Pressure Grouting Drilled Shaft Tips:
Full-Scale Research Investigation for Silty and Shelly Sands

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Abstract

The tip (toe) capacity of drilled shafts in sands can often be many times greater than the side shear component; however, the tip capacity is often discounted from the total shaft capacity due to the relatively large displacements required to mobilize this end bearing component. Concerns of soil disturbance at the shaft tip (i.e. insitu stress relief) and cleanliness also discourage the use of end bearing as available capacity within any reasonable service load displacement criteria. Pressure grouting the shaft tip has been successfully employed throughout the world as a method of mitigating these conditions. However, there is an apparent lack of ration design procedures and construction guidelines for its use. In cooperation with the Florida Department of Transportation, the University of South Florida is researching the effects of post-grouting on shaft capacity. This paper describes two sites at which full scale shafts have been tested: (1) five shafts, including one control shaft, tipped in a shelly sand, and (2) three shafts, including one control shaft, tipped in a silty sand. The results of these tests show that significant tip capacity improvement can be realized through the use of pressure grouting drilled shaft tips.

Introduction

When designing for drilled shaft capacities in sandy soils, engineers typically must significantly reduce end bearing capacity or even discount it altogether to account for potential soft toe conditions. Even in ideal conditions, end bearing is typically not mobilized before service load displacement criteria are exceeded. The bulk of the capacity is therefore derived from side friction which can be developed with relatively small displacements. This is particularly a problem for larger shafts in cohesionless soils which must displace even further to fully develop tip capacity (e.g. AASHTO tip capacities are based on top of shaft displacements of 5% of the shaft diameter). Consequently, the end bearing strength component, which may be on the order of up to twenty times the side friction component, is unavailable to the shafts useful capacity.

As the end bearing component of drilled shafts is highly under-utilized mechanistic procedures to integrate its contribution have been developed using pressure grouting. Pressure grouting

the tips of drilled shafts has been successfully used world wide to precompress soft debris or loose soil relaxed by excavation (Bolognesi and Moretto, 1973; Stoker, 1983; Bruce, 1986; Fleming, 1993; Mullins et. al, 2000). However, the absence of its use in the United States is probably due to associated uncertainties and the lack of a rational design method. Recognizing this shortfall, the Florida Department of Transportation issued a request for proposals (RFP) to assess the effect of pressure grouting on drilled shaft tip capacity. In June 1999, the University of South Florida commenced a two-year research program to investigate this technique.

The overall objectives of this study were to quantify the improvement that could be developed by pressure grouting the tip of drilled shafts and to develop design guidelines for its use. This paper discusses the results relating to the first objective, the pressure grouting and load testing of drilled shafts in sandy soils. A full discussion of the results of the entire research can be found elsewhere (Mullins, et. al, 2001).

Research Program

The research program outlined below is only a portion of a larger project which also included small scale (1:10) laboratory testing in a frustum confining vessel, computer modeling of pressure grouted drilled shaft tips, and two 1.22 m (4.0 ft.) diameter shafts tipped in cemented coquina. This portion discusses the field results from six 0.61 m (2.0 ft.) diameter drilled shafts cast, pressure grouted, and subsequently load tested. The performance of the pressure grouting is based on comparisons with two additional ungrouted control shafts.

A total of eight shafts were constructed, pressure grouted, and load tested at two sites located on a large property (20 acres) in Clearwater, Florida. Shafts at Site I were tipped in a shelly sand. A dimensioned layout of Site I showing the shaft positions, CPT soundings, and SPT borings conducted within the Site are provided in Figure 1; typical CPT and SPT data are shown in Figure 2. Shafts at Site II were tipped in a silty silica sand. The layout of Site II is shown in Figure 3, while typical CPT and SPT data are shown as Figure 4.

The shafts were each 0.61 m (2.0 ft) diameter and were all approximately 4.57 m (15.0 ft.) long. Five shafts, including one control, were tested at Site I; three shafts, including one control, were tested at Site II. All pressure grouting was conducted using an auger style grout pump. All of the shafts were then load tested using a 4 MN Statnamic device with a hydraulic catch mechanism. The results from the grouted shafts have been compared their respective control shafts to assess the load capacity improvement obtained from the tip grouting process.

Site Investigation

These sites were chosen and the tip elevations set, based upon an exploratory investigation program which looked for the types of soil most likely to be improved with tip grouting (Mullins et. al, 2000), loose to medium dense sands. The sand particles at these two sites span the diverse range that can be encountered in cohesionless soils (shelly to silty). The shelly sand of Site I was very angular and flat, while the silty silica sand was very smooth and round.

