

Pressure-Grouting Drilled Shaft Tips in Sand

Gray Mullins¹, Steven D. Dapp², and Peter Lai³

Abstract

Considering the strain incompatibility between end bearing and side friction of drilled shafts, tip capacity is often discounted from total shaft capacity. This is due to the relatively large displacements required to mobilize the tip which often exceed service load displacement criteria. Additionally, concerns regarding shaft tip soil disturbance (i.e. insitu stress relief) and toe cleanliness further discourage designers from using end bearing as available capacity. As a method of mitigating these conditions, pressure-grouting the shaft tip after its construction has been successfully employed throughout the world. With very few exceptions, the benefits of tip-grouting have been disregarded in the United States. Sources of skepticism arise from the uncertainty of the grout formation beneath the tip and the lack of rational design procedures 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 in loose to medium dense sands. This paper presents a review of past and present base-grouting methods used throughout the world and the scope of on-going full-scale load test programs.

Introduction

The use of drilled shafts as structural support has recently increased due to heightened lateral strength requirements for bridge foundations and the ability of drilled shafts to resist such loads. They are particularly advantageous where enormous lateral loads from extreme event limit states govern bridge foundation design (i.e. vessel impact

¹Assistant Professor, Department of Civil and Environmental Engineering, University of South Florida, Tampa, FL 33620

²Graduate Researcher, University of South Florida, Tampa, FL 33620

³State Geotechnical Engineer, Florida Department of Transportation, Tallahassee, FL

loads). Additional applications include high mast lighting, cantilevered signs, and most recently, cellular phone and communication towers. With respect to bridge construction, design procedures, both axial and lateral, have been additionally impacted where increased unsupported pile lengths are mandated by scour depth predictions based on 100 year storm events. This dramatically changes driven pile construction where piles cannot be driven deep enough without overstressing the concrete or without pre-drilling dense surficial layers. In contrast, drilled shaft construction is relatively unaffected by scour depth requirements and the tremendous lateral stiffness has won the appeal of many designers. However, drilled shaft design and construction is plagued with quality control issues (e.g. shaft bottom cleanliness or open excavation time) not experienced during pile driving.

Typically, designers have chosen to significantly reduce end bearing capacity or even discount it altogether to account for soft toe conditions. Even in ideal conditions, full 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 especially problematic for larger shafts which must displace even further to fully develop tip capacity in loose to medium dense cohesionless soils where unit side friction values are comparably low with respect to competing foundation systems. Consequently, the end bearing strength component, which may be on the order of up to twenty times the unit side friction, is unavailable to the useful capacity of the shaft (AASHTO, 1997). In an effort to mitigate shaft toe cleanliness and also balance the useful capacity between end bearing and side friction, projects throughout the world have implemented pressure grouted shaft tips after normal shaft construction (sometimes termed “post-grouting”). An overview of pressure grouting considerations will be presented herein as well as efforts underway to develop recommendations for its use by the Florida Department of Transportation (FDOT).

Background

In the early 1960's, efforts began to obtain more usable tip capacity of drilled shafts using pressure grouting below the shaft tip. In 1975, Gouvenot and Gabiax presented results of a test program where post-grouting large diameter piles led to increased ultimate load capacities up to three times in sands and clays. As a result, post-grouting techniques have become a routine construction process in many parts of the world (Bruce, 1986). The post-grouting process entails: (1) installation of grout pipes during conventional cage preparation that run to the bottom of the shaft reinforcement cage, and (2) after the concrete in the shaft has cured, injection of high pressure grout beneath the tip of the shaft which both densifies the insitu soil and compresses any debris left by the drilling process. By essentially preloading the soil beneath the tip, end bearing capacities can be realized within the service displacement limits.

Although the performance of a drilled shaft is bounded by the maximum contribution of end bearing and skin friction components, these values are not fully realized due to flaws introduced by full scale construction techniques. Three

mechanisms, or combinations thereof, are responsible for the excessively large shaft displacements required to develop bearing capacity:

- Strain incompatibilities typically exist between the end-bearing and side friction components in relation to service displacement criteria. The ultimate side frictional component develops with relatively small shaft displacements compared to the displacements required to mobilize ultimate end bearing. Development of the side friction component can be 50% of ultimate at displacements of approximately 0.2% of the shaft diameter (D) (AASHTO, 1997), and fully developed in the range of 0.5 to 1.0 % D (Bruce, 1986). In contrast, mobilization of the end bearing component can be 50% mobilized at 2.0% D (AASHTO, 1997), and fully mobilized in the range of 10 to 15 % D (Bruce, 1986). The end bearing component therefore requires 10 to 30 times more shaft displacement in order to mobilize the same percentage of its ultimate value as the side shear component. This means that the side friction is strained beyond its ultimate strength and into a residual state by the time the end bearing capacity is realized. In addition, the service load deflection criteria is often exceeded long before any significant amount of end bearing can be developed.
- The pile toe zone is often disturbed by normal construction procedures. This disturbance can occur by soil stress relaxation due to excavation of the overburden, inflow of groundwater due to insufficient hydrostatic head or rapid removal of the excavation tool during the construction process. This soil disturbance of the pile toe zone by normal construction procedures is often difficult or nearly impossible to eliminate. Displacements necessary to overcome this disturbance and mobilize end bearing are usually in excess of allowable service limits. In instances of less competent soil, this problem is further compounded.
- Construction methods and processes may leave soft debris/deposits at the bottom of the excavation. Primary contributing factors are: overall shaft bottom cleanliness, a non-uniform distribution of toe debris causing an initially reduced shaft area bearing on the soil, excessive sand content in the drilling fluid, prolonged time for cage and concrete placement, and deposits of drilling fluid itself at the bottom. These construction related factors may then also be the cause of excessive deflections required to mobilize end bearing due to toe inclusions not evident in an otherwise clean excavation.

