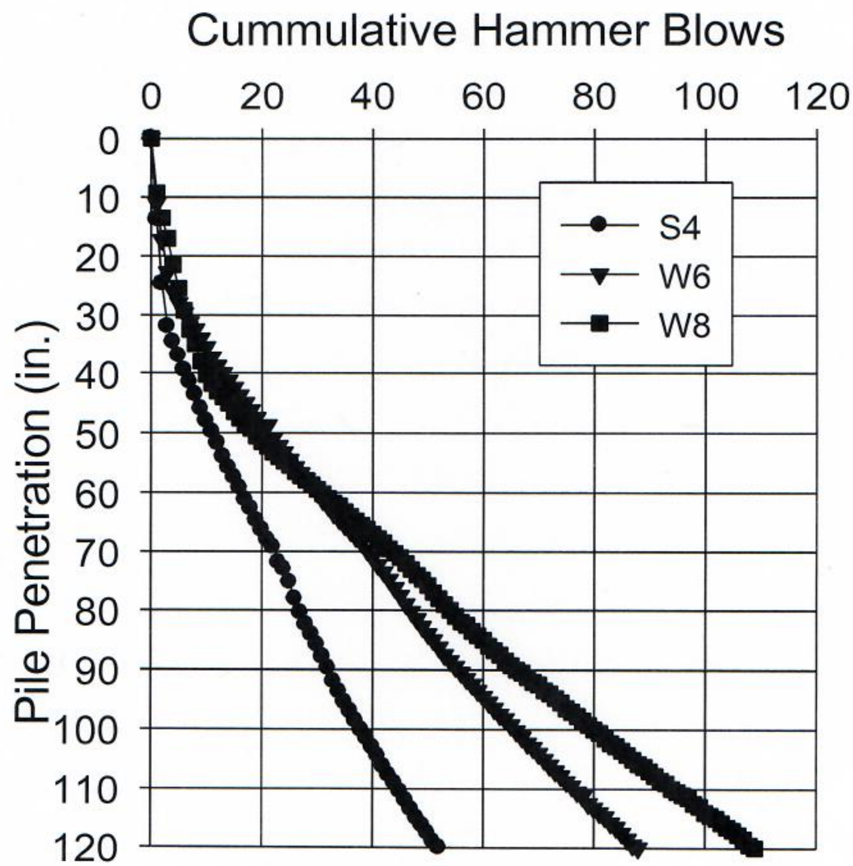

Figure 60. H-Pile Installation for 3 Different Size Piles – UMass Taylor Field Site.

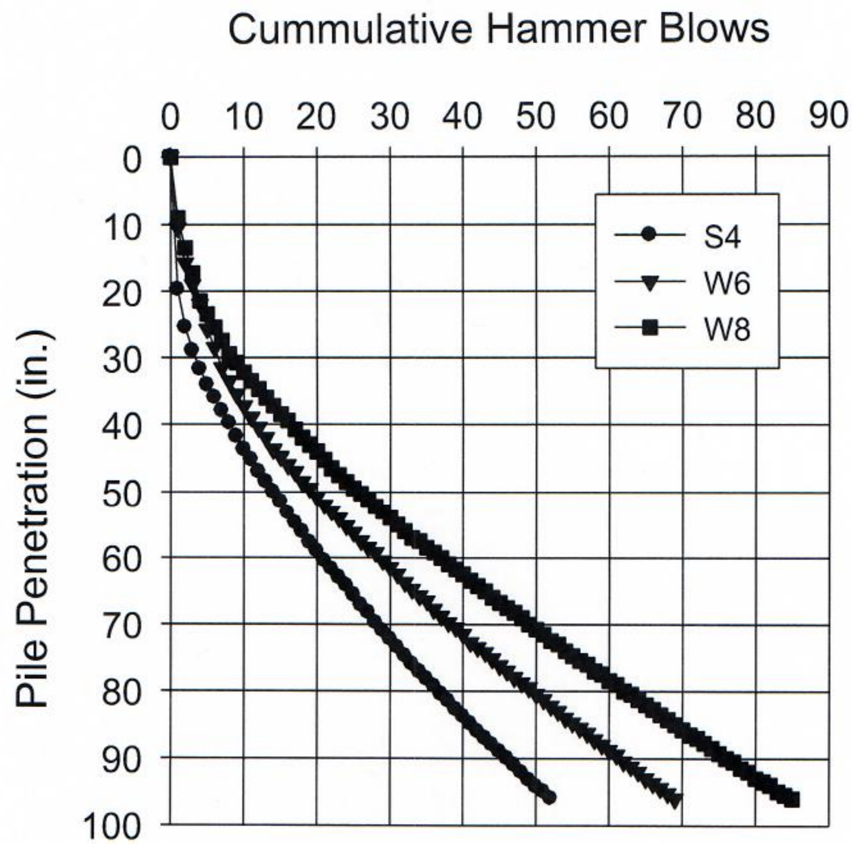

Figure 61. H-Pile Installation for 3 Different Size Piles – UMass Horse Farm Site.