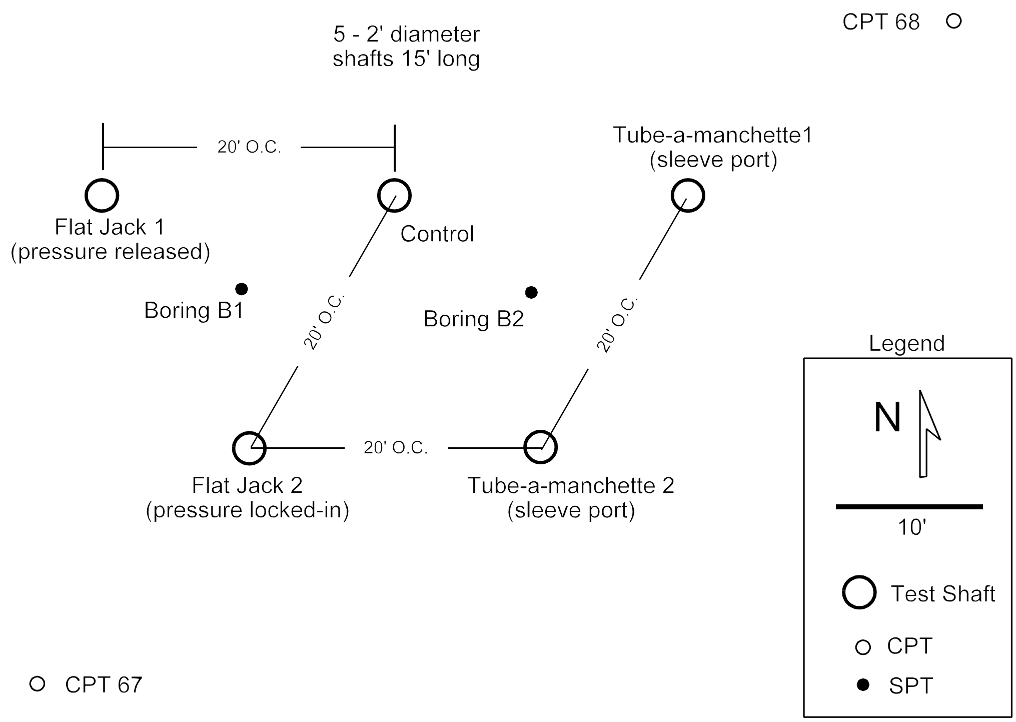

Figure 1. Site I (Shelly Sand) Layout

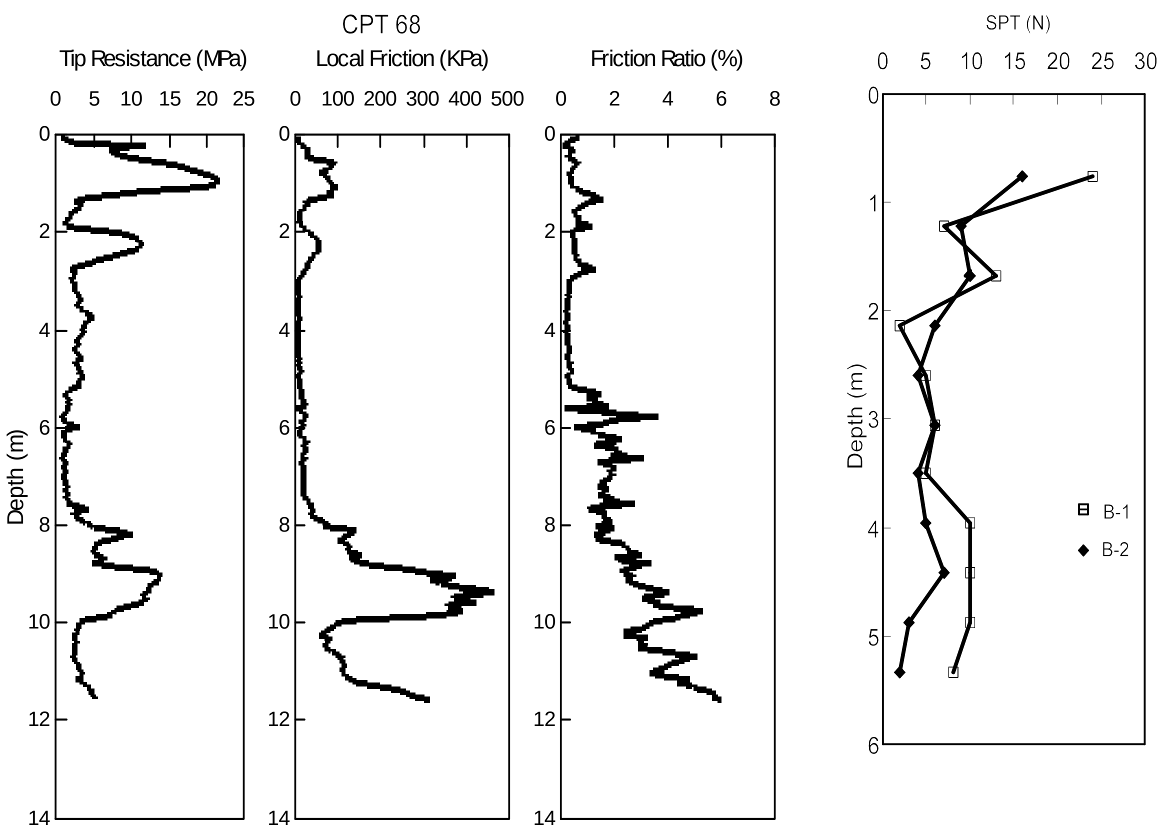

Figure 2. Site I (Shelly Sand), CPT and SPT Data.