Depending on soil type and drilling method, any or all of the above mechanisms may occur at a given excavation. However, each scenario can be mitigated by a procedure, relatively unused in the United States, where post-grouting is performed beneath the shaft tips. This grouting concept accommodates the trend towards large diameter drilled shafts due to lateral load considerations, while allowing for the end bearing component to contribute to the useful capacity of the shaft.

Soil Type Applicability

End bearing strata may be grouped into three broad categories in relation to the

process of post-grouting pile tips. These categories are cohesionless soils (sands to silts), cohesive soils (clays), and soft or fractured rock formations. Although all soils can be improved to some degree by grouting techniques, the applicability and effectiveness of grouting, primarily compaction grouting, is many times more effective in cohesionless soils than other soil types (Baker and Broadrick, 1997). Historically, nearly all of the studies and construction projects involving grouting of the pile tips to increase end bearing have been in cohesionless soils.

Sand and Silt. The first effective large scale grouting of pile tips was performed in sandy soils in 1961 at the Maracaibo Bridge (Sliwinski, 1984). Since then many studies and construction projects have proven the extreme benefits of post-grouting the pile tips in cohesionless soils (Piccione in Cairo, 1984; Sliwinski and Fleming, 1984; Logie in Jakarta, 1984; Stocker in Jeddah-Mecca Expressway, 1983; and Bauer in Brooklyn, NY, 1988). In general, results have shown that post-grouting the pile tips in cohesionless soils has significantly increased end bearing capacities. Figures 1, 2 and 3 show the effectiveness of post-grouted shafts.

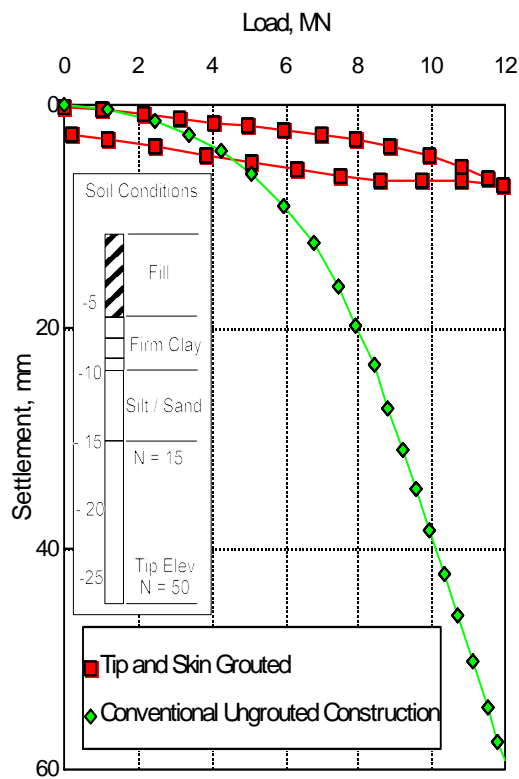


Figure 1. Comparison of two 1.5m diameter drilled shafts (Sliwinski, et al., 1984).

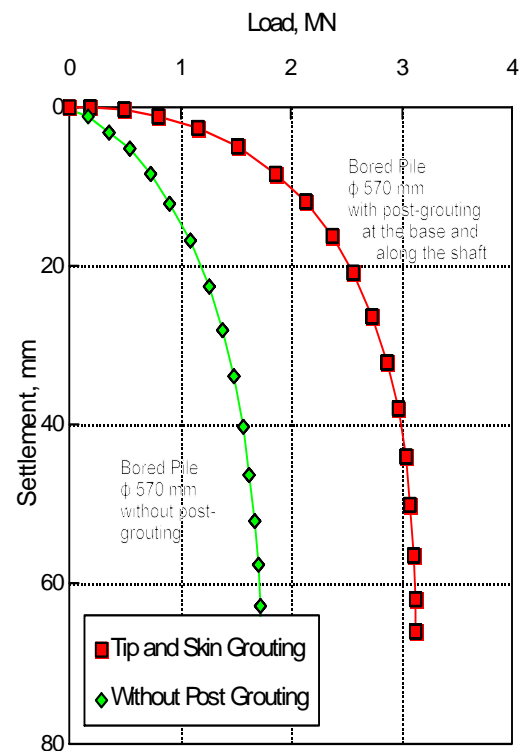


Figure 2. Load-displacement of 570 mm bored piles (Stocker, 1983).

