Review of Augered Pile Practice Outside the United States

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The objective is to document procedures used abroad in order to increase U.S. transportation engineers' confidence in the augered pile system when properly applied to highway construction. Two construction systems for augered piles used extensively in Europe are described: the continuous-flight auger system and the screw-pile system. Real-time acquisition of critical construction data by electronic sensing devices ensures the integrity of such systems; one real-time data-acquisition system is examined. Simple design rules for estimating axial capacity are documented, and some innovative design and construction methods are evaluated.

Augered piles are commonly used for building and transportation construction in Europe and other parts of the world. Augered piles can be distinguished from drilled shafts (bored piles) and from driven piles by the magnitude of effective stress changes they produce in the surrounding soil during construction. To create a drilled shaft, a commonplace in the United States, an auger is repeatedly inserted and withdrawn from the borehole to excavate soil, then the excavated borehole is filled with concrete. In general no attempt is made to maintain the stresses that existed in the ground before construction. With a driven pile, the soil is displaced—even in a so-called nondisplacement pile—and the ground stresses are increased. With an augered pile, ground stresses are maintained near the value that existed before construction by using a continuous flight auger and maintaining high pressure in the concrete as the auger is withdrawn. In principle, the augered pile possesses load-settlement behavior that falls between that of a drilled shaft and a driven pile (Figure 1).

Public transportation facilities in the United States have taken almost no advantage of augered piles. Reasons for not using them include concerns about control of structural integrity and unavailability of design methods (methods for capacity estimation). EBA Engineering Inc. recently reported on U.S. practice; the study discussed equipment, costs, and case histories (*1*). However, few authorities and experts contacted by EBA were willing to discuss design methods for augered piles, which may suggest that no one has developed a standard practice for estimating static capacity. In contrast, quality control and assurance in U.S. practice are covered in detail in a recent manual published by the Deep Foundations Institute (*2*), which describes materials, equipment, tolerances, adjacent piles, installation procedures, and other issues in a guide-specification format for U.S. practice.

Certain aspects of European and international practice for construction and design of augered piles may be of interest to U.S. transportation foundation engineers who are considering whether to use augered piles. Two of the many types of cast-in-place, augered piles commonly used in Europe are the continuous flight auger (CFA) pile, in which excavation is made with a continuous flight auger and the borehole is grouted as the auger is withdrawn (commonly known in the United States as "augercast" piles), and the screw pile (SP), in which a single-turn auger is screwed into the soil and then screwed back out as the concrete is placed. Such piles typically range from 0.3 to 0.8 m in diameter and may be up to 30 m deep.

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CONSTRUCTION PROCEDURES FOR CFA PILES AND SPs

General methods of construction for CFA piles and SPs are shown in Figure 2 (*3,4*). With the CFA pile, soil is excavated by a double-flight continuous auger (Figure 2*a*). Following the Starsol CFA method, once the maximum depth has been reached, the auger is withdrawn a small distance (0.5 m); however, the discharge end of the grout pump line, which is housed but slides freely within the central stem of the auger, remains on the bottom of the hole, and the space beneath the auger and the base of the shaft is grouted with high-pressure grout or concrete with very fine coarse aggregate, which may or may not be fiber reinforced (Figure 2*b*). This procedure contrasts with past U.S. practice, whereby the auger is lifted about 0.3 m, grout is introduced through the stem, and the auger turned and thrust back to the bottom of the borehole once grout pressure increases sufficiently (*4*). Thereafter the auger and grout tube are lifted together, with the outlet port on the grout tube remaining a short distance below the base of the auger during continuous grouting. A reinforcing cage, if specified, is then in-

