Drilled Shafts

Introduction

Drilled shafts are deep, cylindrical, cast-in-place concrete foundations poured in and formed by a bored (i.e. "drilled") excavation. They can range from 2 to 30 feet in diameter and can be over 300 feet in length. The term *drilled shaft* is synonymous with cast-in-situ piles, bored piles, rotary bored cast-in-situ piles, or simply shafts. Although once considered a specialty foundation for urban settings where vibrations could not be tolerated or where shallow foundations could not develop sufficient capacity, their use 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 loads). Further, relatively new developments in design and construction methods of shafts have provided considerably more economy to their use in all settings (discussed in an ensuing section on post grouting drilled shafts). Additional applications include providing foundations for high mast lighting, cantilevered signs, cellular phone and communication towers. In many instances, a single drilled shaft can replace a cluster of piles eliminating the need (and cost) for a pile cap.

With respect to both axial and lateral design procedures for water crossing bridges, all foundation types and their respective designs are additionally impacted by scour depth predictions based on 50 or 100 year storm events. Scour is the removal or erosion of soil from around piles, shafts, or shallow footings caused by high velocity stream flows. It is particularly aggravated by constricted flow caused by the presence of numerous bridge piers. The scour-mandated additional foundation depth dramatically changes driven pile construction where piles can not be driven deep enough without over stressing the piles or without pre-drilling dense surficial layers. Similarly, the increased unsupported length and slenderness ratio associated with the loss of supporting soil can affect the structural stability of the relatively slender pile elements. In contrast, drilled shaft construction is relatively unaffected by scour depth requirements and the tremendous lateral stiffness has won the appeal of many designers.

Construction Considerations

The design methods for drilled shafts presented in this chapter are largely based on empirical correlations developed between soil boring data and measured shaft response to full-scale load tests. In that the database of test cases used to develop these correlations included many different types of construction, these methods can be thought to address construction practices. In reality, most of the design methodologies are extremely conservative for some types of construction and only mildly conservative for others. The construction of drilled shafts is not a trivial procedure. Maintaining the stability of the excavation prior to and during concrete placement is imperative to assure a structurally sound shaft. Various methods of construction have been adopted to address site-specific conditions (e.g. dry or wet drilling; slurry type; cased or uncased; tremie placed or free fall concrete). All of these approaches as well as the fresh properties of the concrete can affect the load carrying capability of the finished shaft. It is important that the design engineer be familiar with drilled shaft construction methods and can assure that good construction practices are being used.

Dry / Wet Construction. *Dry* construction can only be performed in soil formations that are inherently stable when cut (e.g. clay or rock) and where ground water is not present. Any intrusion of ground water into the excavation can degrade the structure of the surrounding soil and hence reduce the capacity of the shaft. In situations where the ground water is present and likely to intrude, some form of wet construction should be used. *Wet* construction implies that a slurry is placed in the excavation that is capable of maintaining a net positive pressure against (or flow into) the walls of the excavation. The slurry can be mineral, synthetic, or natural.

Mineral slurries consist of a bentonite or attapulgite clay premixed with water to produce a stable suspension. As mineral slurries are slightly more dense than water, a 4 - 6 ft head differential above the ground water should be maintained at all times during introduction and extraction of the drilling tool. This head differential initially causes a lateral flow into the surrounding soil which is quickly slowed by the formation of a bentonite (or attapulgite) filter cake. Soil particles can be easily suspended in this slurry type for extended periods of time allowing concrete placement to be conducted without significant amounts of debris accumulation. However, no more than 4% slurry sand content is permitted in most States at the time of concreting.

Synthetic slurries consist of a mixture of polymers and water that form a syrupy solution. A 6 - 8 ft head differential should be maintained at all times during the introduction and extraction of the drilling tool when using a synthetic slurry. This head differential also causes lateral flow into the surrounding soils, but a filter cake is not formed. Rather, the long strings of the polymer stabilize the excavation walls by clinging to the soil as they flow into the soil matrix. As such, the flow remains relatively uniform and generally will not slow. The soil typically falls out of suspension relatively quickly when using synthetic slurries which permits debris to be removed from the bottom in a timely fashion.

Natural slurries are nothing more than readily assessable water (ground water, lake water, or salt water). An 8 - 10 ft head differential should be maintained at all times during introduction and extraction of the drilling tool when using a natural slurry. This head differential causes a lateral flow into the surrounding soil which is fast enough to induce outward lateral stress sufficient to maintain the excavation stability. Although it is possible to use this method in granular soils, it is not recommended nor is it permitted by most State agencies. Slight pressure differentials induced by tool extraction can cause local excavation wall instabilities. As such, this method is most commonly used when excavating clay or rock where the ground water is likely to be present. The above slurry types and the time the slurry is left in an excavation can affect the capacity of the finished shaft (Brown, 2000). To minimize these effects, local specifications have been imposed largely based on past performance in similar soils (FDOT,2002).

Casing. Wall stability can also be maintained by using either partial or full length casing. A casing is a relatively thin walled steel pipe that is slightly larger in diameter than the drilling tool. It can be driven, vibrated, jetted, or oscillated (rotated) into position prior to excavation. The purpose of the casing is to provide stability to weak soils where slurries are ineffective or to bring the top of shaft elevation to a level higher than the surface of free standing bodies of water. When stabilizing weak soils the casing is often temporary being removed after concreting. Shafts constructed over water must use permanent casing that can be removed after the concrete has fully cured. The method of installing and removing temporary casings can also affect the capacity of the finished shaft. Oscillation removal can increase side shear over vibrated or direct extraction methods. Quickly extracted casings can induce necking due to low pressure developed at the base of the extracted casing.

With the exception of full length temporary casing methods, the practical upper limit of shaft length is on the order of 30D (i.e. 90 ft for 3ft diameter shafts) but can be as much as 50D in extraordinary circumstances using special excavation methods.

Concreting and Mix Design. Drilled shaft concrete is relatively fluid concrete that should be tremie placed (or pumped to the base of the excavation) when using any form of wet construction to eliminate the possibility of segregation of fine and coarse aggregate and/or mixing with the insitu slurry. A tremie is a long pipe typically 8 - 12 inches in diameter used to take the concrete to the bottom of the excavation without being altered by the slurry (i.e. mixing or aggregate segregation). Prior to concreting, some form of isolation plug should be placed in-line or at the tip of the tremie to prevent contamination of the concrete flow as it passes through the initially empty tremie. During concrete placement, the tremie tip elevation should be maintained below the surface of the rising concrete (typically 5 - 10 ft). However, until a concrete head develops at the base of the excavation, the potential for initial mixing (and segregation) will always exist. In dry construction, free-fall concrete placement can be used although it is restricted by some State agencies. The velocity produced by the falling concrete can induce higher lateral pressure on the excavation walls, increase concrete density, and decrease porosity/permeability. However, velocity-induced impacts on reinforcing steel may mis-align tied steel stirrups and the air content (if specified) of the concrete can be reduced.

The concrete mix design for drilled shafts should produce a sufficient slump (typically between 6 and 9 inches) to ensure that lateral fluid concrete pressure will develop against the excavation walls. Further, the concrete should maintain a slump no less than 4 inches (slump loss limit) for several hours. This typically allows enough time to remove the tremie and any temporary casing while the concrete is still fluid enough to replace the volume of the tremie or casing and minimize suction forces (net negative lateral pressure) during extraction. However, recent studies suggest that a final slump in the range of 3.5 to 4 inches (or less) at the time of temporary casing extraction can drastically reduce the side shear capacity of the shaft (Garbin, 2003). As drilled shaft concrete is not vibrated during placement, the maximum aggregate size should be small enough to permit unrestricted flow through the steel reinforcing cage. The ratio of minimum rebar spacing to maximum aggregate diameter should be no less than 3 to 5 (FHWA, 1999).

Design Capacity of Drilled Shafts

The capacity of drilled shafts is developed from a combination of side shear and end bearing. The side shear is related to the shear strength of the soil and in sands can be thought of as the lesser of the friction ($F_r = \mu N$) that develops between the shaft concrete and the surrounding soil or the internal friction within the surrounding soil itself. Although a coefficient of friction (μ) can be reasonably approximated, the determination of the normal force (N) is more difficult due to lateral stress relaxation during excavation. In clayey soils or rock side shear is most closely related to the unconfined compressive strength, q_u. The end bearing is analogous to shallow foundation bearing capacity with a very large depth of footing. However, it too is affected by construction induced disturbances and like the side shear has been empirically incorporated into the design methods discussed in the ensuing sections.