Loose to medium dense sands hold the highest potential for increase in useable shaft end bearing. This is due to this soil profile being the most susceptible to the three

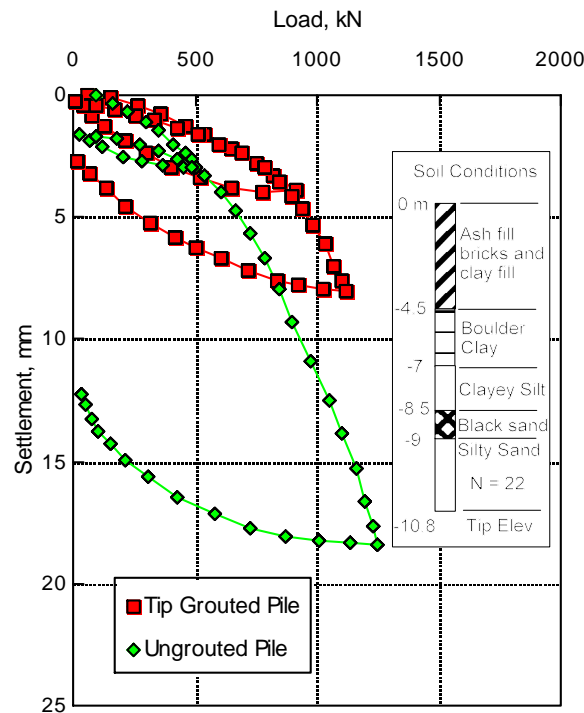


Figure 3. Results from 450 mm shaft load tests (Sliwinski, et al., 1984).

Two different grouting methods, permeation grouting and compaction grouting, are applicable to these soils. The permeation grouting can easily create a very large grout bulb, and compaction grouting can dramatically improve the soil stiffness. Both processes can be done with the use of ordinary cementitious grout.

Dense sands can be both permeation and compaction grouted with cementitious grout in the same manner as loose sands. However, a micro-fine cement may become necessary for permeation grouting, and may not yield significant improvement over compaction grouting alone. The grout volume used in dense sands would be significantly less. Bruce (1986) reviewed many cases to state that there is a direct relationship between ultimate load increase and volume of cement grout injected for all sands; when grouting dense sands the grout volume simply corresponded to the void volume of the gravel pack (discussed later, Figure 8).

Sandy silts can be densified by means of applying effective stresses during compaction grouting with ordinary cementitious grouts, although it is less effective than compaction grouting of clean sands. Permeation grouting in silty soil, however, would involve the use of chemical grouts, such as a silica gel, and is beyond the scope of this paper.

Although disturbance to the shaft toe area during construction is of little practical importance in soft rocks and clays (Sliwinski and Philpot, 1980), Sliwinski and Fleming

(1984) concluded that in sands the end bearing contribution to the total load capacity is extremely sensitive to construction induced soil disturbances. Therein, full scale load testing was used to verify the effectiveness of pressure grouting for mitigating these conditions.

Clay. Post-grouting in clay produces only a minimal gain in end bearing governed by the amount of consolidation that can occur within the set time of the grout. The high pressures introduced by this method may only result in hydrofracture of the soil matrix. Careful consideration would be needed so that the allowable end bearing contribution, even after grouting, would not exceed the creep limit of the clay at the grout bulb/soil interface. The most effective way of grouting in clay material would be to jet-grout, or deep-soil-mix under the shaft tip. While these are certainly viable options for remediating deep foundations in this soil type, it is not the focus of this paper.

Rock. Grouting of fractured and soft rock formations with low strength grout in order to fill voids, fractures, seams, and solution channels is sometimes conducted to alleviate drilling problems associated with karst topography. However, this is usually accomplished prior to drilling, and is not the grouting technique that is discussed herein. These formations typically are incapable of consolidation or densification by effective stresses induced by compaction grouting. Further, permeation grouting of the macro inclusions is effectively accomplished by the concrete head during normal construction, as is evidenced by high concrete over-runs in such cavernous strata.

Although grouting can effectively mitigate soft toe conditions caused by excessive construction debris/deposits at an excavation bottom for all soils, current quality control procedures for drilled shaft production already effectively address shaft bottom cleanliness for clay and rock during normal construction. Thus, only a marginal benefit would be realized in these conditions through the use of post-grouting. An exception may be where shaft bottom cleanliness is problematic due to extreme depths and time requirements such as the My Thuan Bridge Project, Vietnam (Dapp, 1998) or for cases where the capacity of shafts already constructed fall short of adequate (Logie, 1984).

Post-grouting can be effective in all soil types; however, research shows the greatest performance gain in cohesionless soils. As such, an ongoing study at the University of South Florida, Tampa is concentrating on the effectiveness of post-grouting in cohesionless soils (both sands and silt) with an emphasis on identifying the most effective grouting techniques. Effectiveness is evaluated by: (1) the final strain compatibility of the tip and skin friction components, (2) constructability, and (3) overall capacity gain.

Uplift Considerations

In general, upward movement of shafts during compaction grouting should be limited such that the frictional strength of the shaft is not developed beyond its ultimate

value and into a lesser residual value. This is most critical in dense sand where there is a pronounced loss of frictional resistance with large strains (Figure 4). Historically, uplift criteria have been limited to ranges from 2 mm (Stoker, 1983) to 20 mm (Bolognesi and Moretto 1973). Presently, in Taipei, a 3mm uplift criterion is in effect (Mullins, 1999). It is unclear, however, if these criteria were placed only on top-of-shaft movement or if the tip movement associated with elastic compression was also given a maximum permissible movement. Long shafts such as those in Taipei (80 m) can exhibit relatively large displacements at the tip without being detected at the top (and vice versa).