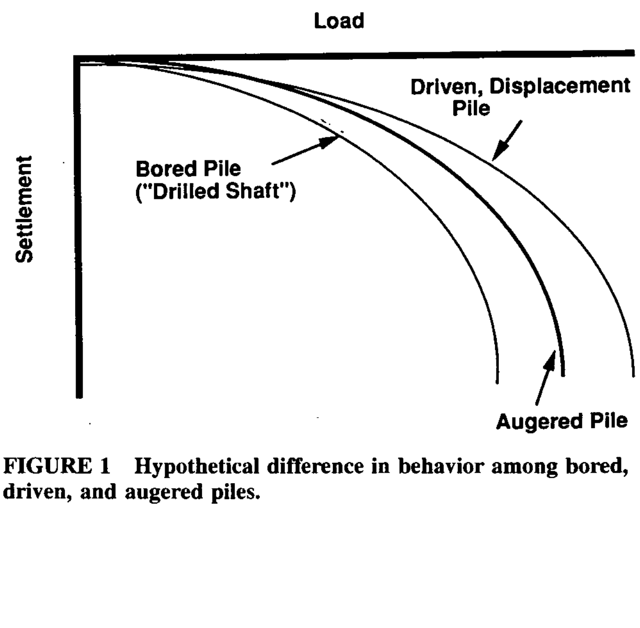

FIGURE 1 Hypothetical difference in behavior among bored, driven, and augered piles.