A summary of the uplift load tests is given in Table 7. Figure 62 shows a typical set of load test results for the three different size piles at Site-6. In general, all the piles displayed similar results at each of the sites and in no case was a dramatic "plunging" uplift failure observed in any of the H-piles. This is in contract to uplift tests performed on plain pipe piles, previously discussed. Failure loads were taken as the final load applied which was at a displacement of approximately 2.5 in., as indicated in Figure 62.

Table 7. Summary of Uplift Load Tests on Driven H-Piles.

Site	Pile	Pile Length (ft.)	Interpreted Ultimate Tension Capacity (lbs.)	f_s (psf) (Unplugged)	f_s (psf) (Plugged)
Site-1	S4	8	3600	286	493
	W6	8	2300	137	200
	W8	8	2750	140	192
Site-2	S4	8	1760	140	242

	W6	8	3000	178	260
	W8	8	1950	100	136
Site-3	S4	10	6900	438	759
	W6	10	7700	366	534
	W8	10	7800	314	432
Site-4	W6	8	18000	1070	1562
	W8	8	15000	761	1046
Site-5	S4	10	4600	386	671
	W6	10	5600	327	478
	W8	10	6800	342	471
Site-6	S4	8	6150	488	846
	W6	8	9500	565	824
	W8	8	10800	548	753

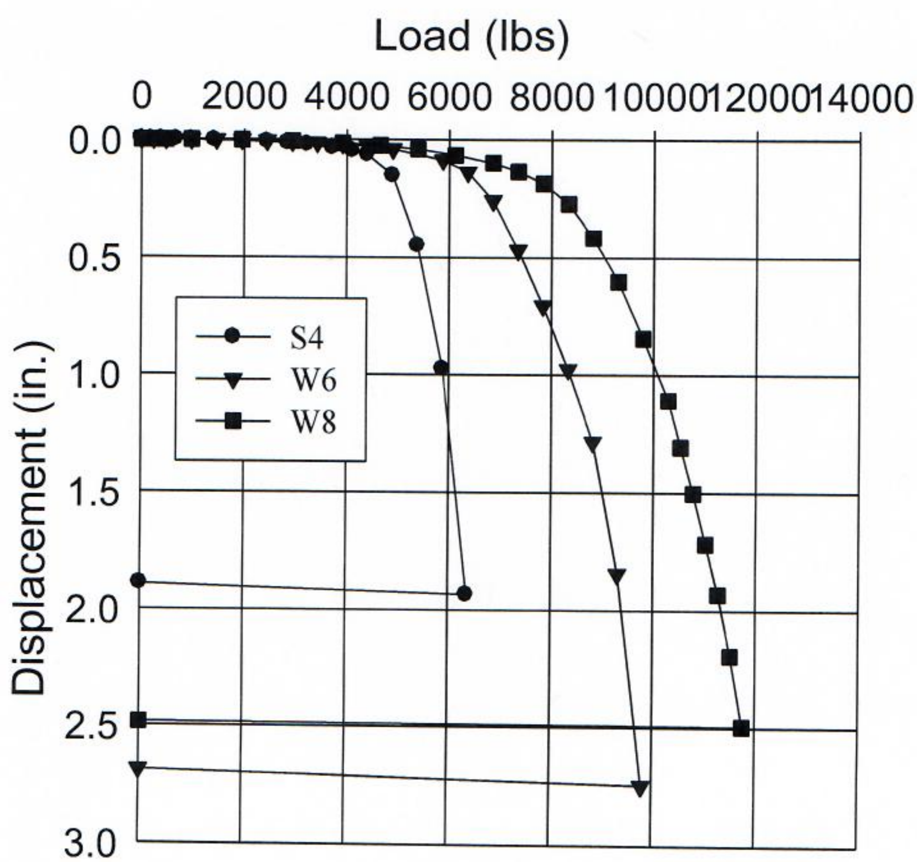

Figure 62. Typical Results of Uplift Load Tests on H-Piles.

Values of unit side resistance calculated from the 17 uplift load tests given in Table 7 range from about 140 $lbs./ft^2$ to over 1000 $lbs./ft^2$. For the clay sites the range in unit side resistance is 470 $lbs./ft^2$ to 850 $lbs./ft^2$ with an average of about 650 $lbs./ft^2$. For the sand sites, the range in unit side resistance is 150 $lbs./ft^2$ to 750 $lbs./ft^2$ with an average of about 400 $lbs./ft^2$.

3.7.4 Design

For design, the uplift load may be obtained from:

$$Q_{ult} = Q_S + W \qquad [5]$$

where:

Q_S = total side resistance = $f_s \times A_S$

For design of H-piles, the side area is taken as the enclosed rectangular area of the section as shown in Figure 63b. The weight, W, could include the weight of the steel plus the soil plug held between the flanges of the section but is usually just taken as the weight of the steel. Back calculated values of unit side resistance for each load test from Table 7 were obtained using the interpreted load at failure from:

$$f_s = (Q_{ult} - W) \qquad [6]$$

where:

f_s = average unit side resistance
Q_{ult} = Interpreted Ultimate Capacity
W = Mass of Pile
A_S = Surface Area

Determining the true value of average unit side resistance along the length of H-Piles is difficult due to the various failure mechanisms that may occur during loading. This is due to the fact that, as noted, the region between the flanges may develop a plug of soil along the web (Fellenius 1955; Peck 1961; Hegedus & Khosla 1983; Coyle & Ungaro 1989; Yoon et al. 1997). When failure occurs, it is not certain whether the failure plane occurs between the soil-pile interface along the length or the plugged soil-soil interface. In many cases, a failure surface in between the two extremes is more probable, i.e., partially plugged. Examples of an unplugged and plugged H-Pile are shown in Figure 63.