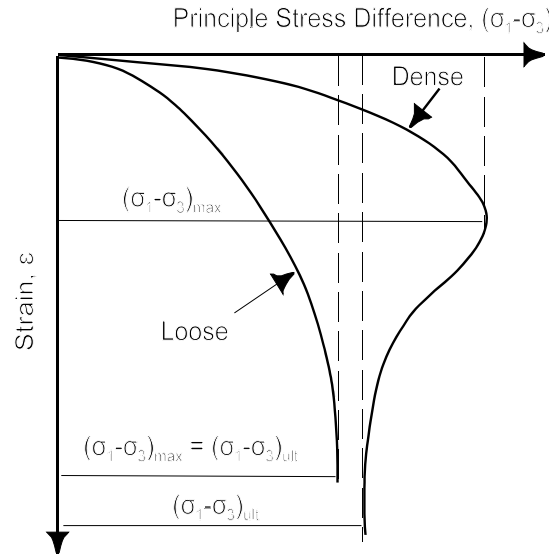


Figure 4. Stress / strain curves for typical loose and dense sands (Holtz and Kovacs, 1981).

Essentially, the maximum amount of end bearing improvement is dependent on how much downward resistance the side friction component of the shaft can provide. As such, post-grouting can also be applied to the sides of the shaft to improve unit side friction values. This aides in providing downward restraint during the tip-grouting process (resisting uplift). This is of particular importance for shorter shafts, and as a consequence skin grouting has been employed to aide in providing reaction (e.g. Bauer system of pile grouting). Additional criteria of maximum grout volume (per stage) and minimum grout pressure are established based on reasonable cavity expansion and the anticipated tip performance, respectively. Figure 5 shows grout pressures that have been used on various sites throughout the world in relation to the shaft tip depth.

Grouting Types

Standard grouting techniques can be divided into two basic categories: permeation grouting and compaction grouting. Staged grouting procedures are often designed which have a combination of these two, first permeation and then compaction. There are also state-of-the-art techniques available for cohesive soils, such as jet

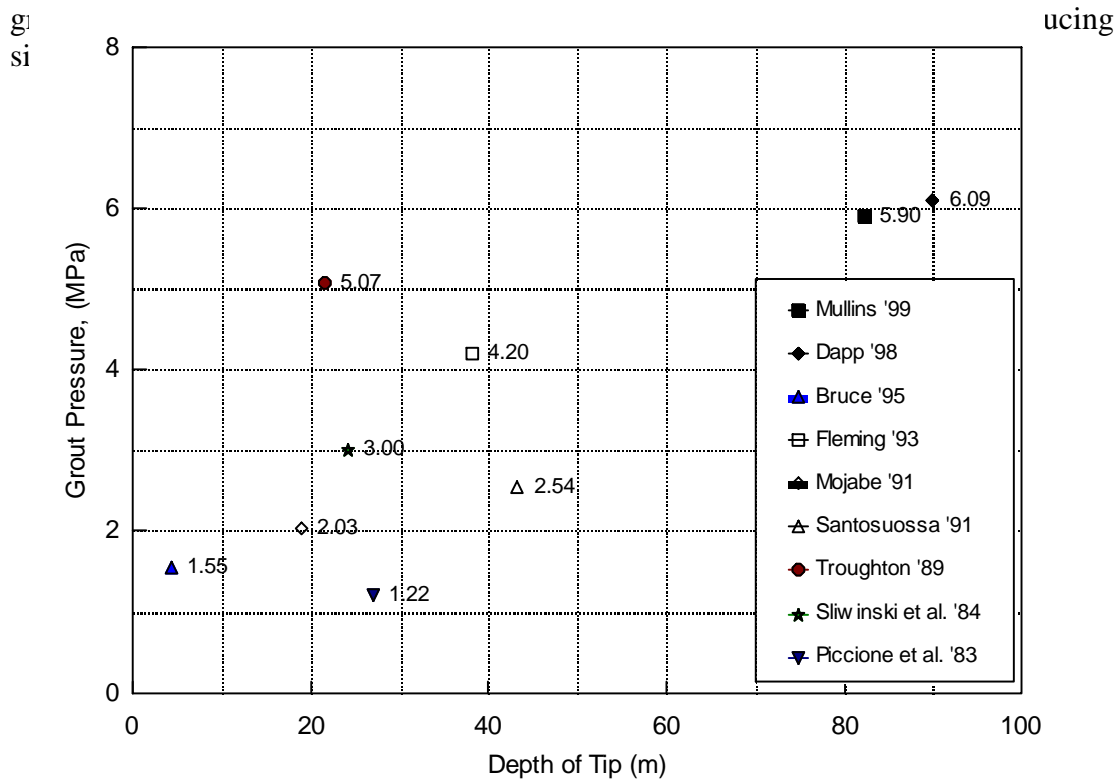


Figure 5. Tip grouting pressures used at various sites worldwide.

Permeation Grouting. Permeation grouting uses a fluid grout which is highly mobile within the soil formation, and therefore travels through the void spaces without providing any significant compaction or densification of the surrounding soils. In this manner a very large zone of improved soil below the pile tip is developed. Careful adjustment of the water-to-cement ratio is used to control the mobility of the grout. The type of grout mix design is also crucial to achieve this mobility. For example Littlejohn (1983), at the Jeddah - Corniche Centre, first tried remediating substandard piles with the use of a cement grout in a dense sand profile interbedded with hard sandy silt. However, grout takes were very low, and the remediation technique failed. Subsequently, a low viscosity resorcinol formaldehyde grout was successfully used.

Compaction Grouting. In contrast to permeation grouting, compaction grouting utilizes a thick, viscous, homogeneous, typically cementitious mass designed to remain together within the soil matrix. Generally there is a distinct interface between the soil and grout material, thus the insitu soil is consolidated and densified by cavity expansion of the grout bulb (Baker and Broderick, 1997). A typical compaction grouting mix design is shown in Table 1. However, this mix is recommended for minimum grout pipe diameters of 100mm.