The design approach for drilled shafts can be either allowable stress design (ASD) or load and resistance factor design (LRFD) as dictated by the client, local municipality, or State agency. In either case, the concept of usable capacity as a function of ultimate capacity must be addressed. This requires the designer to have some understanding of the capacity versus displacement characteristics of the shaft. Likewise, a permissible displacement limit must be established to determine the usable capacity rather than the ultimate capacity which may be unattainable within a reasonable displacement. The permissible displacement (or differential displacement) is typically set by a structural engineer on the basis of the proposed structure's sensitivity to such movement. To this end, design of drilled shafts (as well as other foundation types) must superimpose displacement criteria onto load carrying capability even when using a LRFD approach. This is divergent from other non-geotechnical LRFD approaches that incorporate design limit states independently (discussed later).

The designer must be aware of the difference in the required displacements to develop significant capacity from side shear and end bearing. For instance, in sand the side shear component can develop 50% of ultimate capacity at a displacement of approximately 0.2% of the shaft diameter (D) (AASHTO, 1997), and develops fully in the range of 0.5 to 1.0 % D (Bruce, 1986). In contrast, the end bearing component requires a displacement of 2.0% D to develop 50% of its capacity (AASHTO, 1997), and fully develops in the range of 10 to 15% D (Bruce, 1986). Therefore, a 4 ft diameter shaft in sand can require up to 0.5 inches displacement to develop ultimate side shear and 7.2 inches to develop ultimate end bearing. Other sources designate the displacement for ultimate end bearing to be 5% D but recognize the increase in capacity at larger displacements (Reese and Wright, 1977; Reese and O'Neill, 1988).

In most instances, the side shear can be assumed to be 100% usable within most permissible displacement criteria but the end bearing may not. This gives rise to the concept of mobilized capacity. The mobilized end bearing is the capacity that can be developed at a given displacement. Upon determining the permissible displacement, a proportional capacity can then be established based on a capacity versus displacement relationship as determined by either load testing or past experience. A general relationship will be discussed in the section discussing end bearing determination methods.

ASD vs. LRFD. In geotechnical designs, both ASD and LRFD methods must determine an ultimate capacity from which a usable capacity is then extracted based on displacement criteria. As such the ultimate capacity is never used, but rather a displacement-restricted usable capacity is established as the *effective ultimate capacity*. For drilled shafts, this capacity typically incorporates 100% of ultimate side shear and the fraction of end bearing mobilized at that displacement. Once this value has been determined, the following generalized equations represent the equality that must be satisfied when using either an **A**llowable **S**tress **D**esign or a **L**oad and **R**esistance **F**actor **D**esign approach, respectively.

$$Service\ Load \leq \frac{Effective Ultimate Strength}{S.F.} N \qquad (ASD)$$

or

$$P_u = \Sigma \gamma_i P_i \leq \phi P_n N \qquad (LRFD)$$

where, P_u represents the sum of factored or inflated service loads based on the type of loads, P_n represents the effective ultimate shaft capacity, N is the number of shafts, and ϕ (the resistance factor) reduces the effective ultimate capacity based on the reliability of the capacity determination method. The use of LRFD in geotechnical designs is relatively new and as such present methods have not yet completely separated the various limit states.

Typically there are four LRFD limit states: strength, service, fatigue, and extreme event. These limit states treat each area as mutually exclusive issues. Strength limit states determine if there is sufficient capacity for a wide range of loading conditions. Service limit states address displacement and concrete crack control. Fatigue addresses the usable life span of steel in cyclic or stress reversal regions. Extreme event limit states introduce less probable but more catastrophic occurrences such as earthquakes or large vessel impacts. Any of the four limit states can control the final design. The ASD method lumps all load types into a single service load and assumes the same probability for all occurrences.

Although LRFD strength limit states should be evaluated without regard to the amount of displacement required to develop full ultimate capacity (P_n), present LRFD methods establish geotechnical ultimate capacity based on some displacement criteria. As a result, LRFD geotechnical service limits states are relatively unused. To this end, this chapter will emphasize the design methods used to determine ultimate capacity and will denote (where applicable) the displacement required to develop that capacity. The following design methods are either the most up to date or the most widely accepted for the respective soil type and/or soil exploration data.

SPT Data in Sand

Standard penetration test results are most commonly used for estimating a drilled shaft capacity in sandy soils. For some design methods direct capacity correlations to the SPT blow count (N) have been developed; in other cases correlations to soil properties such as unit weight or internal angle of friction are necessary. Where the unit weight or the internal friction angle (sands) of a soil is required the relationships shown in Figure 1 can be used.

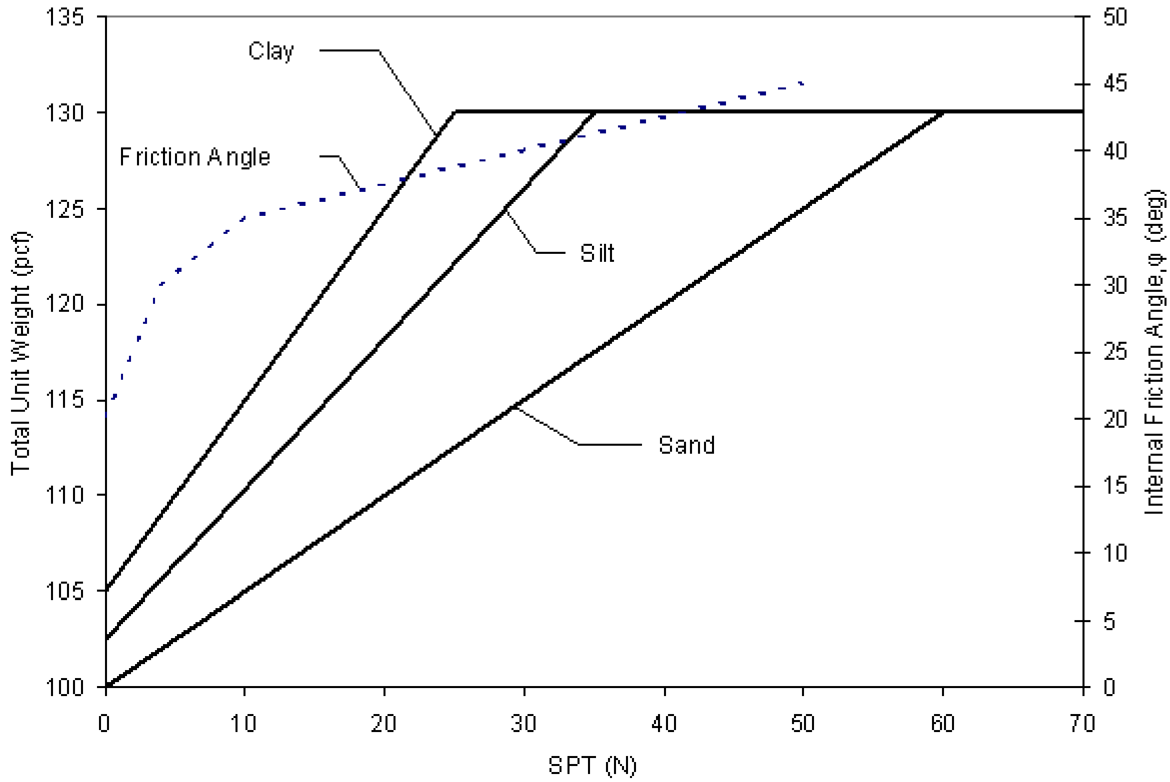


Figure 1 Estimated soil properties from SPT blow count.

Side Shear. The side shear developed between a shaft and surrounding sandy soils can be estimated using the following methods in Table 1. The ultimate load carrying capacity from side shear (Q_s) can be expressed as the summation of side shear developed in layers of soil to a given depth containing *n* layers:

$$Q_s = \pi \sum_{i=1}^{n} f_{si} \, L_i \, D_i$$

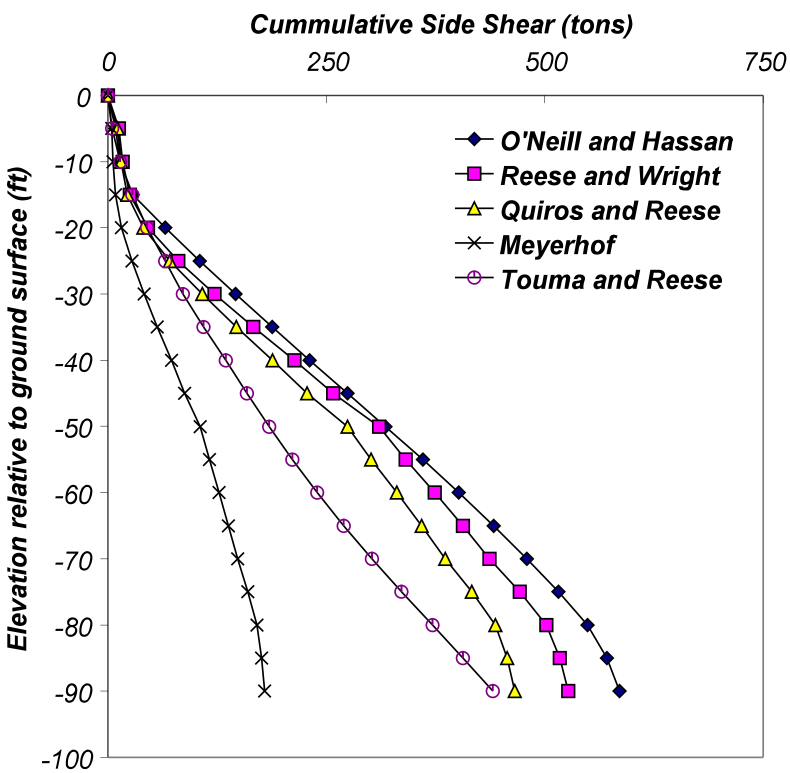
where f_{si} is the estimated unit side shear for the i^{th} soil layer
L_i is the thickness of (or length of shaft in) the i^{th} soil layer
D_i is the diameter of the shaft in the i^{th} soil layer