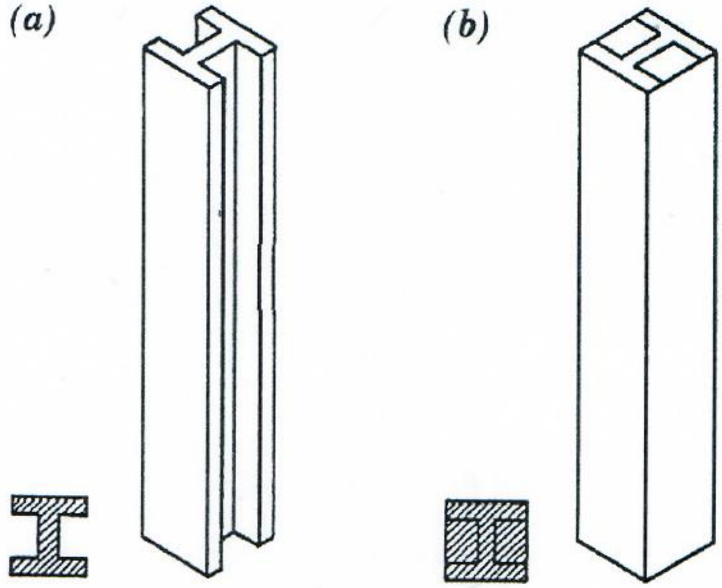

Figure 63. Schematics of Different Surface Area to Calculate Unit Side Resistance (a) unplugged and (b) plugged (Coyle & Ungaro 1989).

Yoon et al. (1997) installed and tested four model H-Piles in sand. H-Piles with greater ratios of web length to flange length experienced higher ultimate uplift capacities than those with lower ratios. This was due to the nature in which the H-Pile plugged. Whereas a greater percentage of the cross sectional area of the H-Piles with lower web to flange ratios were filled with plugged soil, a lower percentage of the cross sectional areas of the H-Piles with higher web to flange ratios were filled with plugged soil which in turn resulted in a higher total surface area of the pile during loading. Therefore, higher unit side resistance was developed with less plugging. A visual representation of this is shown in Figure 64.

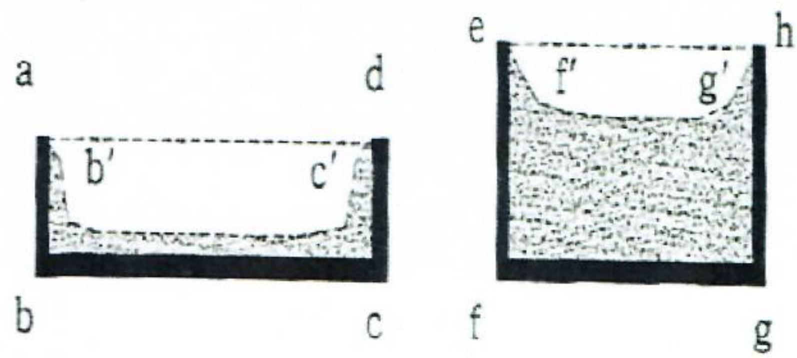

Figure 64. Failure planes of H-Piles with the same section areas (Yoon et al. 1997)

3.7.4 Advantages

A clear advantage of driven or vibrated piles over any form of drilled foundation is that there is minimal disruption or disturbance to the ground surface during installation. There may be some slight to moderate surface disturbance from the tracking of the equipment tires or tracks, but this can often be kept to a minimum. Additionally, the installation is typically very fast, in many cases a 10 ft. to 12 ft. long pile may be driven in less than 10 min. while vibrating piles often can be accomplished in less than 1 min.

There is essentially no cleanup once the installation is competed. Pile sections are readily available from most steel suppliers and can be obtained quickly. If impact driving equipment is used to install piles, the quality control program can include observations and records of the driving resistance of each pile foundation. Vibratory equipment operating at the same vibrating frequency could also be monitored in terms of time of installation as a quality control record.

Another advantage to using driven piles installed with a drop hammer or conventional pile hammer is that the installation can be monitored at every location and pile lengths can be adjusted to suit the soil conditions at each location as needed to provide the design capacity.

3.7.5 Limitations

Load capacity of driven or vibrated piles cannot be validated during installation to indicate whether a pile length should be extended. In some cases, where a large number of piles are to be installed with the same driving equipment, it would be possible to develop a program in which the driving resistance is determined for some test installations and then those test piles are tested in uplift so that a site specific correlation could be developed between driving resistance and load capacity.

3.8 Helical Steel Piles

One of the most attractive and fastest growing types of foundations for ground mount solar panels is steel helical piles. Helical piles are one of the most cost effective types of foundations that can be used to support ground-mount solar panel systems. A helical pile is a manufactured foundation element that consists of a steel pipe shaft with one or more helical plates welded to the bottom end of the shaft, as shown in the schematic of Figure 65. Figure 66 shows a photo of a helical pile with two helical plates.