Table 1. Typical Compaction Grout Mix (Baker and Broderick, 1997)

Description	Quantity	Standard	Comment/Effect
Sand	800-1000 kg	ASTM C-33	Well graded, rounded edge, min. 15% passing 0.075 mm sieve
Cement	110-225 kg	ASTM C-150	Control strength of mix, increase density of mix
Flyash *	90-310 kg	ASTM C-618	Improve pumpability, increase density, reduce cement content required for mix, Class F or Class C
Water	60-160 L		Control slump
Admixtures (optional)	1%-2% of cement		Control set time, control shrinkage

* Depending on the fines available from the sand.

Compaction grouting develops its own "filter cake" at the soil/grout interface which differs from the Bauer system of grouting where a mechanical grouting system uses steel plates and an impermeable cover or a liner embedded between the plates (discussed later). In either case, the mechanism of soil improvement is the same; the grout applies an effective stress to the soil, thus densifying it. A notable difference is that the simple compaction grouting (i.e., with a filter cake) is a uniform stress case at the pile tip, whereas the mechanical compaction system of steel plates provides a uniform strain condition. A potential benefit of compaction grouting the shaft tip is that this procedure could provide a means of proof testing shaft tip capacity during the compaction grouting procedure.

Presently, a testing program is underway to quantify the effectiveness of pile tip grouting to improve end bearing in loose to medium dense sand. Therein, simple compaction grouting will be compared to compaction grouting with a mechanical compaction cell, and a combination of permeation and compaction grouting.

Injection Techniques

Grouting techniques vary in the mechanism by which the grout is dispensed beneath the shaft tip. Variations include whether to use:

- stem, orifice, tube-a-manchette, or mechanical distribution system
- a gravel pack beneath the tip to aide in distribution of grout
- fixed or floating distribution system
- permeation, compaction, or a staged combination.

Two basic distribution systems are mainly used: (1) *simple compaction grouting* in which the tube-a-manchette system employs a network of exposed grout tips, and (2) the

mechanical grouting system in which the Bauer-type system of one or two steel plates with an impermeable membrane is used. Although both systems can be used in a wide variety of soils, the membrane-type mechanisms minimize hydro-fracture grout losses more common with tube-a-manchettes used in weakly layered soils.

The tube-a-manchette has several variations, but is primarily a simple pipe network across the bottom of the shaft pre-drilled along its length on the bottom face and connected to grout tubes to the top of shaft. The pipes are wrapped in a rubber membrane at the location of the holes to prevent blockage of grout passage during normal shaft construction where the tubes become completely encased in concrete. A problem with fixed tube-a-manchette systems is that the grouting must be accomplished immediately after the concrete has set (24 to 48 hours), while its strength is still low enough to burst the encapsulation. A simple tube-a-manchette system fixed to the cage and resting on the bottom of the excavation was used in the shafts supporting a major cable stay bridge in Thailand in 1985, as shown in Figure 6. A similar configuration has recently been used for the foundations supporting the cable-stay bridge over the Mekong River in Vietnam (Dapp, 1998). Presently, at the Taipei Financial Center project in Taiwan, an adaptation is being employed that closely contours the pipe network to the shaft bottom (Figure 7). The shape resembles the reverse circulation cutting tool and minimizes the concrete cover between the grout pipes and the shaft bottom.

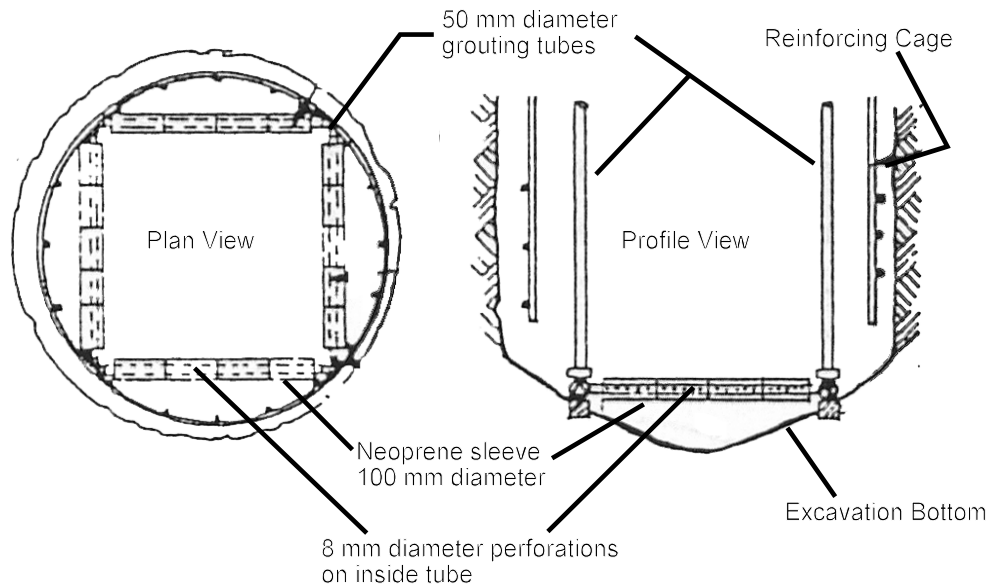


Figure 6. Simple tube-a-manchette compaction grout apparatus (after Bruce, 1986).