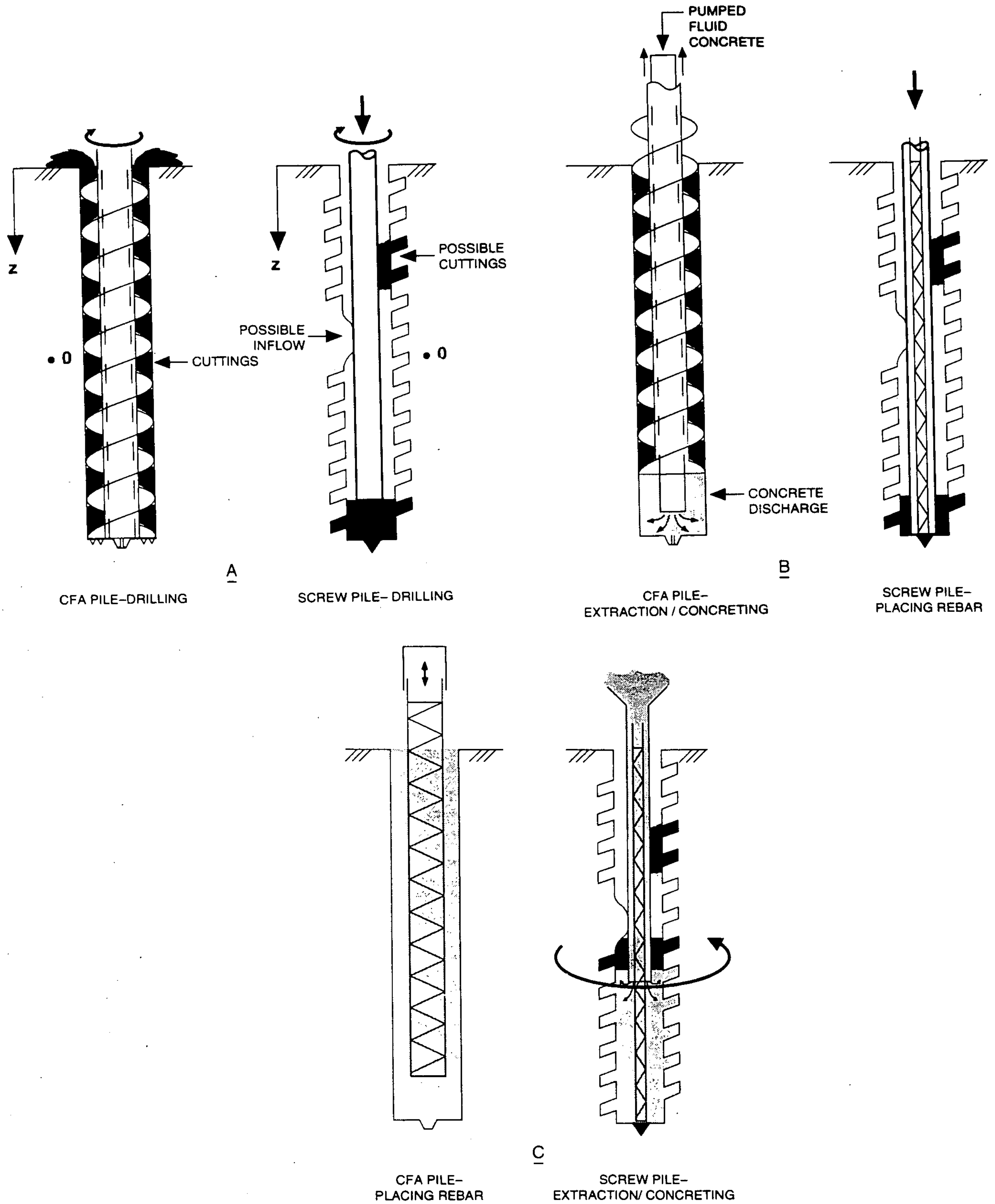

FIGURE 2 Abbreviated construction procedures for continuous flight auger piles and screw piles (*3,4*).

serted into the fresh grout by vibrating it into place after the auger has been completely withdrawn (Figure 2*c*). Obviously the grout or concrete must be designed to resist segregation caused by vibrating the cage.

With the SP, the borehole is formed by rotating a thick-flanged, single-turn auger into the soil without removing the soil (or removing as little as possible) (Figure 2*a*). Instead, the soil is compressed back into the sides of the borehole, especially if the soil possesses some cohesion, forming a screw "tap." When an SP borehole is driven in granular soils, some soil deforms inward ("possible cuttings" and "possible inflow") (Figures 2*a* and 2*b*). Once the maximum depth is reached, the reinforcing cage, if any, is placed through the hollow axle of the auger before any concrete is placed (Figure 2*b*). The axle typically has a larger diameter than the stem of the CFA pile, to accommodate cage placement. Finally, the auger is screwed out, reestablishing the tap pattern in areas where inflow or caving may have occurred. Simultaneously, concrete is added through the hollow core of the axle from a hopper affixed to the top of the axle, providing several meters of excess head for gravity flow of the concrete into the tapped borehole through the bottom of the axle (Figure 2*c*). The point of the auger, which protects the open axle during drilling, is left on the bottom of the borehole as the auger and axle are retracted, which also presumably ensures minimal disturbance of the bearing surface.

The shape of the CFA pile is generally cylindrical, whereas that of the screw pile is generally that of a cylindrical screw. Concrete strength and fluidity are important in the construction of both types of piles, but especially for the SP.

When installing either a CFA pile or SP the strategy is to ensure that effective stresses in the soil are maintained during both excavation and concrete placement. In Figure 2, horizontal effective stresses are measured at a hypothetical Point 0. During construction, effective stresses exist in two ways, as shown conceptually in Figure 3, which depicts the lateral earth pressure coefficient K′ as a function of the depth of the tip of the auger. In one scenario, depicted by the dashed line, the construction process produces a steadily increasing K′, except during the short period of time after the tip of the auger passes Point 0. Note especially that the effective stresses on withdrawal of the auger generally increase as a result of maintenance of high fluid-grout pressure; so, at the end of the process, the change in K′ (ΔK′) is either zero or slightly positive. In the scenario depicted by the solid line, some stress relief occurs after the tip of the auger has passed Point 0, perhaps to inflowing, waterbearing sand. Reductions also occur during extraction of the auger, perhaps because sufficient pressure is not maintained in the grout during extraction or the auger is withdrawn too rapidly.

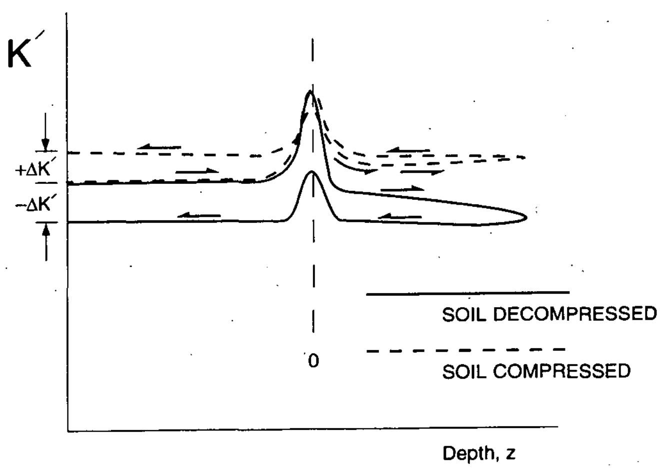

FIGURE 3 Change in lateral earth pressure coefficient during construction.