There are two basic styles of helical pile used to support PV ground-mount systems; one consists of a pipe shaft with one or more individual helical plates welded to the shaft, as shown in Figure 66. The other consists of a thin continuous spiral helical thread welded to the pipe shaft along the lower section, sometimes referred to as a ground screw. This style will be discussed in the next section. Figure 67 shows a rendition of a PV system supported by four helical piles. Of course fewer piles may be used depending on the design.

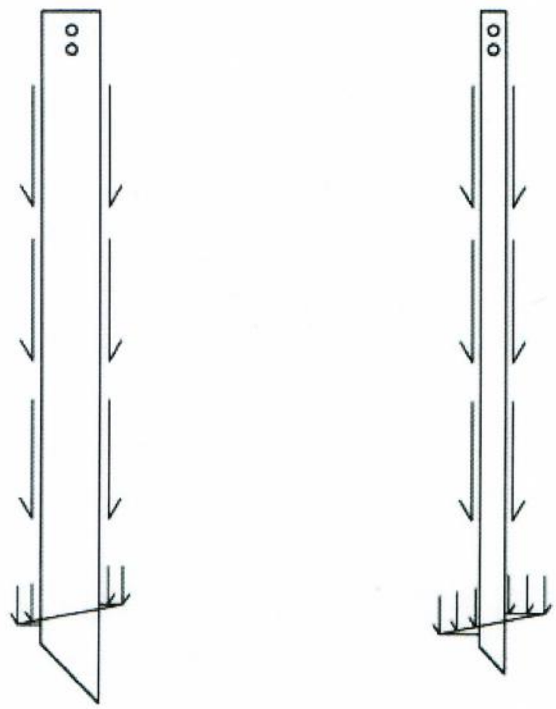

Figure 65. Schematic of Single-Helix Helical Pile.

Figure 66. Helical Piles Consisting of Steel Central Shaft and Two Helical Plates.

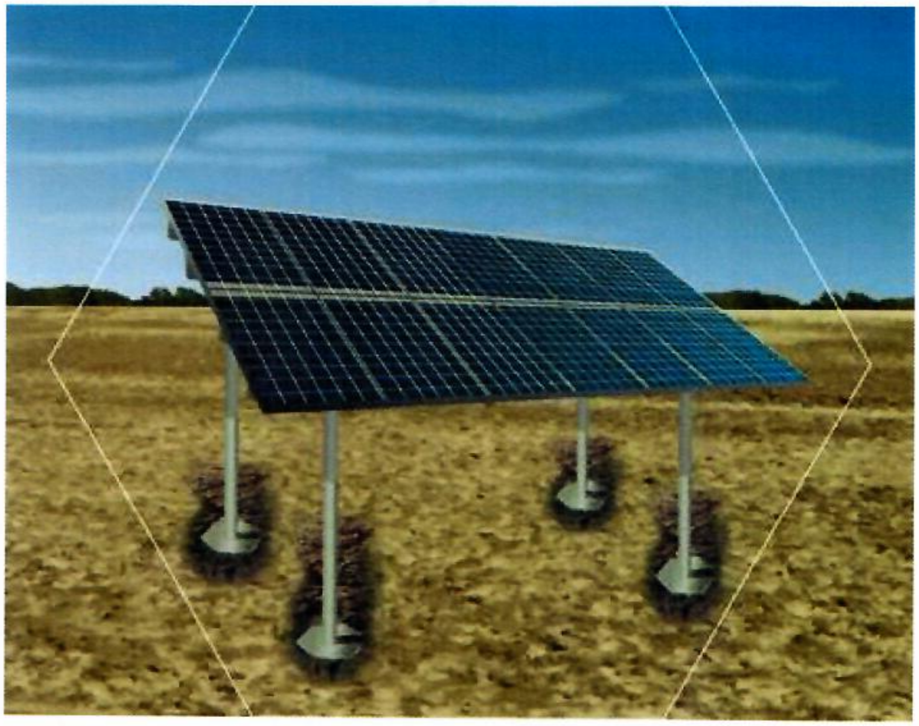

Figure 67. Rendition of PV System Supported by Helical Piles.

Figure 68. Installation of Helical Piles Using a Compact Excavator.

The installation of helical piles is performed with conventional construction equipment, such as a skid steer, compact excavator, backhoe, etc., fitted with a high torque low speed hydraulic torque head, as shown in Figures 68. This is very different than a high speed low torque hydraulic head typically used to drill holes with an auger. Figures 69 and 70 show other construction equipment used to install helical piles.

Figure 69. Using a Small Skid Steer to Install Helical Piles.

Figure 70. Using a Bobcat Loader to Install Helical Piles.

As noted, the uplift load is developed as a combination of side resistance along the pipe shaft and end bearing from the helical plate or only side resistance. In fact the single-helix style produces large resistance to uplift in most soils. Relative to a plain steel pipe pile, a helical pile can develop a much higher load capacity in uplift as the helical plate can produce very large end bearing resistance above the plate in uplift.