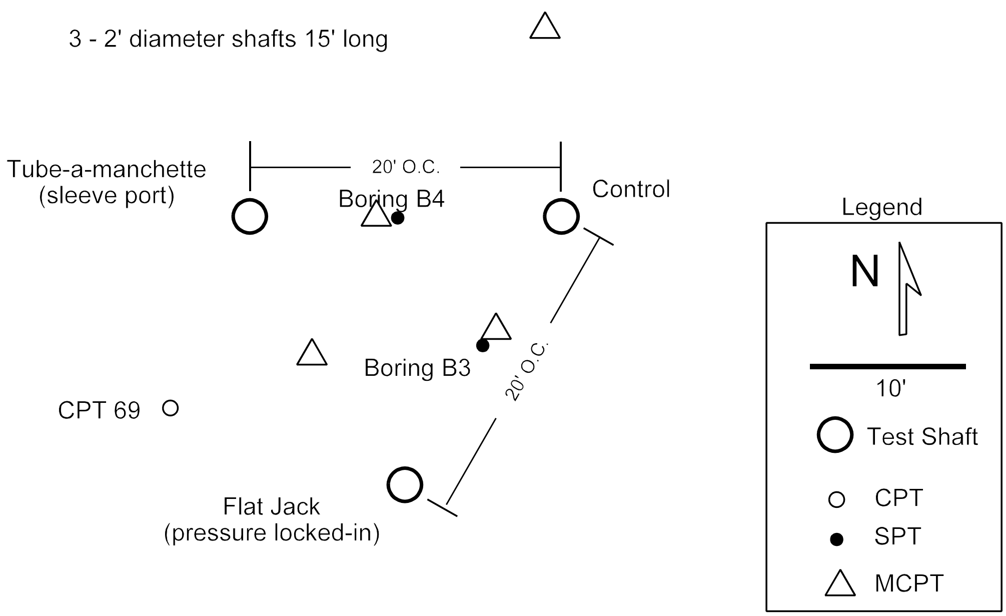

Figure 3. Site II (Silty sand) Layout

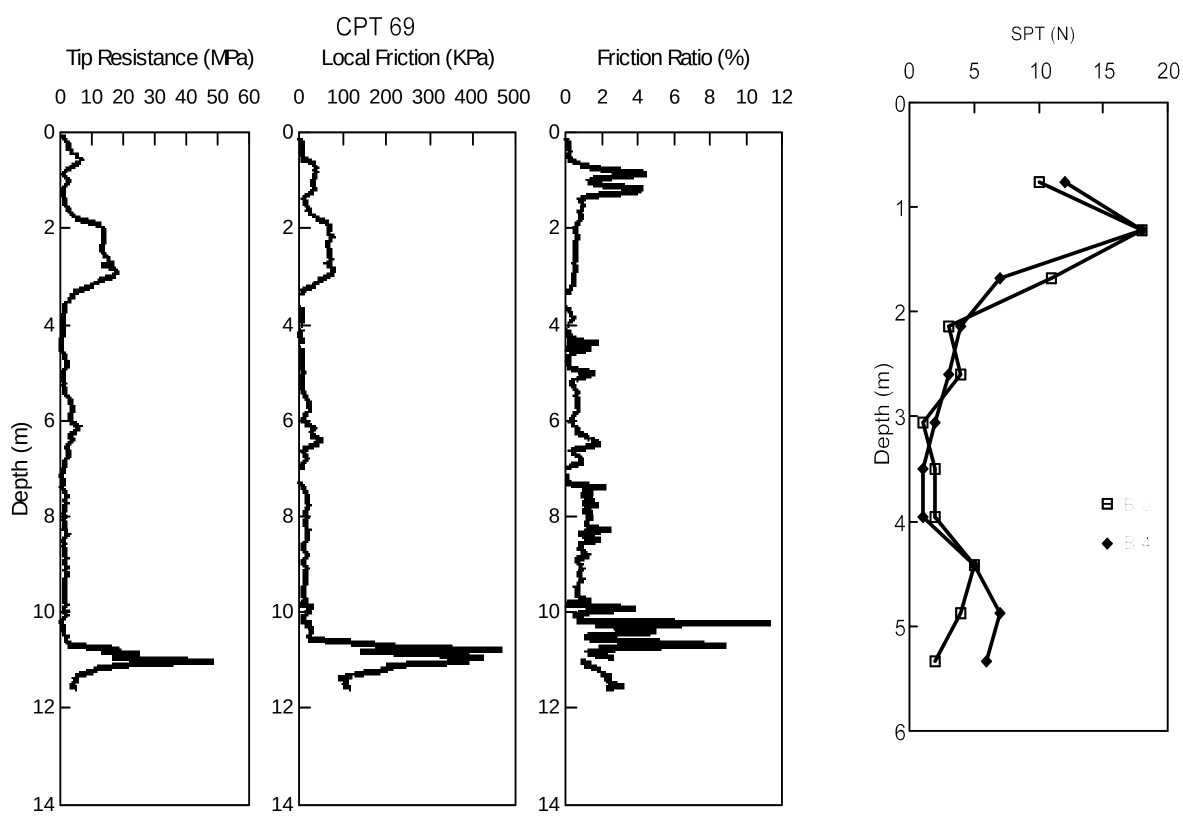

Figure 4. Site II (Silty Sand), CPT and SPT Data.

Grout Distribution Systems

Both flat-jack and sleeve-port type grouting apparatus were tested. A flat-jack consists of a steel plate wrapped in a rubber membrane thus providing a debonded pressurizing surface beneath the shaft tip. Sleeve-port systems use a rubber tube over a perforated pipe section which provides a smaller grouting area, but more flexibility in staged grouting applications. A summary of the grout distribution systems used is contained in Table 1. Two flat-jack devices had the grout pressure locked in during the grout cure, while one did not. Two sleeve-port devices had a steel plate above them, while one did not.

Table 1. Grouting Apparatus.

		Site I (Shelly Sand)	Site II * (Silty Sand)
Control Shaft (no Grouting)		1	1
Flat - Jack	Release Grout Pressure	1	
	Hold Grout Pressure	1	1
Sleeve-Port	With Steel Plate Above	1	1
	No Plate	1	
Total Shafts		5	3

* The two tip grouted shafts of Site II (only), were also skin grouted.

Flat-Jacks. All flat-jacks were identical in construction; however, one in Site I was allowed to release grout pressure immediately after grouting. The remaining two, one in Site I and one in Site II, had the grout pressure locked in with the use of a ball valve in the grout supply lines during the grout cure. Figure 5 illustrates the flat-jack tip grouting apparatus.

A scuff ring was incorporated into the design of the flat-jacks, which protected the 0.8 mm ($^{1}/_{32}$ in.) thick natural gum rubber membrane where it wraps around the edge of the plate. This was to ensure durability in the event that the cage was improperly handled during placement. Further, the scuff ring had tabs attached to the top which allowed for the ring to be bolted to the steel plate, therefore providing a better seal between the rubber membrane and the top plate (rather than just relying on the rubber cement contact adhesive alone). The flat-jack apparatus was tied into the reinforcing cage, and securely fixed in place.