Table 1. Drilled Shaft Side Shear Design Methods for Sand (adapted from AASHTO, 1998)

Source	Side Shear Resistance, f_s (in tsf)
Touma and Reese (1974)	$f_s = K\sigma_v' \tan\phi < 2.5$ tsf where $K = 0.7$ for $D_b \leq 25$ ft $K = 0.6$ for 25 ft $< D_b \leq 40$ ft $K = 0.5$ for $D_b > 40$ ft
Meyerhof (1976)	$f_s = N / 100$

Quiros and Reese (1977)	$f_s = 0.026\ N < 2.0$ tsf
Reese and Wright (1977)	$f_s = N / 34$, for $N \le 53$ $f_s = (N - 53) / 450 + 1.6$, for $53 < N \le 100$ $f_s \le 1.7$
Reese and O'Neill (1988) *Beta Method*	$f_s = \beta\sigma_v' < 2.0$ tsf, for $0.25 \le \beta \le 1.2$ where $\beta = 1.5 - 0.135\ z^{0.5}$, z in ft
O'Neill and Hassan (1994) *Modified Beta Method*	$f_s = \beta\sigma_v' < 2.0$ tsf, for $0.25 \le \beta \le 1.2$ where $\beta = 1.5 - 0.135\ z^{0.5}$ for $N > 15$ $\beta = N/15\ (1.5 - 0.135\ z^{0.5})$ for $N \le 15$

Using the above methods, the variation in estimated side shear capacity is illustrated for a 3 ft diameter shaft and the given SPT boring log in sandy soil in Figure 2. Although any of these methods may correlate closely to a given site or local experience, the author recommends the O'Neill and Hassan approach in spite of its less conservative appearance.



Boring B-1; GWT -20ft

Elev. (ft)	SPT (N)
0.00	18
-5.00	9
-10.00	3
-15.00	6
-20.00	15
-25.00	25
-30.00	30
-35.00	32
-40.00	34
-45.00	32
-50.00	38
-55.00	22
-60.00	24
-65.00	23
-70.00	22
-75.00	25
-80.00	22
-85.00	11
-90.00	7

Figure 2 Comparison of estimated side shear capacities in sandy soil (3 ft diam).

End Bearing. Recalling the importance of the mobilized end bearing capacity concept, a parameter termed the tip capacity multiplier (TCM) will be used to quantify the relationship between ultimate and usable end bearing capacity. Four design methods using two different approaches to mobilized capacity are discussed. The first and second assume ultimate end bearing occurs at 1.0 inch displacement (Touma and Reese, 1974; Meyerhoff, 1976). The others assume ultimate end bearing occurs at a 5% displacement as shown in Figure 3 (Reese and Wright, 1977; Reese and O'Neill, 1988). This figure shows the latter relationship in terms of the permissible displacement expressed as a percentage of the shaft diameter. Therein, the TCM for convention shafts tipped in sand is linearly proportional to the displacement where the TCM = 1 at 5% displacement. This concept can be extended to the first two design methods as well where TCM = 1 at 1.0 inches displacement. Table 2 lists the four methods used to estimate the ultimate end bearing to which a TCM should be applied.

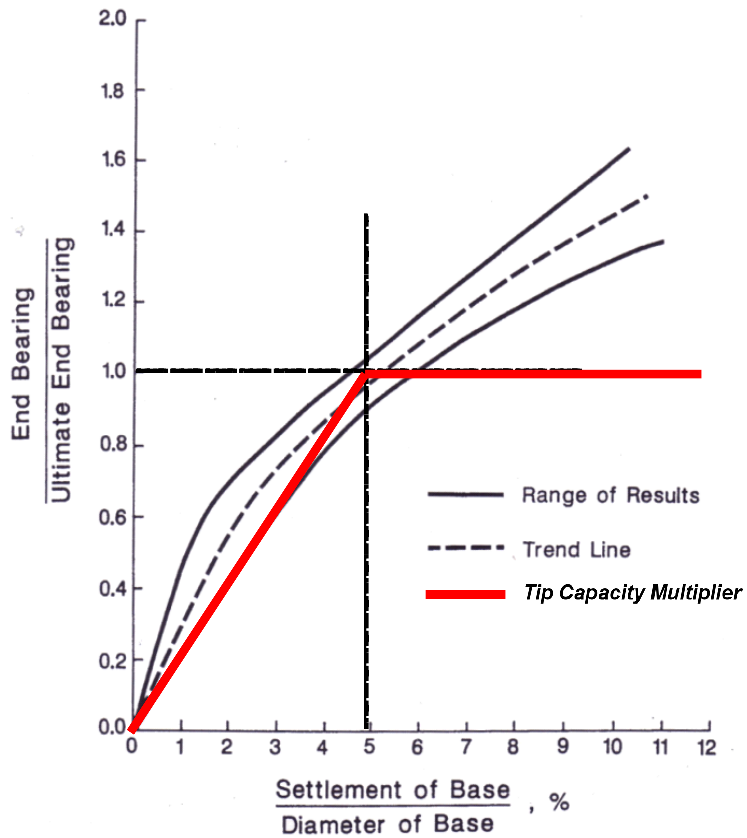


Figure 3 End bearing response of sands as a function of displacement (based on Reese and O'Neill, 1988).

Figure 4 shows the calculated ultimate end bearing using each of the four methods in Table 2. The Reese and Wright or Reese and O'Neill methods are recommended by the author for end bearing analysis. Using the combined capacity from 100% side shear and TCM*q_p using O'Neill and Hassan and Reese and O'Neill methods, respectively, the effective ultimate capacity of a 3 ft diameter drilled shaft can be estimated as a function of depth, Figure 5. This type of curve is convenient for design as it is a general capacity curve independent of a specific design load. However, when using a LRFD approach, the factored load(P_u) should be divided by the appropriate resistance factor before going to this curve.

Table 2. Drilled Shaft End Bearing Design Methods for Sands (AASHTO, 1998)

Source	End Bearing Resistance, q_p (in tsf)**
Touma and Reese (1974)	Loose Sand, $q_p = 0.0$ Medium Dense Sand, $q_p = 16 / k$ Very Dense Sand, $q_p = 40 / k$ where $k = 1$ for $D_p = 1.67$ ft $k = 0.6\ D_p$ for $D_p \geq 1.67$ ft only for shaft depths > 10 D
Meyerhof (1976)	$q_p = (2N_{corr}D_{b)} / (15\ D_p)$ $q_p < 4/3\ N_{corr}$ for sand $q_p < N_{corr}$ for non-plastic silts
Reese and Wright (1977)	$q_p = 2/3$ N for $N \leq 60$ $q_p = 40$ for $N > 60$
Reese and O'Neill (1988)	$q_p = 0.6$ N for $N \leq 75$ $q_p = 45$ for $N > 75$