Complications can arise if the excavation depth is lower than the tube-a-manchette elevation. In such instances, the grout pressure is unable to break the encapsulation and modify the soil. Such was the case when the first 90 m deep excavation in Vietnam was inadvertently over-excavated by 0.5m by way of extensive

clean out procedures. This caused the tube-a-manchette to be embedded in an extra 0.5m of concrete. To avoid this problem, floating tube-a-manchettes were used for subsequent shafts which used slip joints allowing the distribution system to adjust to the actual bottom of excavation elevation. Other systems have used flexible grout hoses to overcome this problem. It is thus recommended that suspending the tube-a-manchette, by either method, should be considered necessary for proper steel placement of extremely long shafts where cage length and excavation depths may not be consistent.

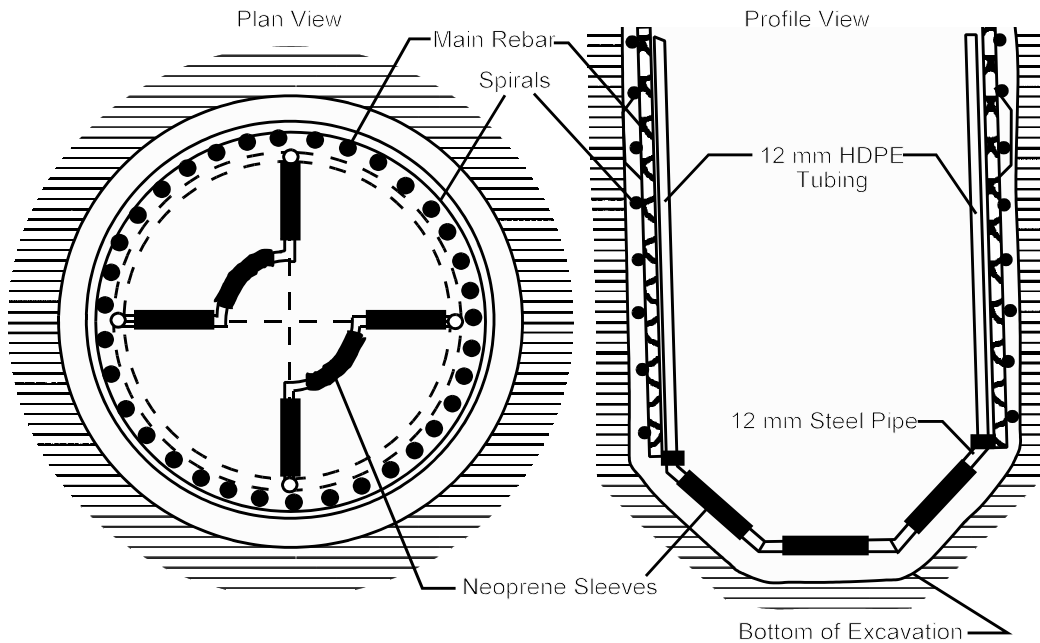


Figure 7. Sliwinski and Fleming (1984) described first placing a gravel plug in the excavation, then the tube-a-manchette with a steel plate above (both suspended from the cage). This configuration is shown in Figure 8. The steel plate has the benefits of isolating the tube-a-manchette and gravel plug from the concrete so that the post-grouting process can take place after the concrete has gained design strength, the tube-a-manchette is protected from the tremie during concreting operations, and the steel plate gravel interface provides a consistent bearing surface for the compaction grouting pressure to act against (important for proof testing aspects). The gravel is beneficial for both permeation and compaction grouting by exposing more soil interface to the grout, as well as providing aggregate to knit the soil bulb together directly below the shaft tip.

Lizzi (1981) discussed a mechanism consisting of two steel plates separated by mechanical spacers (to allow grout pressure to initially act upon the full face of the

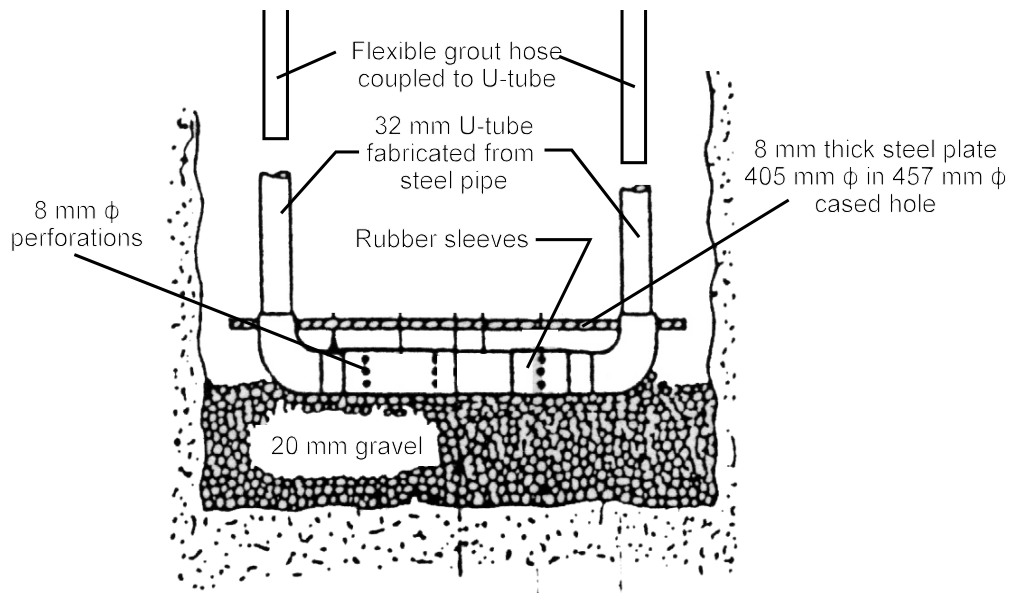


Figure 8. U-shaped grouting cell positioned at bottom of excavation (after Sliwinski plates). This technique is similar to the Bauer system of tip grouting used on the Jeddah-Mecca Expressway (Bruce, 1983) and the Brooklyn Queen's Expressway (1988). The difference is that Lizzi had the plates covered with an impermeable liner to ensure that separation of grout injection ports and concrete was maintained (see Figure 9). The impermeable liner ensured that no permeation into the surrounding soil occurred.