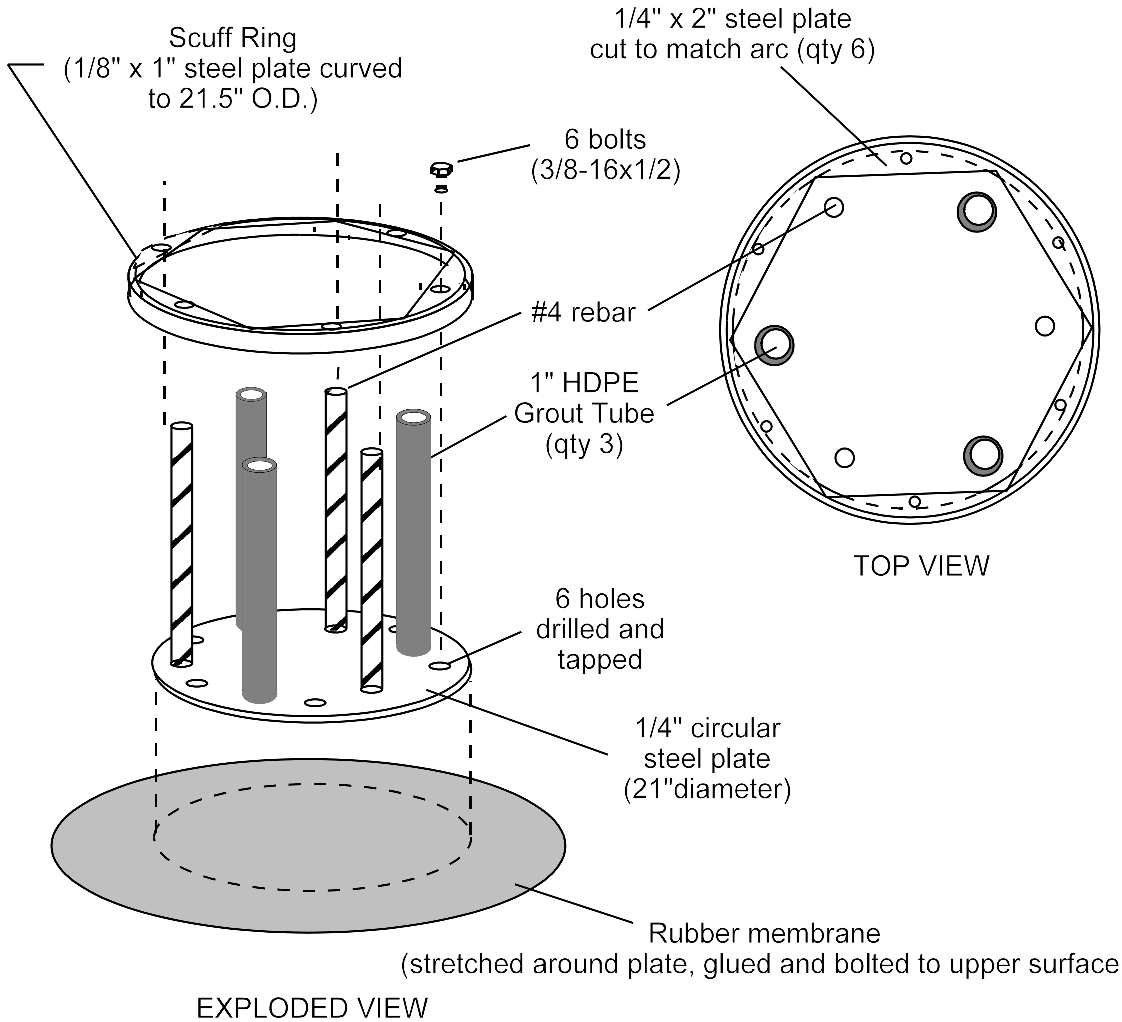

Figure 5. Flat-Jack Grouting Apparatus.

Sleeve-Ports. Sleeve-port systems were identical in construction with the exception that one in Site I did not have a steel plate above it, as is shown in Figure 6. Note that all the sleeve ports locked in the grout pressure due to the nature of their design. The two tip grouted shafts in Site II were also skin grouted, while their respective control shaft was not. The skin grouting of Site II shafts provided more reaction during tip grouting that would not have otherwise been available. Discussion of skin grouting will otherwise not be addressed here.

The grout delivery pipe consisted of a 254 mm (10 in.) section of 19mm ($^{3}/_{4}$ in.) galvanized steel pipe with 7 pairs of diametrically opposed 6mm ($^{1}/_{4}$ in.) grout delivery ports drilled through both pipe walls at any location along the pipe. These 7 sets of ports alternated in circumferential position by 90^{o}, and were equally spaced along the length of the pipe. A natural gum rubber tube (the sleeve) with an initial inside diameter of approximately 24mm ($^{15}/_{16}$ in.), and a wall thickness of 6mm ($^1/_4$ in.) was then pulled over the grout delivery pipe, with the use of soapy water as a lubricant. Alternately, baby powder and compressed air can be used to install the sleeve. The elastic stretch in the rubber tube was needed to seal the grout delivery system during shaft construction, and acts as a one way valve to allow for staged grouting. Often in industry, electrical tape will be wrapped around the edges of the rubber tube to ensure that the pipe is not infiltrated by concrete/cement during shaft construction.

When the option of a steel plate above the sleeve-port apparatus was utilized, each plate had four 38mm (1-$^1/_2$ in.) diameter holes drilled through. The male fitting from the grout pipe would then simply fit through the hole in the plate and screw into the female 90^o elbow ends of the sleeve-port. The system was constructed in this way such that the plate assembly could be tied into the cage via its reinforcing bar tie attachments, and thus its weight (or any forces experienced during placement) would not be supported by the pressure fittings to the sleeve-port.