Figure 71 shows a comparison between a single helix helical pile and a plain pipe pile of the same diameter in sand obtained at the UMass-AgFarm Site in S. Deerfield, Ma. It can be seen that the plain pipe pile fails at a very low load and has no reserve capacity after failure. By contrast, the helical pile continues to develop capacity with additional movement, as the helical plate is engaged. Similar results were obtained in clay at the UMass DOE Site, Figure 72.

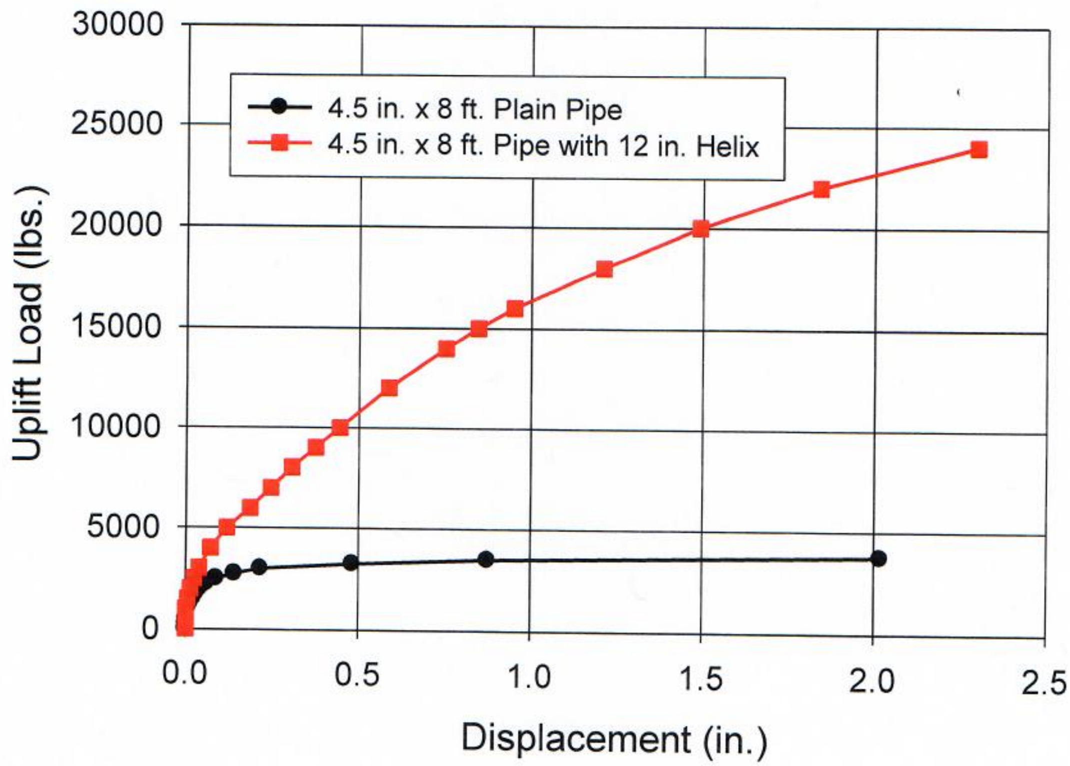

Figure 71. Results of Uplift Load Tests on a Single-Helix Helical Pile and a Plain Pipe Pile of the Same Diameter at the UMsss AgFarm Site.