** For D > 4.17 ft, the end bearing resistance should be reduced to $q_{pr} = 4.17q_p / D$.

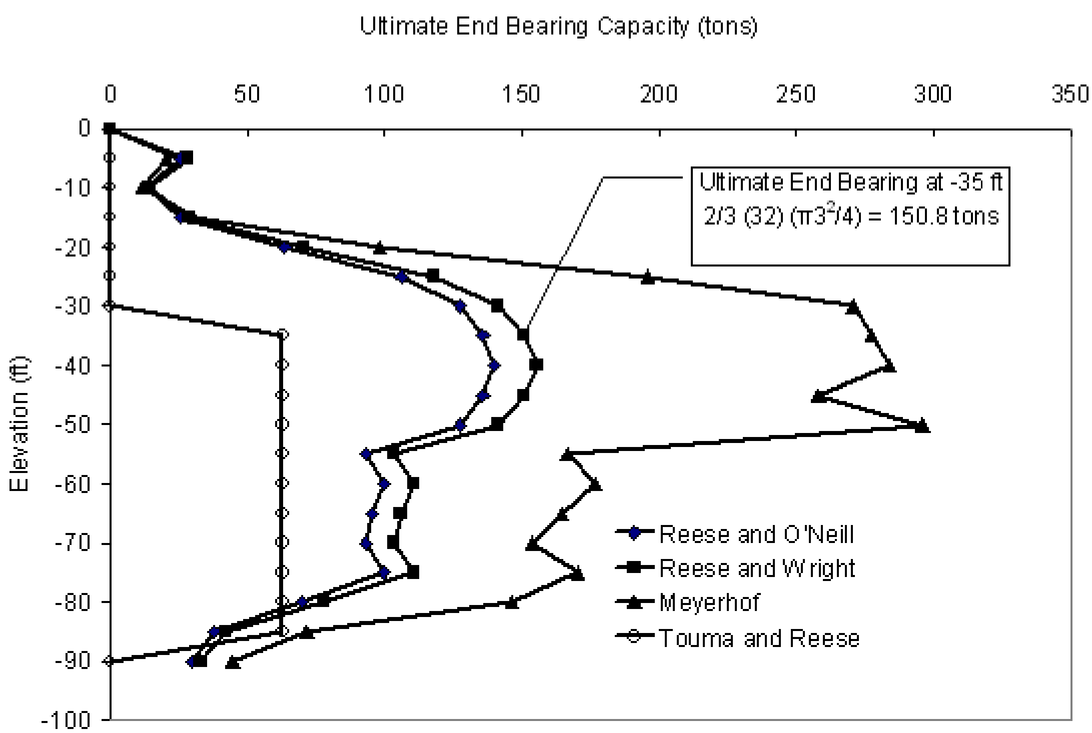


Figure 4 Comparison of end bearing methods in sand (3 ft diam, Boring B-1).

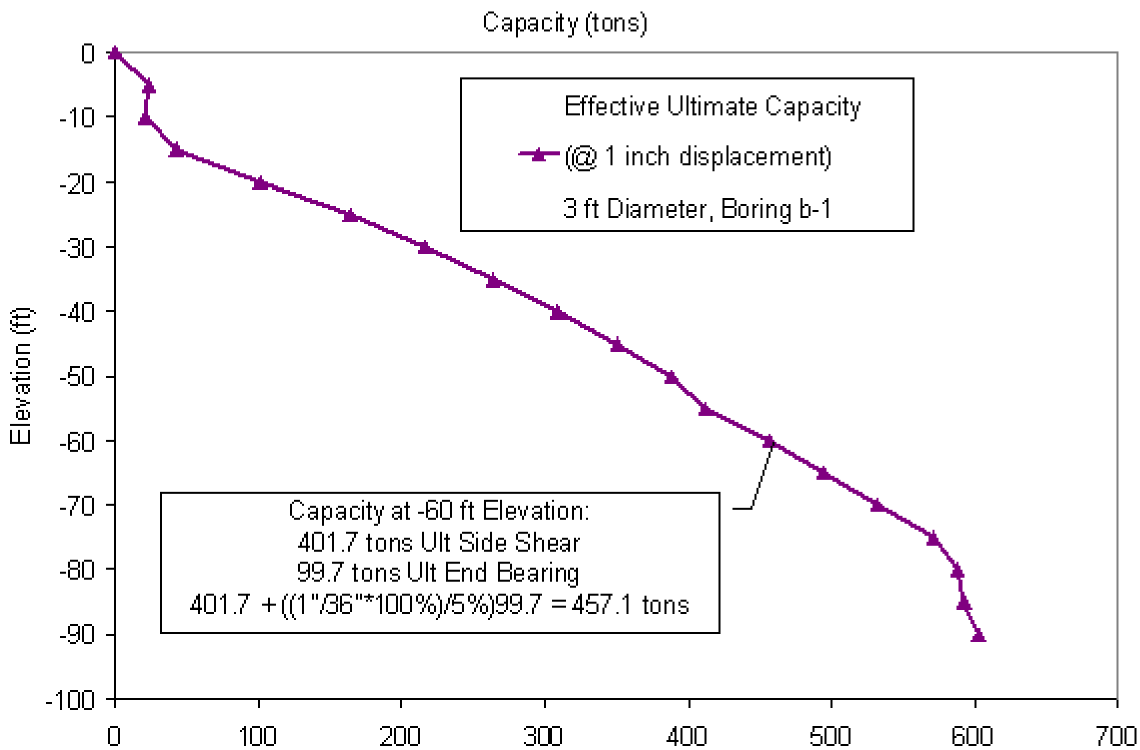


Figure 5 Example design curve using Boring B-1 from Figure 1.

Triaxial or SPT Data in Clay

Unconsolidated, undrained (UU) triaxial test results are preferred when estimating the side shear or end bearing capacity of drilled shafts in clayey soil. The mean undrained shear strength (S_u) is derived from a number of tests conducted on Shelby tube specimens where $S_u = 1/2\ \sigma_{1\ Max}$. In many instances, both UU and SPT data can be obtained from which local SPT(N) correlations with S_u can be established. In the absence of any UU test results, a general correlation from Kulhawy and Mayne (1990) can be used

$$S_u = 0.0625\ N, \text{ in units of tsf}$$

Side Shear *(alpha method)*. The alpha method of side shear estimation is based on correlations between measured side shear from full-scale load tests and the clay shear strength as determined by UU test results. Therein, the unit side shear f_s is directly proportional to the product of the adhesion factor (α) and S_u.

$$f_s = \alpha\ S_u$$

Table 3. Adhesion factor for drilled shafts in clayey soils.

Adhesion Factor, α (dimensionless)	Undrained Shear Strength, Su (tsf)
0.55	< 2.0
0.49	2.0 - 3.0
0.42	3.0 - 4.0
0.38	4.0 - 5.0
0.35	5.0 - 6.0
0.33	6.0 - 7.0
0.32	7.0 - 8.0
0.31	8.0 - 9.0
Treat as Rock	> 9.0

The side shear developed around drilled shafts in clayey soil has several limitations that were not applied previously applied to shafts cast in sand. Specifically, the top 5 feet of the shaft sides are considered non contributing due to cyclic lateral movements that separate the shaft from the soil as well as potential dessication separation of the surficial soil. Additionally, the bottom 1D of the shaft side shear is disregarded to account for lateral stresses that develop radially as the end bearing mobilizes.

Although rarely used today, belled ends also affect the side shear near the shaft base. In such cases, the side shear surface area of the bell as well as that area 1D above the bell should not be expected to contribute capacity.

End Bearing. The end bearing capacity of shafts tipped in clay is also dependent on the mean undrained shear strength of the clay within two diameters below the tip, S_u. As discussed with shafts tipped in sands, a TCM should be applied to estimated end bearing capacities using the relationship shown in Figure 6. At displacements of 2.5% of the shaft diameter, shafts in clay mobilize 75 to 95% of ultimate capacity. Unlike sands, however, there is little reserve bearing capacity beyond this displacement. Therefore, a maximum TCM of 0.9 is recommended for conventional shafts at displacements of 2.5%D and proportionally less for smaller permissible displacements.

Similar to shallow foundation analyses, the following expressions may be used to estimate the ultimate end bearing for shafts with diameters less than 75 inches (AASHTO, 1998):

$$q_p = N_c \, S_u \leq 40 \text{ tsf}$$

where

$$N_c = 6\,[\,1 + 0.2(Z/D)] \leq 9 \text{ for } S_u > 0.25 \text{ tsf}$$
$$N_c = 4\,[\,1 + 0.2(Z/D)] \leq 9 \text{ for } S_u < 0.25 \text{ tsf}$$

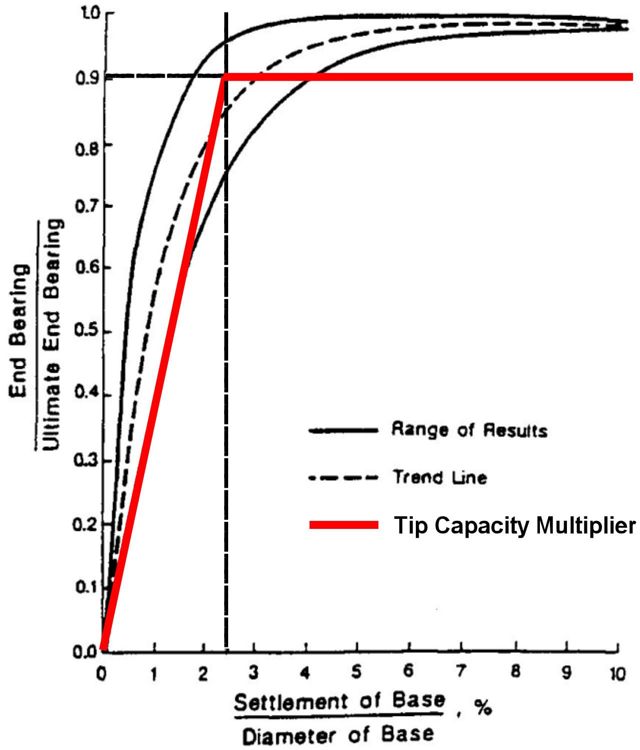


Figure 6 End bearing response of shafts tipped in clays (Reese and O'Neill, 1988).

and Z/D is the ratio of the shaft diameter to depth of penetration. For shafts greater than 75 inches in diameter a reduction factor should be used as follows:

$$q_{pr} = q_p F_r$$

$$F_r = \frac{2.5}{12aD_p + 2.5b} \leq 1.0$$

where:

and

$$a = 0.0071 + 0.0021\ Z/P \leq 0.015$$

$$b = 0.45\ (2\ S_u)^{0.5}$$

for

$$0.5 \leq b \leq 1.5$$

Designing Drilled Shafts from CPT Data

Cone penetration test data is considered to be more reproducible than SPT data and can be used for shaft designs in cohesionless and cohesive soils using correlations developed by Alsamman (1995). Although that study provided design values for both mechanical and electric cone data, a single approach is presented below that can conservatively be used for either based on that work.