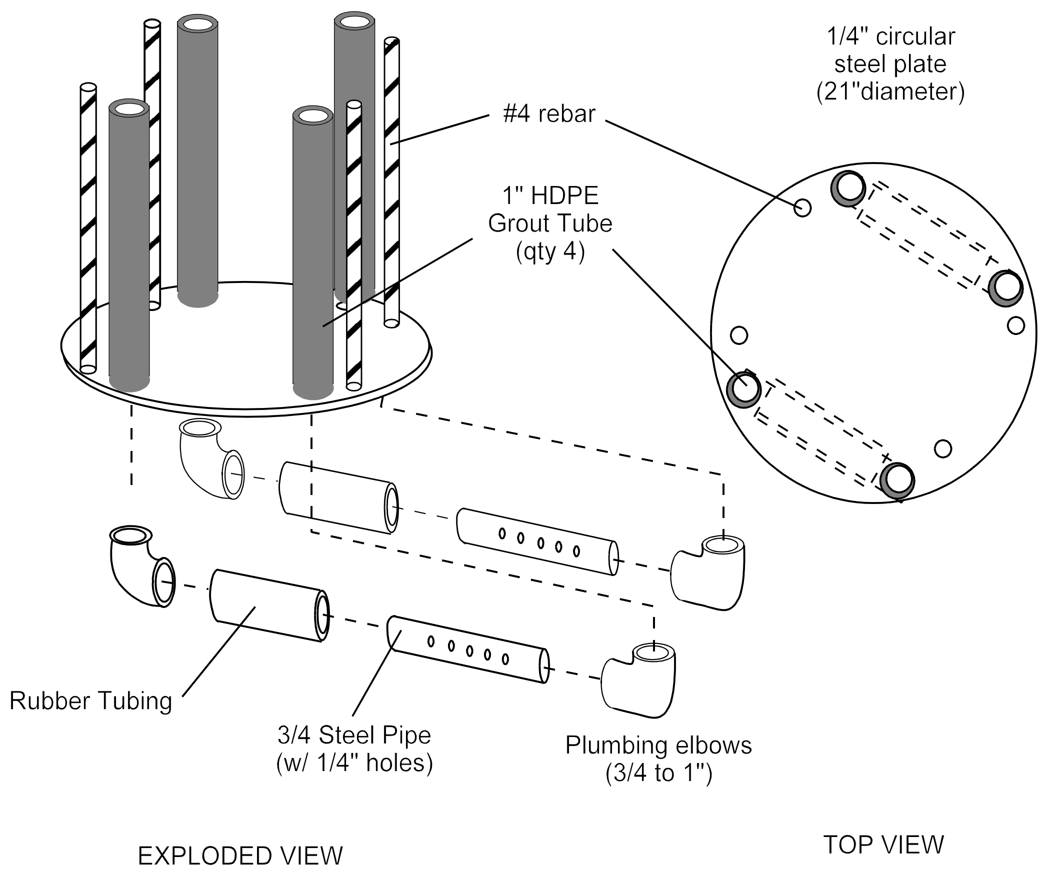

Figure 6. Sleeve-Port Grouting Apparatus (with Optional Steel Plate).

Data Acquisition and Instrumentation

Two separate data acquisition systems were utilized simultaneously during the test, and both uploaded their respective data to a common laptop computer. The two systems used during the test were a MEGADAC manufactured by Optim Electronics, and the Foundation Pile Diagnostic System (FPDS) made by The Netherlands Organization (TNO). A variety of transducers were utilized, in both the grouting and load testing operations. Table 2 shows this system arrangement, and the transducers that were monitored during grouting and load testing operations.

Table 2. Data Acquisition and Instrumentation.

	GROUTING	LOAD TESTING
Data Acquisition System	laptop, Megadac	laptop, FPDS, Megadac
Top of Shaft	LVDT, grout pressure transducer	load cell, accelerometer, laser displacement sensor
Along Shaft Length	strain gages	strain gages
Shaft Tip	strain gages, tension tell-tales	strain gages,

The total load imparted to the top of shaft by the Statnamic device is directly measured by a load cell built into the Statnamic piston mounted to the top of shaft. A laser sensor is also built into this piston which provides a direct measurement of displacement during load testing. Accelerometers are used to confirm displacement and provide direct measurements of acceleration.

Top of shaft displacement during grouting operations was directly measured utilizing linear voltage differential transformers (LVDT's), while bottom of shaft displacements were made with linear cable potentiometers attached to tension tell-tales. Grout pressure was monitored by a pressure gage in-line with the grout supply hose. A rubber membrane and grease pocket within the fitting protected the instrument from the grout.

Strain gages in groups of three were embedded at two levels within the shaft at approximately 0.46 m (1.5 ft.) and 3.2 m (10.5 ft.) from the shaft tip. These strain measurements yield the force at these levels using a composite shaft modulus determined by concrete strengths (concrete cylinder breaks), and the area fraction of steel reinforcement.

The tension tell-tales consisted of a 12mm casing (½ in. schedule 40 PVC pipe), capped at the bottom end, and a stainless steel braided cable strung through the center of the casing. The steel braided cable was secured to the bottom end of the casing through the end cap, clamped, and sealed. Ample length was left outside the end cap and looped to provide development length for tension that would be put upon it during their use in grouting. Significant elastic shaft shortening was not expected during grouting at these sites, nor did it occur.

Shaft Construction

The vertical reinforcement consisted of six 25 mm (# 8) reinforcement bars equally spaced about the perimeter. The stirrup reinforcement consisted of 16 mm (# 5) reinforcement bars, equally spaced every 0.46 m (1.5 ft.). All reinforcing bars used were Grade 60. The finished cages were 4.42 m (14.5 ft.) in length, and had an outside diameter at the stirrups of 0.46m (1.5 ft.).

The grout pipes were made from 25mm (1 in.) high density polyethylene (HDPE) tubing (CTS SDR-9). A 25mm (1 in.) male pipe thread fitting was secured at either end of the grout pipe made of brass (C84-44 1" CTS Compression MIPT Adapter) and had thin stainless steel inserts (#52 Stainless Insert) to keep the tubing from being crushed by the pipe fitting. These materials are common and readily available. This male fitting could suitably connection to either type of grouting apparatus used. This system has a working pressure rating of 1000-1400 kPa (150-200 psi), and was more than adequate for the grouting pressures experienced on this site. This system of continuous rolled tubing was easy to install, and would be especially convenient with long multi-section reinforcing cages. Figure 7 shows completed cages with the various types of grouting apparatus tied in.