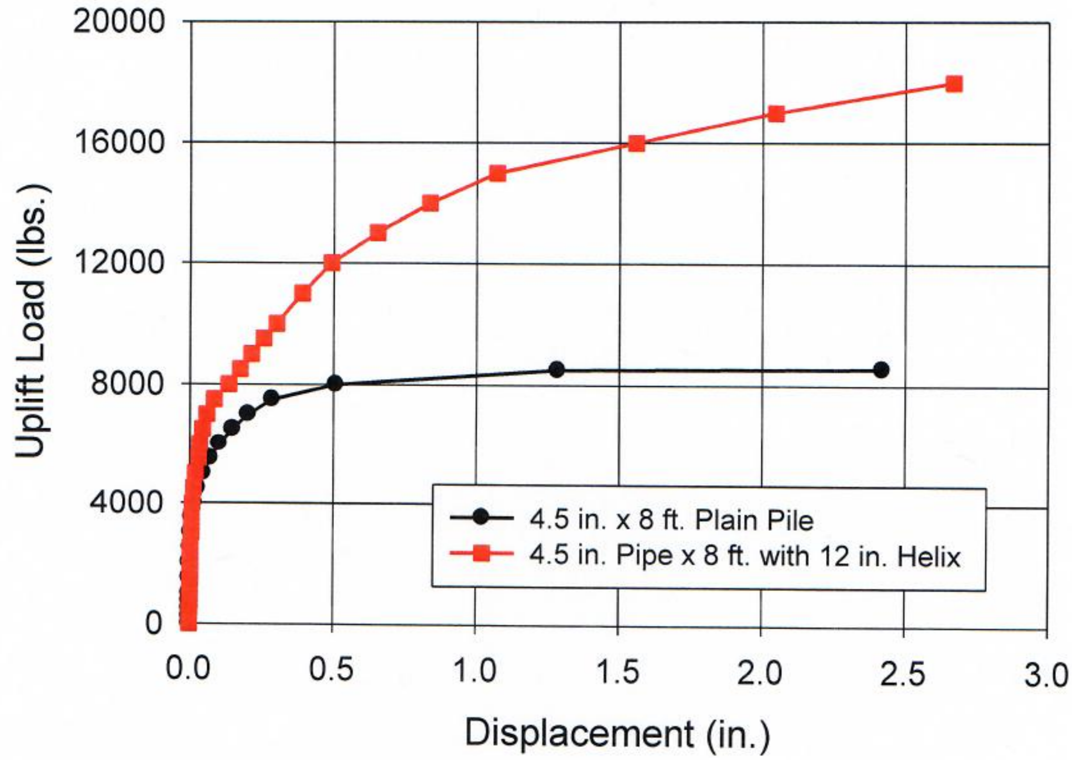

Figure 72. Results of Uplift Load Tests on a Single-Helix Helical Pile and a Plain Pipe Pile of the Same Diameter at the UMsss DOE Site.

The absolute uplift load capacity of helical piles depends on the exact geometry of the pile, number and size of helical plates, depth of embedment and soil type. Figure 73 shows the results of three helical pile uplift load tests conducted on different diameter round shaft piles all equipped with the same diameter helical plate (12 in.) installed to a depth of 8 ft. in sand at the UMass AgFarm Site in S. Deerfield, Ma.

Similar results are shown in Figure 74 for uplift load tests performed in clay at the UMass DOE Site. Results from these tests at both sites clearly show that the uplift capacity increases as the pipe diameter increases and illustrate the importance of the pipe shaft to overall behavior.

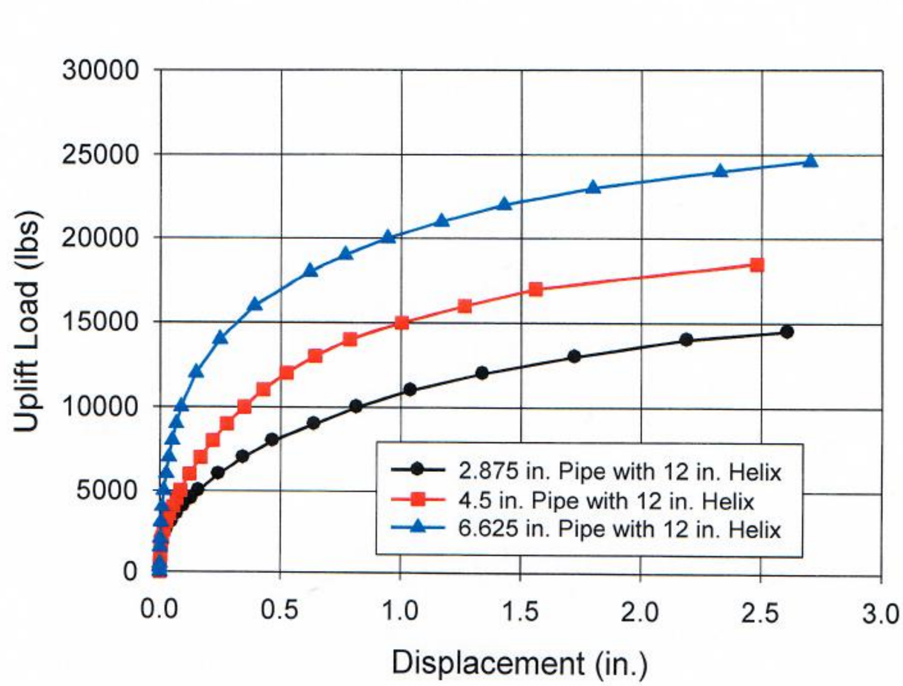

Figure 73. Results of Uplift Load Tests on Helical Piles with Different Diameter Pipe Shafts but Same Diameter Helical Plate in Sand – UMass AgFarm Site.

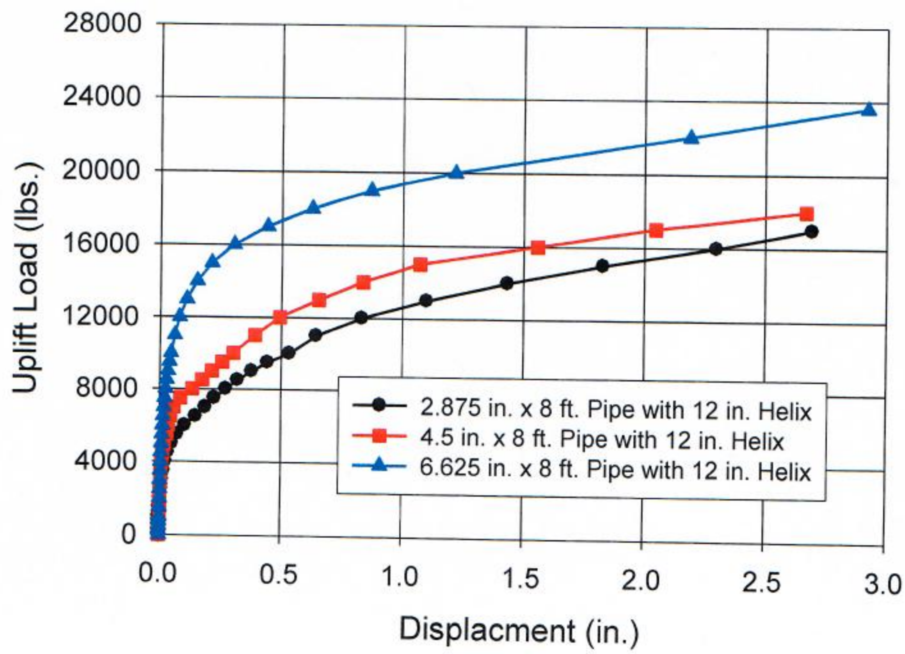

Figure 74. Results of Uplift Load Tests on Helical Piles with Different Diameter Pipe Shafts but Same Diameter Helical Plate in Clay – UMass DOE Site.