Side Shear. This method for determining side shear resistance in cohesionless soils is divided into two soil categories: gravelly sand/gravel or sand/silty sand. In each case below in Table 4, the side shear is correlated to the cone tip resistance, q_c, instead of the sleeve friction due to the absence of that data from some case studies at the time of the study. In cohesive soils, a single expression is given which is also dependent on the total vertical stress, σ_{vo}. The same regions of the shaft should be discounted (top 5 ft and bottom 1D) when in cohesive soils as discussed earlier.

Table 4. Side Shear Resistance from CPT data

Soil Type	Ultimate Side Shear Resistance, q_s (tsf)	
Gravelly Sand / Gravel	$f_s = 0.02\ q_c$ $f_s = 0.0019\ q_c + 0.9 \leq 1.4$	for $q_c \leq 50$ tsf for $q_c > 50$ tsf
Sand / Silty Sand	$f_s = 0.015\ q_c$ $f_s = 0.0012\ q_c + 0.7 \leq 1.0$	for $q_c \leq 50$ tsf for $q_c > 50$ tsf
Clay	$f_s = 0.023\ (q_c - \sigma_{vo}) \leq 0.9$	

The upper limits for side shear recommended by Alsamman are somewhat less than those cited from AASHTO (e.g. 2.0 tsf for sands using the *Beta Method*). However, CPT data can also be used to estimate the internal friction and soil density necessary for the Touma and Reese or Beta methods.

End Bearing. Expressions for estimating the end bearing using CPT data were also recommended by the same study (Alsamman, 1995). Therein, the end bearing categories were limited to cohesionless and cohesive soils. Table 5 provides correlations based on those findings.

Table 5. End Bearing Resistance from CPT data

Soil Type	Ultimate End Bearing Resistance, q_p (tsf)	
Cohesionless Soils	$q_p = 0.15\ q_c$ $q_p = 0.05\ q_c + 10 \leq 30$	for $q_c \leq 100$ tsf for $q_c > 100$ tsf
Cohesive Soils	$q_p = 0.25\ (q_c - \sigma_{vo}) \leq 25$	

The capacities estimated from Table 5 expressions are ultimate values that should be assigned a proportionally less usable capacity using the general relationships shown in Figures X and Y for sands and clays, respectively.

Designing from Rock Core Data

A common application for drilled shaft is to be socketed in a rock formation some distance, H_s. In these cases, the side shear of softer overlying materials is disregarded due to the mismatch in the displacement required to mobilize both material types. Rock sockets require relatively small movements to develop full capacity when compared to sand or clay strata. Further, although the end bearing strength of a rock socket can be quite considerable, it too is often discounted for the same reason. Alternately, a rock socket may be designed for all end bearing instead of side shear knowing that some side shear capacity will always be available in reserve.

Side Shear. The side shear strength of rock-socketed drilled shafts is similar to that of clayey soils in that it is dependent on the insitu shear strength of the bearing strata. In this case rock cores are taken from the field and tested in various methods. Specifically, mean failure stress from two tests are commonly used: the unconfined compression test, q_u; and the splitting tensile test, q_s. The test results from these tests can be used to estimate the side shear of a rock socket using the expressions in Table 6. The estimated side shear capacity can be reduced by multiply q_s by either the rock quality index, RQD, or the percent sample recovered from the rock core. Local experience and results from load tests can provide the best insight into the most appropriate approach.

Table 6. Drilled Shaft Side Shear Design Methods for Rock Sockets

Source	Side Shear Resistance, f_s (tsf)
Carter and Kulhawy (1988)	$f_s = 0.15\ q_u$ for $q_u \le 20$ tsf
Horvath and Kenney (1979)	$f_s = 0.67\ q_u^{0.5}$ for $q_u > 20$ tsf
McVay and Townsend (1990)	$f_s = 0.5\ q_u^{0.5}\ q_s^{0.5}$

End Bearing. When determining the end bearing resistance (as well as side shear) of drilled shafts in rock, the quality of rock and type of rock can greatly affect the capacity. In competent rock the structural capacity of the concrete will control the design. In fractured, weathered rock or limestone, the quality of the formation as denoted by the RQD or %recovery should be incorporated into the capacity estimate. However, these parameters are influenced by drilling equipment, driller experience and the type of core barrel used to retrieve the samples. The designer should make some attempt to correlate the rock quality to load test data where possible. The Federal Highway Administration recommends the following expression for estimating the end bearing resistance in rock (FHWA, 1988):

$$q_b = 2.5\ q_u\ \%\text{Rec} \le 40\ \text{tsf}$$

The value of 40 tsf is undoubtedly conservative with respect to ultimate capacity, but when used in conjunction with a rock socket side shear it may be reasonable. Under any circumstances, load testing can verify much higher capacities even though they are near impossible to fail in competent rock.

Designing from Load Test Data

The use of an instrumented load test data for design is thought to be the most reliable approach and is given the highest resistance factor (LRFD) or lowest safety factor (ASD) as a result. This method involves estimating the shaft capacity using one of the previously discussed method (or similar) and verifying the estimated capacity using a full-scale prototype shaft loaded to ultimate capacity. These tests can be conducted prior to construction or during construction (denoted as design phase or construction phase load testing, respectively). In either event, the shaft should be loaded well in excess of the design load while monitoring the response (i.e. axial displacement, lateral displacement and/or internal strains).

An instrumented load test is one that incorporates strain gages along the length of the foundation to delineate load carrying contributions from various soil strata. The test can merely distinguish side shear from end bearing or additional information from discrete shaft segments / soil strata can also be obtained. Any test method capable of applying the ultimate load can provide useful feedback to the designer. Tests conducted to lesser loads are still useful but provide only a "proof test" to the magnitude of the maximum load and can only provide a lower bound of the actual capacity. As such, the designer should realize that a test shaft that fails geotechnically, thus providing the ultimate capacity, is desirable in such a program so that the upper limit of capacity can be realized. The challenge then is to design a shaft that fails at a load reasonably close to the desired ultimate without being too conservative. However, the loading apparatus should have sufficient reserve to account for a slightly conservative capacity estimate.

Side Shear. The ultimate side shear can be determine from load testing by evaluating the response from embedded strain gages at various elevations in the shaft. It is desirable to delineate bearing strata by placing these gages at the interface between significantly different soil strata (e.g. clay / sand interface). At a minimum, one level of gages should be placed at the tip of the shaft to separate the load carrying contributions from the side shear and end bearing. By monitoring the strain at a given level, the corresponding load and difference in load between levels can be determined. It is further desirable to use four gages per level to help indicate eccentricities in the loading as well as provide redundancy.

The load at a particular level can be evaluated using strain gage data using the following expression:

$$P_i = \varepsilon_i E_i A_i$$

where

- P_i is the load at the i^{th} level
- ε_i is the strain measured at the i^{th} level
- E_i is the composite modulus of the i^{th} level, and
- A_i is the cross sectional area of the i^{th} level.

The side shear from a given shaft segment can then be calculated from the difference in measured load from the two levels bounding that segment.

$$f_s = (P_i - P_{i+1}) / (L \pi D)$$

where L and D are the length and diameter of the shaft segment, respectively. If only using a single gage level at the toe of the shaft, P_i is the applied load to the top of the shaft and P_{i+1} is the load calculated from strain at the toe.