Quality Control

Maintenance of insufficient lateral stress in the soil may be accompanied by inward movement of the soil and loss of ground, which can be detrimental to adjacent structures. Maintenance of concrete or grout pressures lower than the total soil pressures beneath the extracting auger may cause necking and structurally defective piles. Instrumentation is frequently used in European practice to prevent these two phenomena. For example, the Enbesol instrumentation system, used to monitor augered pile construction by Soletanche, is shown in Figure 4. Four parameters are monitored: (*a*) machine torque as the auger is being inserted, (*b*) drilling rate, that is, penetration velocity, (*c*) concrete or grout pressure at the pump, and (*d*) ratio of actual to theoretical concrete or grout taken by the borehole.

These data are acquired by electronic sensing devices, and a continuous printout is usually provided to the drilling machine operator and kept for construction records. The operator and field engineer can use the data display diagnostically to correct errors in drilling. For example, if the grout pressure drops below the total vertical pressure in the soil at a given level and the actual/theoretical grout concrete take drops below 1, there is probably a necking problem in the pile. In order to correct the problem, the operator can stop auger extraction and concreting, redrill through the fluid concrete to below the level of the probable neck, then reintroduce the grout or concrete at the proper pressure (perhaps after increasing pump pressure), and extract the auger (perhaps more slowly than before). Other CFA systems used in Europe have similar automated data-acquisition systems.

In addition, penetration velocity data can be used to assess quality, and torque data can be used to verify crudely the soil profile and shaft resistance of the constructed pile.

The one difficulty with such real-time systems is that concrete or grout pressure is measured either at the pump or in the pump line at the top of the auger, so that an assumption must be made about head loss in the grout tube that extends through the stem of the auger. A device that measures pressure directly on the bottom of the auger would be better.

Variations

Several variations exist on the construction method described previously. One such promising variation has been applied by European contractors for several years: the use of an expandable body (*5*). The body consists of a folded, thin steel sleeve. With a CFA pile or an SP, the expanding body can be affixed to the bottom of the reinforcing cage. After introduction it fits between the bottom of the cage and the bottom of the borehole, where it then expands against the sides and base of the borehole when filled with grout under high pressure. Massarsch and Wetterling found significant increases in pile capacity (*5*).