Figure 75 shows load test results obtained from a 12 in. diameter drilled concrete pier with a length of 10 in comparison to a single helix helical pile with a pipe shaft diameter of 6 5/8 in. and a helix diameter of 12 in. installed to a depth of just 4 ft. In this case the two foundations give effectively the same uplift capacity of around 10,000 lbs. The drilled pier took about 2 hours to install, not including cleanup, while the helical pile was installed in about 3 min.

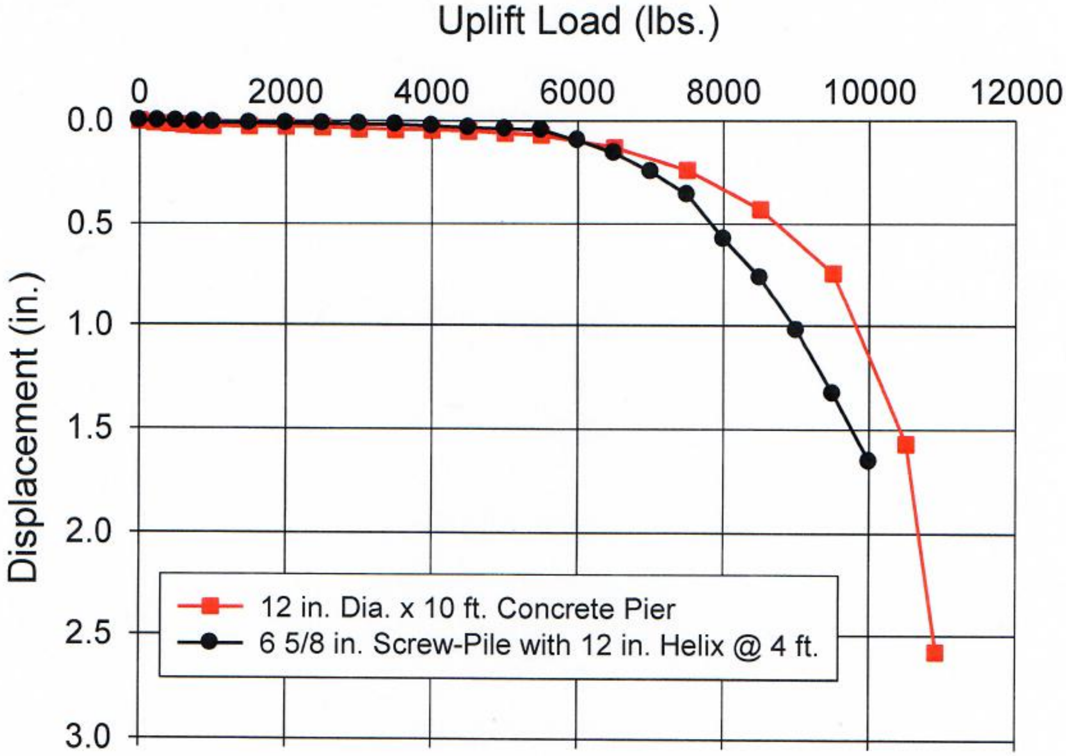

Figure 75. Comparison Between Uplift Load Test of Drilled Concrete Pier and Helical Pile in Clay – UMass DOE Site.

3.8.2 Advantages

Some of the advantages to using helical piles is that they are available in a wide ranges of sizes and lengths; the size and number of helical plates can be adjusted to suit nearly any ground conditions; they can be installed in a few minutes; they produce no soil cuttings and little ground surface disturbance and there is little to no cleanup.

Another principle advantage of helical piles over other foundation alternatives is that the installation may be monitored over the full length by measuring the installation torque at intervals of 1 ft. In this way, the torque profile can be used to verify soil conditions at each installation location. Some Engineers like to equate the installation torque, T, to ultimate capacity, Q_{ult}, via a Torque Factor, K_T, as:

$Q_{ult} = TK_T$ [7]

A number of factors can influence the accuracy of this approach however within a given geology and for a single piece of equipment, hydraulic configuration and pile geometry, this approach is viable.

Installation of helical piles is not influenced by location of the water table as there is no excavation. There is minimal surface disturbance or disruption at the ground surface and there are no soil cuttings to dispose of making site cleanup easy. They may be removed, in the case of temporary installations and may be reused at other sites. The installation is generally very rapid with a 10 ft. helical pile installed in about 3 to 5 minutes.

Once installed, the construction above ground may proceed immediately so there isno delay.