End Bearing. The end bearing can be similarly determined from strain data. However, the ultimate end bearing is not necessarily established. Rather, the effective ultimate capacity (usable capacity) is determined on the basis of permissible displacement. Although several approaches do exist that attempt to extract a single capacity value from test data, the entire load versus displacement response should be noted. Figure 7 shows the end bearing response as measured from a load test. A comparison between the measured and predicted values should be prepared so that the original design approach can be calibrated. The end bearing strength is determined from strain gage data using the following expression:

$$q_b = P_{toe} / A = \varepsilon_{toe} E_{toe}$$

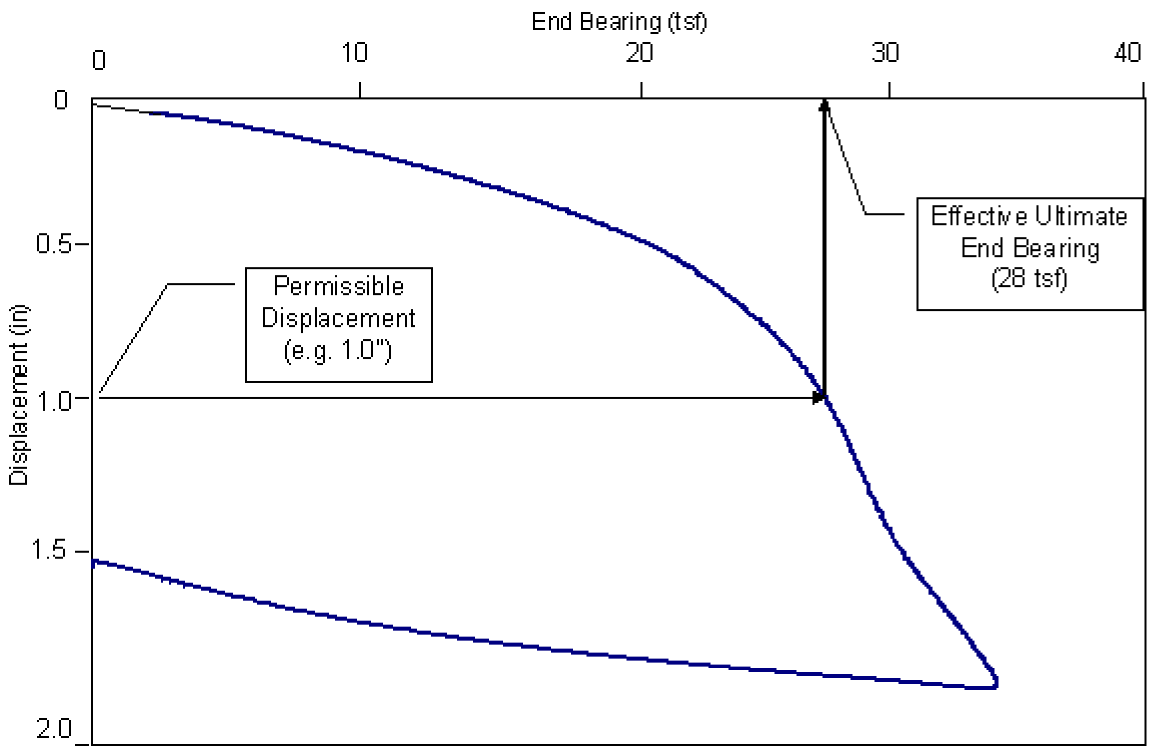


Figure 7 End Bearing Load Test Results.

Designing Post Grouted Shafts

The end bearing component of drilled shafts is only fractionally utilized in virtually all design methods (TCM < 1.0) due to the large displacement required to mobilize ultimate capacity. Consequently, a large portion of the ultimate capacity necessarily goes unused. In an effort to regain some of this unusable capacity, mechanistic procedures to integrate its contribution have been developed using pressure grouting beneath the shaft tip (also called post grouting or base grouting). Pressure grouting the tips of drilled shafts has been successfully used worldwide to precompress soft debris or loose soil relaxed by excavation (Bolognesi and Moretto, 1973; Stoker, 1983; Bruce, 1986; Fleming, 1993; Mullins et al., 2000; Dapp and Mullins, 2002). The post-grouting process entails: (1) installation of a grout distribution system during conventional cage preparation that provides grout tube-access to the bottom of the shaft reinforcement cage, and (2) after the shaft concrete has cured, injection of high pressure grout beneath the tip of the shaft which both densifies the in-situ soil and compresses any debris left by the drilling process. By essentially preloading the soil beneath the tip, higher end bearing capacities can be realized within the service displacement limits.

Although post grouting along the sides of the shaft has been reported to be effective, this section will only address the design of post grouted shaft tips. The overall capacity of the shaft is still derived from both side shear and end bearing where the available side shear is calculated using one or a combination of the methods discussed earlier. Further, the calculation of the available side shear is an important step in determining the pressure to which the grout can be pumped.

Post Grouting in Sand. The design approach for post grouted drilled shaft tips makes use of common parameters used for a conventional (un-grouted) drilled shaft design. This methodology includes the following seven steps:

(1) Determine the ungrouted end bearing capacity in units of stress.

(2) Determine the permissible displacement as a percentage of shaft diameter (e.g. 1"/48"*100% ≈ 2%).

(3) Evaluate the ultimate side shear resistance for the desired shaft length and diameter (in units of force).

(4) Establish a maximum grout pressure that can be resisted by the side shear (ultimate side shear divided by the tip cross sectional area).

(5) Calculate the Grout Pressure Index, GPI, defined as the ratio of grout pressure to the ungrouted end bearing capacity (*Step 3 / Step 1*).

(6) Using design curves from Figure 8, determine the Tip Capacity Multiplier, TCM, using the GPI calculated in *Step 5*.

(7) Calculate the grouted end bearing capacity (effective ultimate) by multiplying the TCM by the ungrouted end bearing (TCM * *Step 1*).

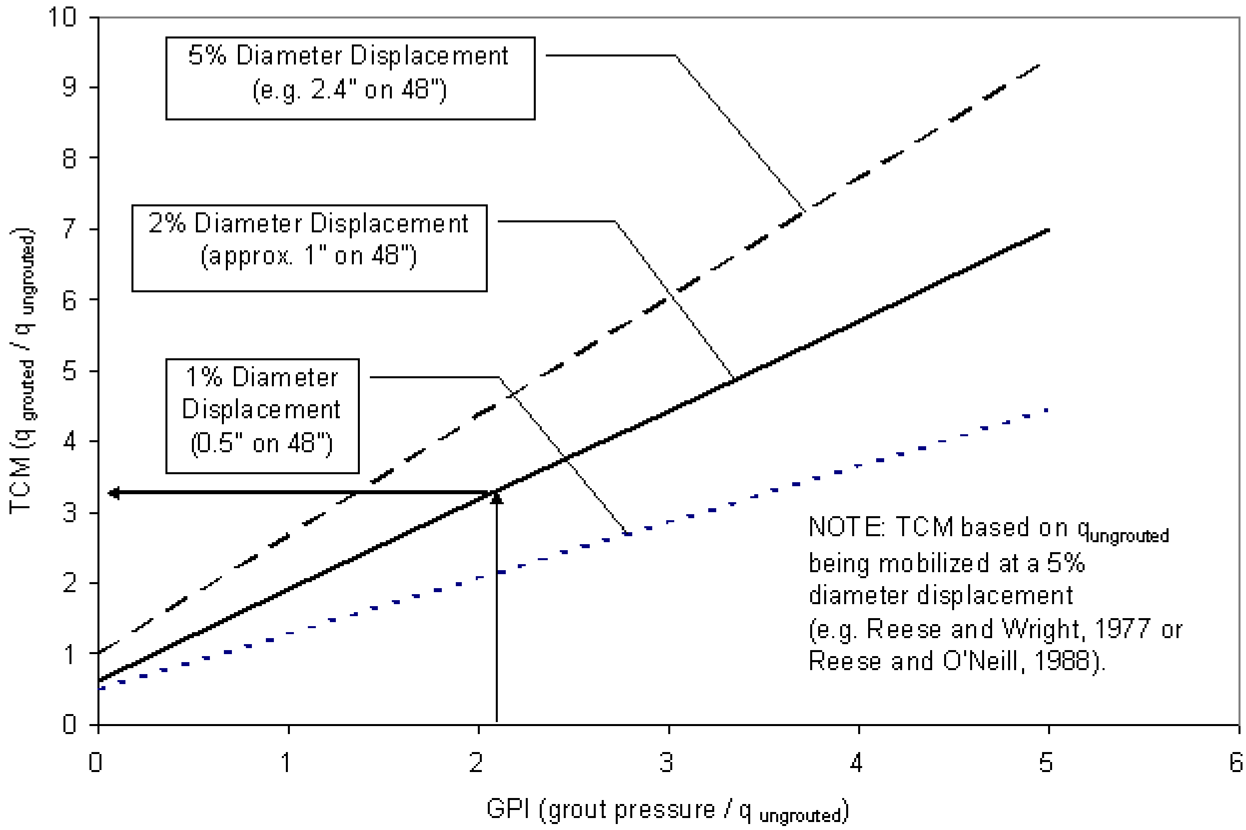


Figure 8 Correlations used in *Step 6* to establish TCM (Mullins, et al., 2001).