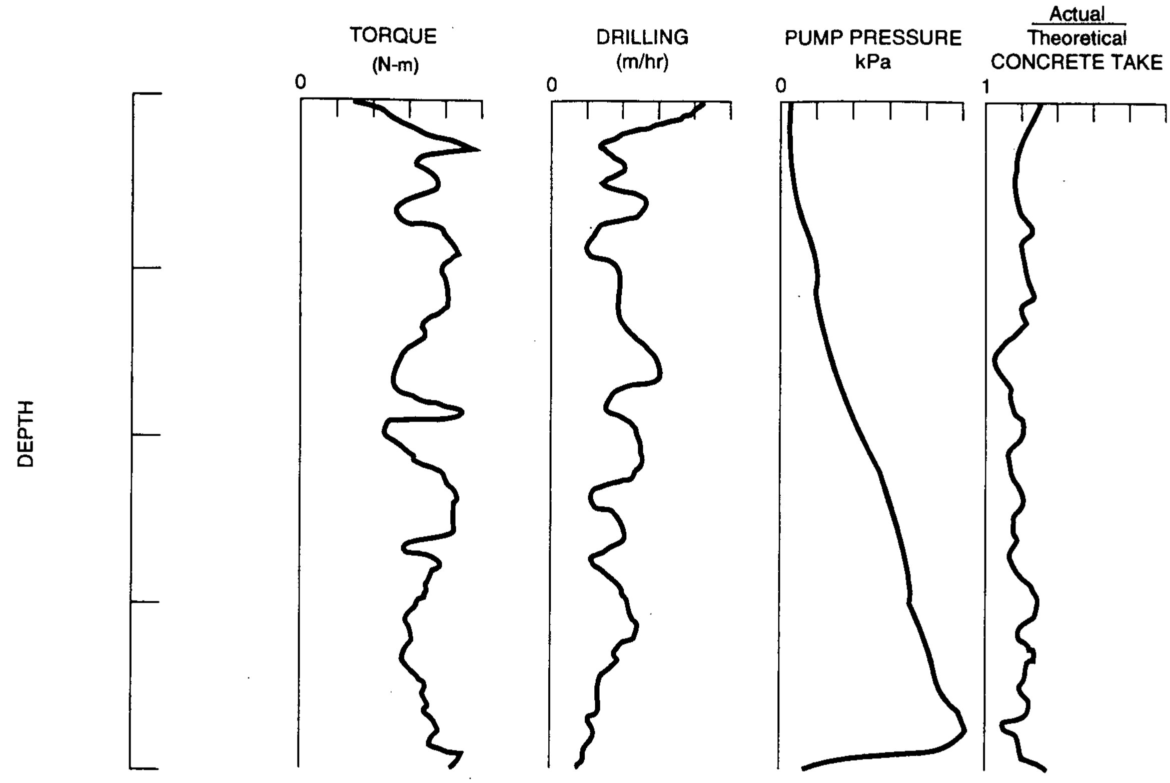

FIGURE 4 Monitored parameters during construction (*4*).

ENSURING NO SOIL DECOMPRESSION WITH CFA PILES

One of the key concerns when using CFA piles is that the soil surrounding the pile not be decompressed during drilling (i.e., effective ground stresses not be reduced through inward flowing of the soil). Viggiani (*6*) presents a simple analysis of the displacement produced by the hollow stem of the CFA auger, compared with the soil removed by the drilling action of the auger. If d_o = diameter of the auger stem (axle), and v = rate of downward penetration of the auger, the volume of soil displaced by the stem V_d in a time increment Δt is given by

$$V_d = \frac{\pi d_o^2}{4} v \Delta t \quad (1)$$

The volume of soil removed by the rotating action of the auger V_r is given by

$$V_r = \frac{\pi}{4} (d^2 - d_o^2)(n p - v) \Delta t \quad (2)$$

where

d = outside diameter of the auger (flange tip to flange tip),
n = number of revolutions of the auger per unit of time, and
p = pitch of the auger in units of length (e.g., m per turn).

For there to be no soil decompression, $V_d \geq V_r$, so that

$$v \geq n p \left[1 - \frac{d_o^2}{d^2}\right] \quad (3)$$

If the velocity of penetration is less than the expression on the right in Equation 3, decompression can occur. In fact, decompression can occur even if the above condition is satisfied, if the soil being excavated is waterbearing sand with sufficient groundwater head to force the cuttings up the auger. A contractor must provide a drilling rig with sufficient torque and crowd to obtain the velocity of penetration in Equation 3. Otherwise, the equations for computing bearing capacity may not be conservative. According to Van Impe et al. (*7*), the same relationship can be used for screw piles.

ESTIMATION OF AXIAL CAPACITY OF CFA PILES

European and other engineers use several methods for estimating the static capacity of CFA augered piles.

German Standard

According to Rizkallah (*8*), the German standard for estimating capacity of augered, cast-in-place piles does not distinguish between bored piles (drilled shafts) and CFA piles. DIN 4014 (*9*) specifies computations based on the tip resistance, q_c, in the cone penetration test, as follows:

Sand

$$f_{max} = 0.008 q_c \quad (4)$$

$$q_{0.05}\ (\text{MPa}) = 0.12\ q_c + 0.1 \qquad (q_c \le 25\ \text{MPa}) \tag{5}$$

where f_{max} is the maximum unit side shearing resistance on the pile, which has the nominal diameter of the auger, and $q_{0.05}$ is the unit end-bearing corresponding to a movement of 5 percent of the pile diameter, which, according to Reese and O'Neill (*10*) can be considered the deflection corresponding to end-bearing failure in bored piles. Note that the ultimate axial capacity of the pile is equal to the net unit base capacity $q_{0.05}$ times the base area, plus the unit shaft capacity f_{max} times the shaft area (taken in segments, if appropriate).