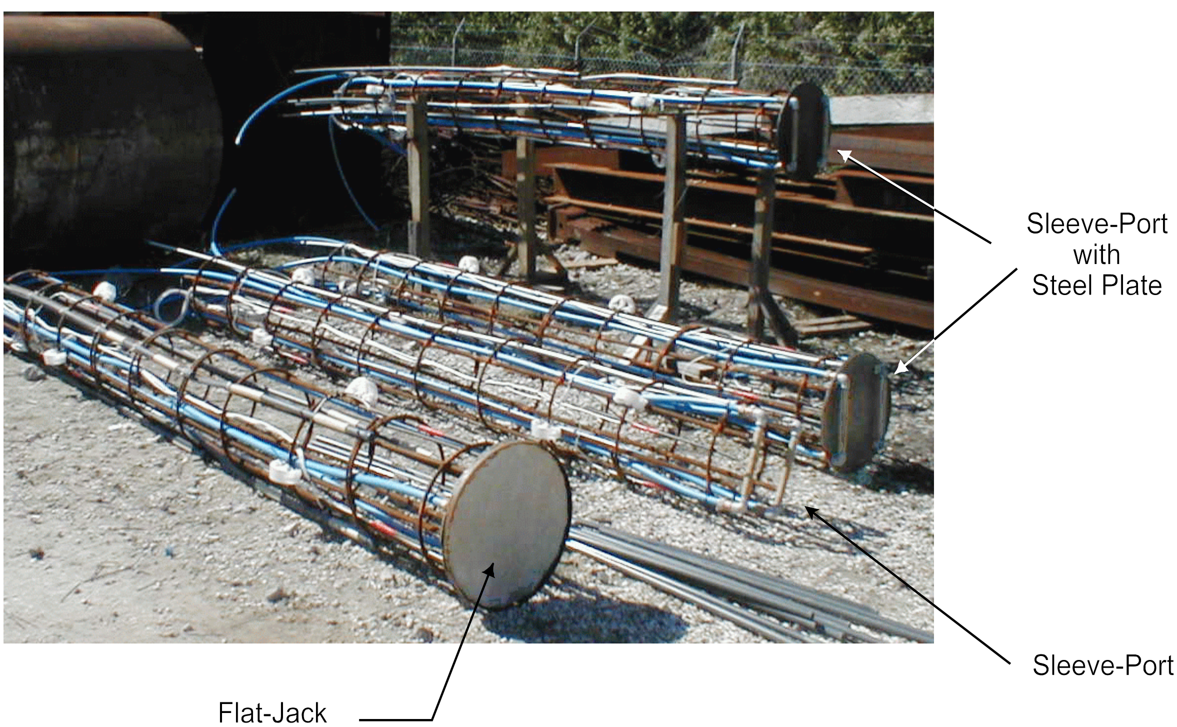

Figure 7. Reinforcing Cages Complete with Instrumentation and Grouting Apparatus.

A truck mounted Texoma 700 drill rig was utilized to excavate the shafts of Site I, while a track mounted BG-7 drill rig was used to excavate Site II. Both were excavated using a synthetic slurry. Both Site I and II utilized the same drill auger in construction. A clean-out bucket was not used during construction as it would not produce a significant improvement in tip condition, nor would the use of standard down-hole or air-lift pumps. Even under ideal conditions it is essentially impossible to thoroughly "clean out" any shaft excavation at the tip, as is customary with rock socketed tips, as any action taken simply tends to further upset the excavation. A benefit of tip grouting is that this condition is mitigated, and is a point that this study confirmed. A Hitachi 60 ton track mounted hydraulic crane was used to pick and set the cages.

Site I Flat-Jack 2 (locked-in pressure) was set nearly 0.3 m (1.0 ft.) too high due to a tight tolerance between the plate and borehole. Careful observations of the grouting of this shaft confirmed that 51 liters (1.8 $ft.^3$)of grout was needed to fill the void under the flat-jack plate before any grout pressure above that required to pump the grout through he lines could be detected. This represents a volume that would be equivalent to a column the same diameter as the flat-jack by 0.22m (0.7 ft.) high. As a direct result, an unanticipated variable was tested between the two flat-Jacks of Site I; the effect of varying amounts of soft debris and/or voids below the flat-jack grouting apparatus. Results show that this condition was mitigated by the tip grouting procedure, this grout volume of 51 liters (1.8 $ft.^3$) is not included in the subsequent analysis.

Grouting and Load Testing

The grouting at these sites was then accomplished using a helical style grout pump and paddle type mixer, as shown in Figure 8. The pump was adequate, as shaft uplift or grout volume proved to be the limitation at all test shafts. In all cases the shaft concrete was allowed to gain sufficient strength before grouting operations commenced. The grout mix utilized was a water-cement slurry (Type I and II cement) with a water to cement ratio of 0.5. The grouting lines of both sleeve-port and flat-jack apparatus were gently flushed with grout before the return lines were capped and grout pressure was applied.

Prior to any grouting work, the sleeve-port apparatus on these sites were "burst" open using water pressure. This was done the day following concrete placement, as is common practice, such that the concrete had set up, yet had not gained significant strength. The intent of this action is to open a path for subsequent grouting, which may be performed at a later date when the shaft has reached acceptable strength. The volumetric flow during this process should be minimal to reduce the soil disturbance. The helical style grout pump was first used; however a small widely available pressure washer was better suited for this task.

The load testing was carried out with the use of a 4 MN Statnamic device after the grout was given time to obtain sufficient strength. A minimum of three load cycles were performed on each test shaft. The hydraulic catch mechanism made reloading of the shaft proceed rapidly, as all three load cycles would typically occur within a 30 minute time span. In all cases the test shafts were displaced many times more than the ultimate capacity displacement, such that the load vs. displacement response would be fully defined. Figure 9 shows a test in progress.

Figure 8. Trailer Mounted Grout Mixing and Pumping System.

Figure 9. Statnamic Load Test in Progress.

Results

Grout pressure and shaft displacement were directly measure as previously described. The grouting rate should be slow enough to build pressure, and not simply hydrofracture the soil. The grout consistency can be adjusted to control this aspect. The maximum sustained grout pressure, peak upward displacement, and total grout volume for each of the grouted shafts are listed in Table 3. In general, the flat-jacks have a lower grout take than the sleeve-ports due to the flat-jacks acting as a confined pressure cell. Conversely, the grout from the sleeve-ports is in intimate contact with the surrounding soil matrix, and thus has a greater potential to migrate away from the immediate tip area.

Table 3. Pressure Grouting Data Summary.

Site	Grout Delivery Mechanism	Shaft Designation	Grout Volume liters (ft.^{3})	Grout Pressure kPa (psi)	Shaft Uplift mm (in)
Site I (Shelly Sand)	Flat-Jack	S1-FJ1 (release press.)	50 (1.75)	586 (85)	3.78 (0.149)
		S1-FJ2 (hold press.)	107 * (3.79)	462 (67)	4.83 (0.190)
	Sleeve-Port	S2-SP1 (with plate)	165 (5.82)	1138 (165)	2.74 (0.108)
		S2-SP2 (no plate)	86 (3.05)	1220 (177)	1.42 (0.056)
Site II (Silty Sand)	Flat-Jack	S2-FJ (hold press.)	217 (7.65)	683 (99)	3.81 (0.150)
	Sleeve-Port	S2-SP (with plate)	180 (6.34)	862 (125)	1.23 (0.043)

* Does NOT include grout volume to fill void left under this flat-jack apparatus.