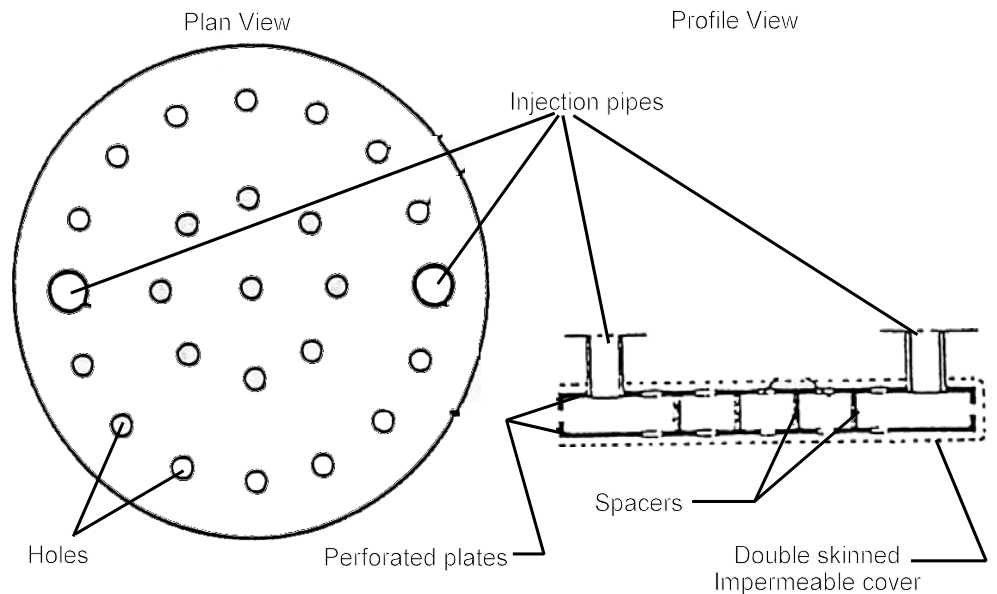
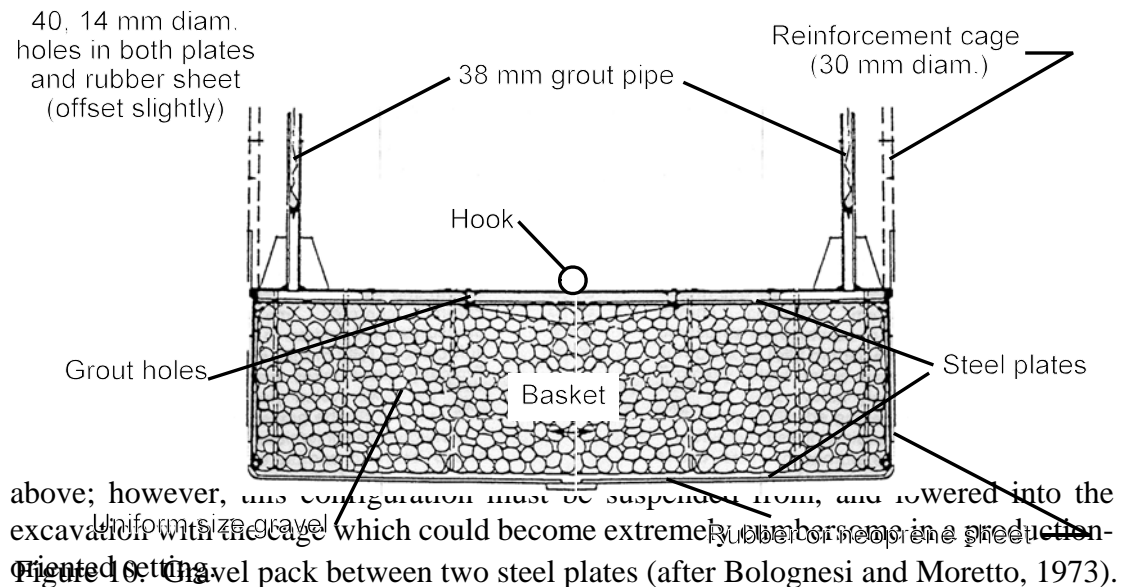


Figure 9. Mechanical compaction grout apparatus (from Lizzi, 1981)

Consideration can also be given as to whether a gravel pack should be included between

the two plates, as was discussed in the early work by Bolognesi and Moretto in Paranah River (1973), shown in Figure 10. The benefit of the gravel pack again is as stated