The ungrouted capacity (GPI = 0) is represented by these curves at the y-intercept where TCM = 1 for a 5% displacement (no improvement). The 1% and 2% intercepts reduce the end bearing according to the normal behavior of partially mobilized end bearing. Interestingly, the grouted end bearing capacity is strongly dependent on available side shear capacity (grout pressure) as well as the permissible displacement. However, it is relatively independent of the ungrouted end bearing capacity when in sandy soils. As such, the end bearing in loose sand deposits can be greatly improved in both stiffness and ultimate capacity given sufficient side shear against which to develop grout pressure. In dense sands and clays significant improvement in stiffness can be realized with more modest effects on ultimate capacity. Figure 9 shows the effective ultimate capacity that can be expected from a grouted shaft similar to that from Example 1.

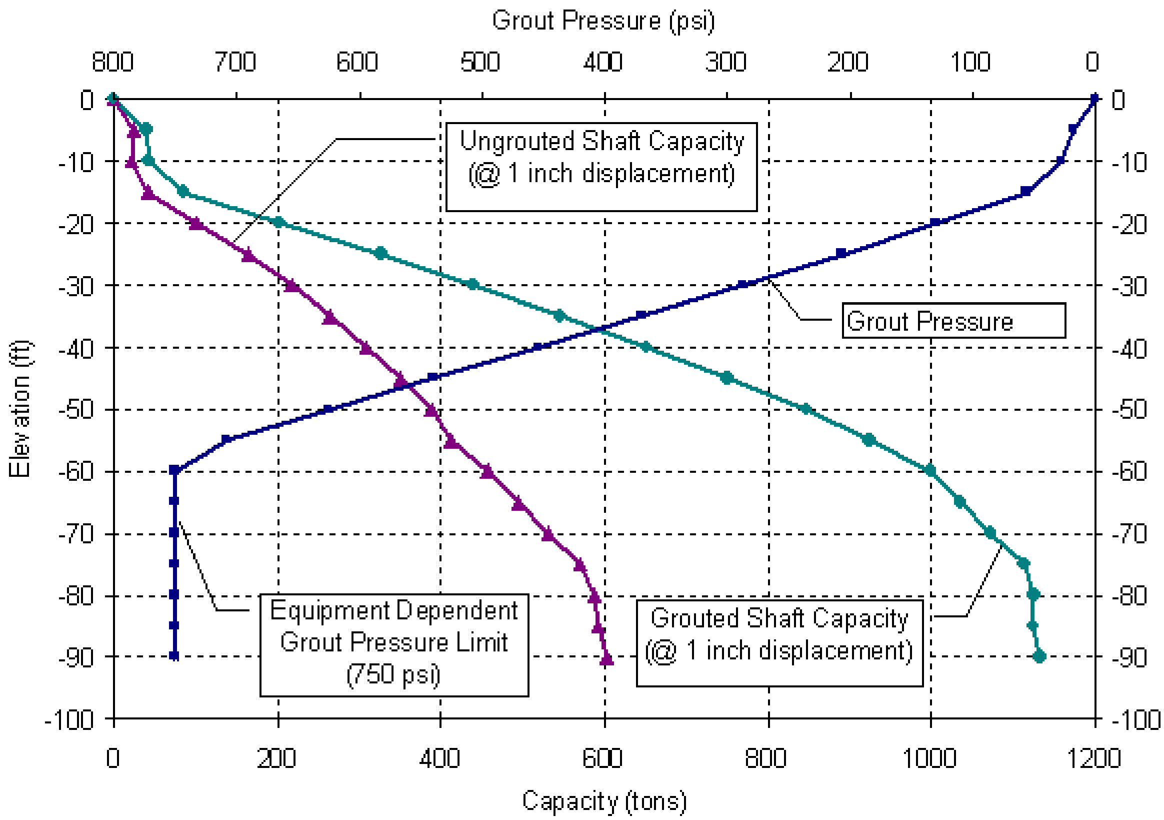


Figure 9 Post grouted shaft capacity extended from Example 1.

Post Grouting in Other Formations. Post grouting shaft tips in other formations such as clays, silts, and rock can be advantageous for the same reasons as in sand. However, the degree of improvement may be more modest. In clays and plastic silts, the TCM can be assumed to be 1.0 although studies have shown it to be as high as 1.5 if sufficient side shear can be developed (Mullins and O'Neill, 2003). In non plastic silts, the TCM can be assumed to be 1.0 for initial designs but a verification load test program is recommend as much higher values may be reasonable. In rock, post grouted shafts have the potential to engage both the side shear and end bearing simultaneously. In all soil types the achieved grout pressure can be used as a lower bound for usable end bearing and the attainable grout pressure is always dependent on the available side shear against which to react. In contrast, sufficient side shear capacity does not assure that grout pressure can be developed without excessive volumes of grout.

Post grouting shaft tips provides a capacity verification for every shaft grouted. To optimize its use and design, a full load test program should be scheduled at the onset to confirm the TCM most appropriate for a given site and soil type.

Economy of Load Testing

Although the cost of foundations is most closely linked to the presence of an adequate bearing strata and the applied load, it is also directly affected by the design approach and the diameter of the shaft selected. As such, a designer may employ a range of safety factors (or resistance factors) given the level of confidence that can be assigned to a particular scenario. The most common method of establishing a particular level of certainty is via some form of testing. This testing can range from applying the full anticipated load (static or statnamic tests) to a minimum of a subsurface investigation to estimate insitu soil properties. Load tests result in the highest increase in designer confidence and can be incorporated into the design in the form of adjusted/calibrated unit strengths, reduced safety factors, or increased resistance factors. The effects of design uncertainty can be illustrated by the AASHTO (1998) specifications for driven piles where the designer must select from nine different resistance factors ranging from 0.35 to 0.80 based on the design methodology. Four of these conditions are selected based on the level/quality of testing that is anticipated. Therein, the highest resistance factor (0.8) and confidence is associated with a load test. The next highest (0.65) is assigned to test methods related to installation monitoring. In contrast, the lowest confidence and resistance factor (0.35 - 0.45) is assigned when a design is based solely on capacity correlations with SPT data. Although some resistance factors for drilled shafts are not given by AASHTO, the resistance factors most commonly range from 0.5 to 0.8 for no testing to load testing, respectively.

The following two examples will use estimated costs to illustrate the impact of shaft size (diameter) and design approach (ϕ factor) on cost effectiveness. The cost of shaft construction and testing can vary significantly based on the number of shafts and type of material excavated as well as the physical conditions and location of the site. Even though a typical unit price of a drilled shaft includes each of these parameters, this approach can be used for comparisons using updated site-specific values.

Given:

3 ft diameter shaft	\$100 / lineal foot	excavation and concreting
4 ft diameter shaft	\$200 / lineal foot	excavation and concreting
6 ft diameter shaft	\$400 / lineal foot	excavation and concreting
Static load test	\$125 / ton of test	1% of shafts tested (1 min.)
Statnamic load test	\$35 / ton of test	1% of shafts tested (1 min.)

Use: Boring log and effective ultimate capacity calculations from Example 1, as well as the following resistance values (slightly update from most recent AASHTO)

Static load test	$\phi = 0.75$
Statnamic load test	$\phi = 0.73$
No testing (SPT only)	$\phi = 0.55$

Assume a maximum excavation depth of 30D

Selecting the Most Economical Shaft Diameter. Many options are available to the designer when selecting the diameter of shaft to be used for a specific foundation. For instance, a long, small diameter shaft can provide equivalent axial capacity to a shorter, larger diameter shaft. Figure 10 shows the result of re-evaluating Example 1 for 3, 4, and 6 ft diameter shafts while incorporating the cost per ton of capacity using $100, $200, and $400 per ft of shaft, respectively. These curves are based on axial capacity and the cost may further vary given significant lateral loading and the associated bending moment requirements. In this case, the 3 ft diameter shaft is the most cost effective at all depths.