3.8.3 Limitations

For the best load performance the installation should be performed by a qualified installer. It should be noted that helical piles may be difficult to install in soils that contain high amounts of gravel or cobbles. The cost is somewhat more expensive in terms of the actual piles however since the installation is very rapid this may offset the materials cost making the total cost competitive with other foundations.

3.9 Ground Screws

As previously noted, another style of helical pile is a ground screw consisting of a spiral steel thread wound around a central shaft as shown in Figure 76 and 77 Several manufacturers provide ground screws in different sizes, most notably, Krinner from Germany (www.krinnerfoundations.us). Figures 78 and 79 show ground screws delivered to a site and equipment used for installation. No ground screws were evaluated in this work and therefore the uplift load behavior is unknown, however they are another option to support PV systems.

Figure 76. Krinner Ground Screw.

Figure 77. Close-up of Helical Thread on a Krinner Ground Screw.

Figure 78. Ground Screws Delivered to a Site to Support PV Panels.

Figure 79. Track Mounted Equipment Used to Install Ground Screws- Sturbridge, Ma.

3.10 Driven/Vibrated Pile Groups, Helical Pile Groups, Micropile Groups

Rather than use a single foundation element to support solar panels, it may be advantageous to consider the use pile groups. Pile groups consisting of small diameter drilled cast-in-place concrete piers or drilled micropile groups, driven/vibrated pile groups or helical pile groups can be used. In some cases it may be easier for the Contractor to install several smaller foundation elements and then tie them together as a unit as opposed to installing a single larger element. This allows the use of smaller construction equipment which is more economical to mobilize and operate.

3.11 Precast Ballasted Blocks

In some locations the use of precast concrete ballasted foundations is popular. Although not a very common type of foundation, there are some locations where a traditional foundation installed below grade will not be allowed or simply cannot be constructed. For example, most sites that have shallow bedrock will not allow the construction of one of the foundation types previously discussed in this Report. In these situations and alternative is to use as precast concrete base to support solar panels, as shown in Figure 80. Ballasted systems are independent of the subsurface soil and ground water conditions, but probably provide the least resistance to uplift, however this depends solely on the mass of the blocks.

Figure 80. Ballasted Support Using Precast Concrete Blocks – Northfield, Ma.

The bases are manufactured and supplied at a concrete precasting plant and then delivered to the site. They typically have a connection for attaching the pole or column specified by the user to support the solar panel frame. One advantage of using a precast base is that the construction process can go very fast; bases are delivered directly to the site and placed at each panel location. An alternative for users it to create their own concrete base by placing a wood form directly on the ground surface at each panel location and pouring concrete into the form. Ideally, several sets of forms would be used so that there is no waste concrete.

The uplift capacity of a precast or cast-in-place concrete base depends entirely on the mass of the concrete:

$$Q_{TOTAL} = Q_{MASS} \qquad [8]$$

Q_{MASS} (lbs.) = L x W X H x 150 lbs./ft^3

where:
L = Length (ft.)
W = Width (ft.)
H = Height (ft.)

For example, in order to provide an uplift capacity of 3000 lbs., the concrete base would have to have dimensions on the order of 6.5 ft. long x 3 ft. wide x 1 ft. thick.

3.11.1 Advantages

Precast ballasted foundation supports for PV systems may be obtained easily from a precasting manufacturer. There are available in a variety of sizes. They do not require and ground excavation and therefor are suitable to sites with shallow depth to bedrock, cobbles or coarse gravel where other drilled or driven foundations can not be installed.

3.11.2 Limitations

Precast blocks may be expensive and they may have low resistance to overturning.

4.0 FOUNDATION SELECTION FACTORS AND COMPARISON

Selection of a foundation for a ground mount solar system is similar to selection of a foundation for other structures and must consider a number of factors, including: 1). Load Capacity; 2). Site Soil Conditions; 3). Shallow Groundwater Conditions; 4). Site Access; 5). Ease of Construction; 6). Speed of Installation; and 7). Site Cleanup. Table 8 lists a number of primary and secondary factors that should be considered in the selection of an appropriate foundation system to support PV systems.

Table 8. Factors to Consider in Selection of Foundation System.

Primary Factors	Secondary Factors
Site Conditions	Disruption and Disturbance to the Site
	High Groundwater Conditions
	Sensitivity of Site to Soil Cuttings and Site Cleanup
	Site Accessibility for Equipment
Installation	Weather Restrictions
	Speed of Installation
	Availability of Qualified Contractor
Effectiveness	Delay Time Between Construction and Loading
	Load Capacity
	Resistance to Frost Heave
Cost	Cost of Materials
	Cost of Delivery
	Cost of Installation
	Cost of Cleanup

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APPENDIX – Load Testing Procedures

Static tensile uplift load tests were performed for this work using the incremental maintained load "Quick" described in ASTM Standard D3689-90 *Standard Test Method for Individual Piles Under Static Axial Tensile Load.* Load increments were held for 2.5 min. The ultimate capacity was interpreted as the point at which the load could not be maintained with continuous pumping in the case where piles failed by rapid pullout. In cases where the failure was more gradual, the ultimate capacity was interpreted as the last load increment that was applied leading to large displacements, generally in excess of 10% of the pile diameter.