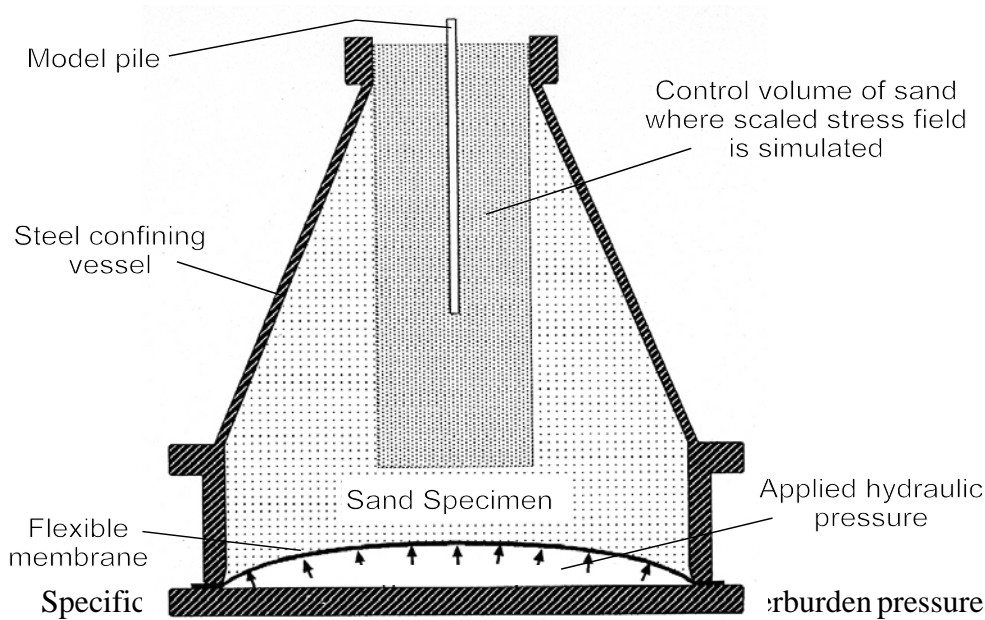


Scope of Testing and Research

To investigate the various mechanisms of post-grouting and evaluate their effects on shaft performance, the University of South Florida, Tampa is presently conducting concurrent laboratory and full-scale testing on post-grouted drilled shafts. The laboratory component is looking at parameters such as: grout bulb formation, strength gains, grout mechanisms, and residual stress states after grouting; whereas the field component is addressing issues such as constructability, applicable mechanisms, mechanism durability, maintaining production, as well as strength gain and design recommendations.

The laboratory testing is being conducted using a relatively new device called a Frustum Confining Vessel (FCV) which provides a method of physically modeling pile-type insitu stresses on small-scale piles without the use of a centrifuge. The device, developed by Berminghammer Foundation Equipment in conjunction with McMaster University (Sedran, 1999), is a conical-shaped steel vessel in which sands are placed and stressed as shown in Figure 11. The resulting vertical and horizontal stress distributions are reasonably similar to those of full-scale prototype piles. In the control volume portion of the FCV stresses are distributed similar to those encountered in the field. Some significant advantages of the FCV with respect to other physical modeling

methods are the simplicity of testing, relatively low cost, and its ability to model relatively large model piles (e.g. 1 m long, 100 mm diameter).



burden pressures that provide adequate reaction to pre-load the soil beneath the tip. Furthermore, the effects of scaling the size of the shaft will require additional foundation mass to restrain the upward movement of the shaft. Although scaling parameters require close attention to conduct meaningful load testing, the initial goal is merely to show the formation of the grout bulb at various relative densities for the sand. Subsequent load testing of post-grouted model shafts in the FCV is an added benefit that will afford interesting results.

The performance of post-grouted shafts is largely dependant on the strain compatibility of the tip and skin resistance. Additionally, for post-grouting to be fully beneficial, the soil must be returned to an unstressed state at the completion of the grouting process to remove locked-in stresses from negative skin friction. The benefit from post-grouting is therefore derived from the improved stiffness associated with reloading. The effects of residual stress are being investigated by maintaining various grout pressures during the curing of the grout and then testing the capacity of these model shafts. Instrumentation is included within the model shafts to confirm the state of stress locked into the soil.

The full scale portion of the program presently involves two sites where drilled shafts are being installed with post-grouting mechanisms. The first site has eight relatively short shafts, 5 meters long, installed in loose to medium dense silty sand that will be used to investigate both the effects of tip grouting as well as skin grouting. Four

shafts will be installed with only tip grout mechanisms (similar to those in Figures 6, 8, and 9), two with tip and skin grouting, and two with no grouting systems as a controls. Figure 12 shows three variations of grouting mechanisms currently in use at the University of South Florida test site.



The second site has three proposed test shafts in sand and limestone involving five 30MN load tests. Due to the variable nature of the site, post-grouting has been selected for shafts tipped in sand to reduce their length. Two of the shafts will be tested prior to post-grouting and then subsequently after post-grouting for direct comparison. Figure 12. Post grouting mechanism used at the first site. The third shaft will only be tested without grout effects. The tip grout mechanism will be selected based on the performance evaluation of the previous site. All shafts from both sites will be fully strain instrumented and continuously monitored to determine the distribution of load and the presence of residual stresses throughout the load test/post-grout/load test procedures.

Summary

Pressure grouting drilled shaft tips (post-grouting) has been successfully employed throughout the world for over thirty years with surprisingly little use within the United States. Recently, the FDOT has contracted research to investigate the parameters affecting its performance and to develop recommendations and guidelines for its use on Florida roadway projects. The method provides a means by which to mitigate many of the factors that presently exclude the contribution of the end bearing from the useful shaft capacity (e.g. toe cleanliness). Presently, researchers at the

University of South Florida are examining many of the considerations designers will need to address for its eventual use. By including significant end bearing contributions into the useful capacity, the design of drilled shafts can be drastically improved. Moreover, this inexpensive procedure could directly provide test results for every shaft installed.

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