The "bottom-up" load displacement response of a shaft is obtained when combining these time traces with the load based upon strain gage data, as is consistent with bi-directional load testing procedures. The measured displacement is the shaft uplift, and the load is the side friction component, which is equal but opposite to that of the end bearing component. In order to compare the grout pressure readings to this curve, a shaft tip load is also calculated by multiplying the pressure reading by the shaft tip cross-sectional area.

Typical shaft response during grouting utilizing flat-jack apparatus is presented as Figure 10. The shaft tip loads calculated by means of both the strain gage and grout pressure data correspond extremely well to each other, even at the beginning of the grouting cycle. This is attributed to the flat-jack apparatus design allowing the grout to disperse rapidly across the

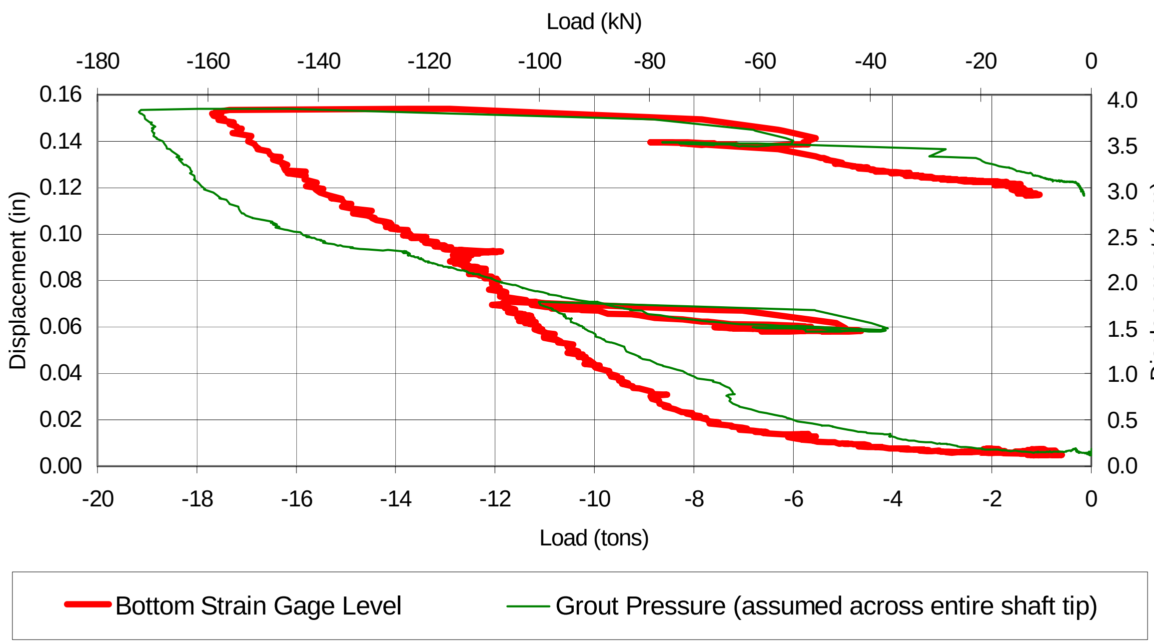

Figure 10. Typical Flat-Jack Tip Grouting Load vs. Displacement.

entire shaft tip between the top plate and rubber membrane. This illustrates the strong advantage of a flat-jack apparatus in a production setting providing a "proof test" of the shaft capacity. The load displacement response of every grouted shaft can be obtained by simply utilizing top of shaft displacements and grout pressure. If the shafts are significant in length, the elastic shortening must be considered.

An ungrouted control shaft, was constructed and load tested at both Sites I and II, such that the improvement in load capacity due to pressure grouting could be assessed by direct comparison to its respective control shaft. Figure 11 presents a typical shaft load comparison. Both the top and tip of shaft response is shown for both a grouted and ungrouted control shaft.

As most tip resistance designs are based on an assumed displacement of 5% of the shaft diameter, (e.g. Reese and O'Neill, 1988), the improvement in shaft tip and total capacity is evaluated at displacements of 5% of the shaft diameter for all the test shafts, as summarized in Table 4. The grouted improvement is defined by the following:

$$\%\ Improvement = \frac{(Capacity_{grouted\ pile} - Capacity_{control})}{Capacity_{control}} * 100\%$$