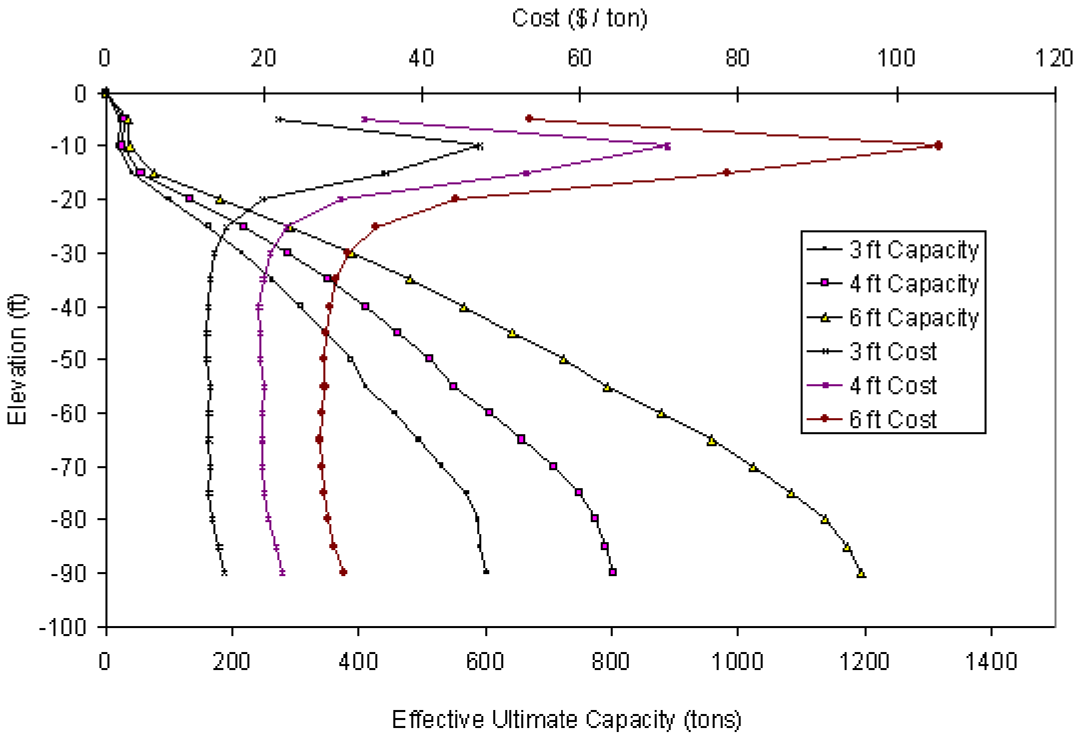


Figure 10 The effect of shaft size selection on cost.

Selecting the Most Economical Design Method. The next comparison that can be made is that which evaluates the cost effectiveness of various design/testing methods. As additional testing (beyond soil exploration) incurs extra expense, a break even analysis should be performed to justify its use. In this case, a 3 ft diameter shaft will be used due to the results shown in Figure 10 where it was consistently less costly. The maximum capacity that can be reasonable provided by a 3 ft diameter shaft will be calculated to be 602 tons at a depth of 90 ft (30D)*. The effective ultimate capacity is then reduced based on the presumption of testing (or no testing) and the appropriate resistance factor. Using these values a 3500 ton factored pier load (P_u) would require more or fewer shafts given various resistance factors as shown in Table 7.

** NOTE: As deeper excavations are possible, the ultimate structural capacity based on concrete strength should not be exceeded.*

Table 7. The effect of various design approaches on required number of shafts.

Design Method or Test Scheme	Resistance Factor	Eff. Ult. Capacity @ 90' P_n (tons)	Usable Capacity ϕP_n (tons)	Number of Shafts Required (P_u = 3500 tons)	Total Shaft Costs
Static	0.75	602	451.5	7.75 (8)	$69,750
Statnamic	0.73	602	439.5	7.96 (8)	$71,640
No testing	0.55	602	331.1	10.57 (11)	$116,640

The above shaft costs will also have to incorporate the cost of testing as well. As such, larger projects can justify more extensive testing, whereas very small projects may not warrant the expense. Figure 11 incorporates the cost of testing while extending the above example to a wide range of project sizes (expressed in terms of total structure load and not the number of shafts). The individual curves representing the various design approaches exhibit different slopes based on the permissible load carrying capability per unit length of shaft.

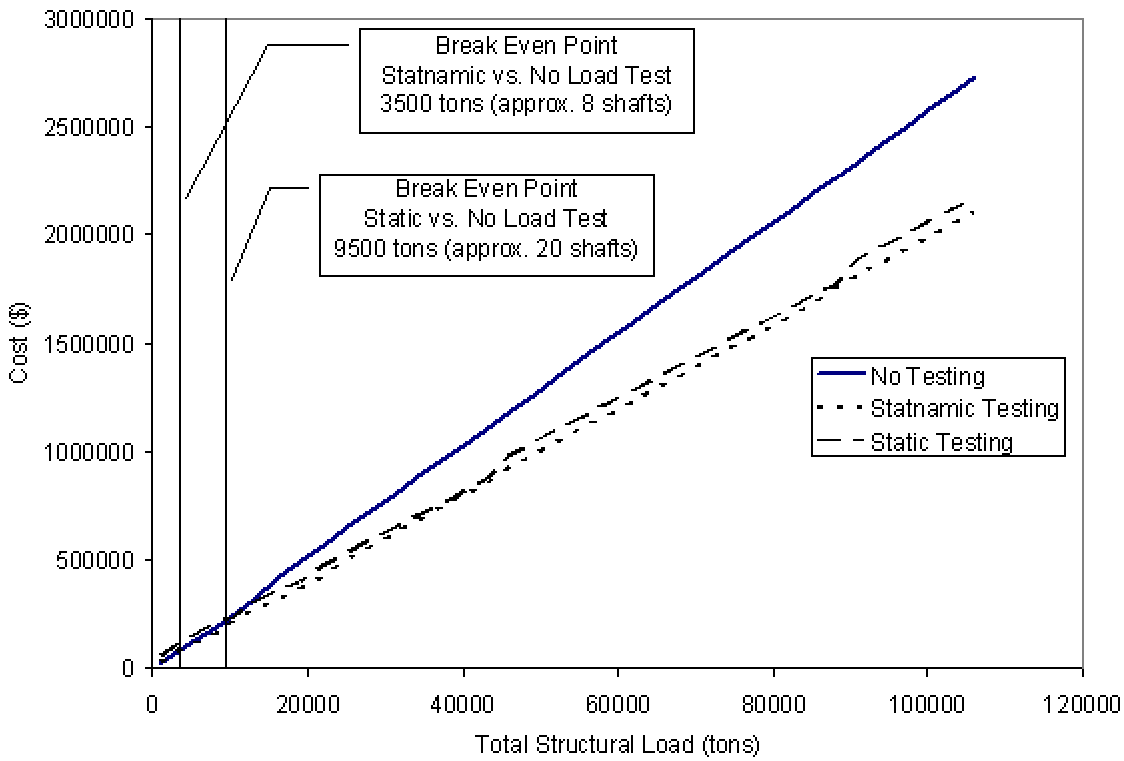


Figure 11 Break even analysis of various design / testing methods.

Comparing the costs for each design and test approach it can be seen that for smaller projects up to 8 shafts (less than 3500 tons total), no testing (over and above SPT) is most cost effective. Above 3500 tons, the cost savings produced by statnamic or static load testing become significant with statnamic costs being slightly less in all cases. The selection of load test method and the associated cost is often based on the availability of test equipment capable of producing the ultimate geotechnical capacity.

Further, the disparity between testing and no testing can be even more drastic when design phase testing can be implemented. Therein, the estimated ultimate capacity based on empirical design methods is often conservative and can be raised using the results of a test program which further widens the range of shaft numbers (testing versus no testing) required for a given pier.

In general, load test results typically show that predictions of ultimate capacity are conservative. This form of verification can be helpful in all instances: when under-predictions are severe, the design capacity of the foundations can be adjusted to provide cost savings; when over-predictions are encountered, more moderate design values can be incorporated to circumvent possible failures.

List of Abbreviations

%R	=	percent recovery of rock coring (%)
	=	adhesion factor applied to S_u (DIM)
	=	coefficient relating the vertical stress and the unit skin friction of a drilled shaft (DIM)
$_m$	=	SPT N corrected coefficient relating the vertical stress and the unit skin friction of a drilled shaft (DIM)
D	=	diameter of drilled shaft (FT)
D_b	=	depth of embedment of drilled shaft into a bearing stratum (FT)
D_p	=	diameter of the tip of a drilled shaft (FT)
f	=	angle of internal friction of soil (DEG)
f_s	=	nominal unit side shear resistance (TSF)
	=	unit weight (pcf)
k	=	empirical bearing capacity coefficient (DIM)
K	=	load transfer factor
N	=	average (uncorrected) Standard Penetration Test blow count, SPT N (Blows/FT)
N_c	=	bearing capacity factor (DIM)
N_{corr}	=	corrected SPT blow count
q_b	=	end bearing resistance (units of stress)
q_c	=	cone penetration tip resistance (units of stress)
q_s	=	average splitting tensile strength of the rock core (TSF)
Q_s	=	side shear capacity (units of force)
q_u	=	average unconfined compressive strength of the rock core (TSF)
$'_v$	=	vertical effective stress (TSF)
S_u	=	undrained shear strength (TSF)
ε	=	measured strain from embedded strain gage

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