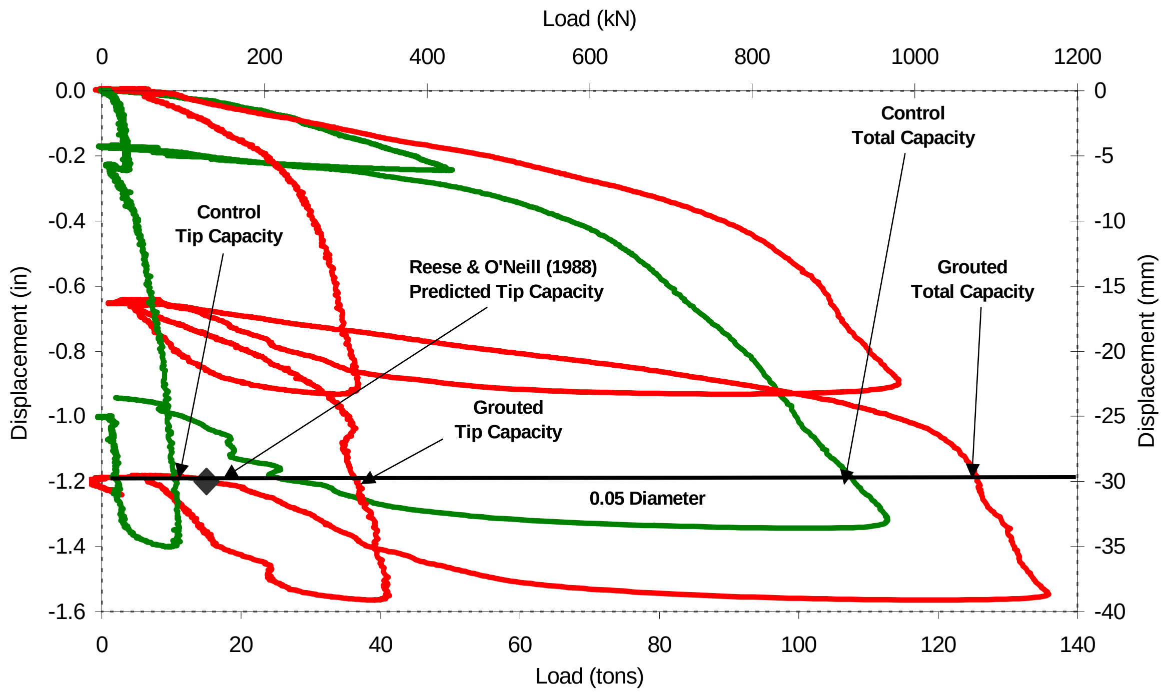

Figure 11. Load Test Comparison of Typical Grouted Shaft to Control Shaft.

Table 4. Load Performance at a Displacement of 5% Shaft Diameter.

Site	Grout Delivery Mech.	Shaft Desig.	Total Load kN /(tons)	Tip Load kN /(tons)	Tip Contrib. (%)	Tip Improve. (%)	Total Improve. (%)
Site I (Shelly Sand)	Control	S1-CON (no grout)	961 (108)	98 (11)	10	N/A	N/A
	Flat-Jack	S1-FJ1 (release press.)	1094 (123)	347 (39)	32	255	14
		S1-FJ2 (hold press.)	1210 (136)	365 (41)	30	273	26
	Sleeve-Port	S2-SP1 (with plate)	1379 (155)	507 (57)	37	418	44
		S2-SP2 (no plate)	1450 (163)	569 (64)	39	482	51
Site II (Silty Sand)	Control	S2-CON (no grout)	890 (100)	62 (7)	7	N/A	N/A
	Flat-Jack	S2-FJ (hold press.)	1290 (145)	463 (52)	36	643	45

The improvement for the shaft tip capacity is further evaluated in Figure 12 where it is plotted for all measured displacements until ultimate capacity is reached. The greatest amount of tip improvement occurred in Site I at a displacement of only 2% of the shaft diameter, 12 mm (0.48 in.), and in Site II at a displacement of 4% to 5% of the shaft diameter, 24 mm (0.96 in.) to 30 mm (1.20 in.). Undoubtedly, this was due to the tip grouting locking in some amount of negative side shear, and thus more readily transferring the subsequent load to the shaft tip which was also prestressed in compression. Although the total shaft improvement is more modest, the increased stiffness of the grouted shaft aides in meeting service limit displacements.

Summary

The field results of a two year, full-scale load testing program have been presented identifying the magnitude of improvement that can be obtained by pressure grouting drilled shaft tips. Three grout distribution systems were used: (1) flat-jack, (2) sleeve-port, and (3) sleeve-port and plate. A total of eight test shafts were constructed and tested at two sites, Site I (shelly sand) and Site II (silty sand). All grouted shafts showed increased tip capacity, *regardless of distribution system*, when compared to ungrouted control shafts. Sleeve-port systems developed higher grout pressures, while flat-jack systems produced more uniform tip loading and higher uplift displacements. The tip improvement (at 0.05 shaft diameters of displacement) for sleeve-port systems was 455% and 843% for Sites I and II, respectively. Flat-jack systems produced 264% and 643% improvement, similarly. Although not presented herein, a design procedure has been developed based on the results of this research which is being incorporated into the FDOT design guidelines.

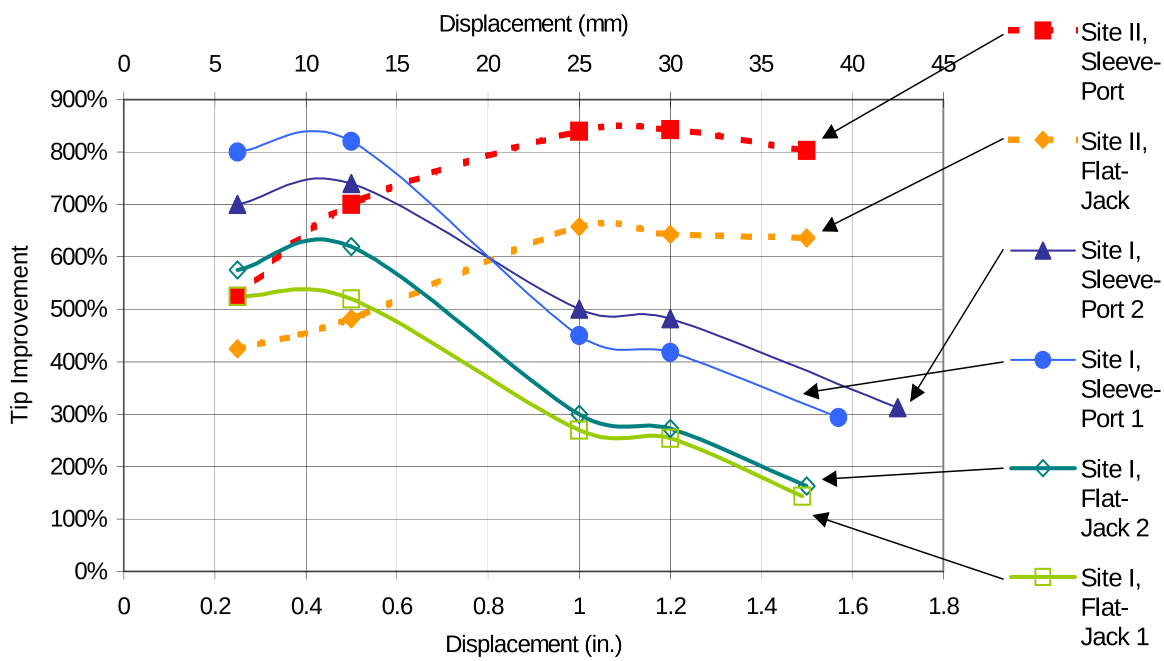

Figure 12. Grouted Tip Capacity Improvement vs. Shaft Displacement.

Acknowledgments

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