Clay

$$f_{max}\ (\text{MPa}) = 0.02 + 0.2\ c_u \quad (0.025 \le c_u \le 0.2\ \text{MPa}) \tag{6}$$

$$q_{0.05} = 6\ c_u \qquad (0.025 \le c_u \le 0.2\ \text{MPa}) \tag{7}$$

where the undrained shear strength c_u is given by Equation 8.

$$c_u = \frac{q_c - \sigma_{vz}}{16 - 22} \tag{8}$$

and σ_{vz} is the total vertical stress at the elevation of the bottom of the pile. Presumably, c_u could also be determined conservatively from unconfined compression tests, with $c_u = 0.5\ q_u$, where q_u = unconfined compression strength. This method is typical of other methods used currently in Europe.

Rizkallah compared the results of axial loading tests from a large data base and concluded that the above formulae were accurate for prediction of capacity of "nondisplacement" CFA piles and were conservative for predicting capacity of "displacement-type" screw piles.

Other Methods

Viggiani (*6*) suggests simple correlations for CFA piles in cohesionless pyroclastic soils, based on pile-loading tests and corresponding cone penetration tests in the Naples, Italy, area:

$$f_{max} = \alpha\ q_c \tag{9}$$

$$q_b = q_{c\,\text{avg}(+4d,-4d)} \tag{10}$$

where

q_b = net ultimate unit-bearing capacity of the pile base,
$q_{c\,\text{avg}(+4d,-4d)}$ = average CPT tip reading between 4 pile diameters above the base and 4 diameters below the base, and
α = a correlation factor given by Equation 11.

$$\alpha = \frac{6.6 + 0.32\ q_c\ (\text{MPa})}{300 + 60\ q_c\ (\text{MPa})} \tag{11}$$

Decourt (*11*) proposed a method for estimating the capacity of CFA piles in residual silts from the maximum torque measured when twisting a standard split-spoon sampler—after having been driven into the bottom of the sample borehole—as per a normal standard penetration test (SPT), to remove the influence of the dynamic driving conditions in the normal SPT. Correlations with loading tests indicate that

$$f_{max} = f_{max}(\text{SPT-T test}) \tag{12}$$

$$q_b = 0.5\ K'\ N_{eq} \tag{13}$$

where K' is a soil factor [0.10 MPa for clays, 0.12 MPa for clayey silts, 0.14 MPa for sandy silts, and 0.20 MPa for sands (at the base of the pile)] and N_{eq} is the average equivalent N value from the SPT-T (blows/0.3 m) test near the base of the pile, which can be taken as a dimensionless correlation factor. According to Decourt, in residual silts, $N_{eq} = T/1.2$, where T is the torque (in kgf-m, units reported in Decourt's original publication) measured by twisting the SPT split-spoon sampler. For large bored piles and barrettes, Decourt suggests that the corresponding values from Equations 12 and 13 be halved (for unit shaft resistance, Equation 12) and doubled (for base capacity, Equation 13).

D. O. Wong (personal communication, 1993) studied the load-settlement behavior of two CFA piles in the United States that were constructed in hydraulic fill that behaved as a normally consolidated clay. Wong concluded that $f_{max} = c_u$, where c_u is measured following the usual U.S. practice of recovering and testing cohesive soil samples using undrained compression tests. Reese et al. (*12*) describe tests on CFA piles at a site with a layered profile of normally consolidated and heavily overconsolidated clay. S.-T. Wang (personal communication, 1993) indicated that the parameters recommended by Reese and O'Neill (*10*) for estimating the capacities of bored piles provided accurate estimates of capacities of the augered piles at this test site, that is, on the average $f_{max} = 0.55\ c_u$ and unit bearing capacity = $9\ c_u$ (at pile base).

Neely (*13*), on review of "augercast" pile tests in sand from around the world, found that the unit shaft resistance was essentially independent of the relative density of the sand, which is consistent with Reese and O'Neill's study (*10*), which addresses only bored piles. Neely proposed that Equation 14 be used to evaluate average unit-shaft resistance, based on an analysis of 58 loading tests in sand:

$$f_{max}(avg) = \beta\ \sigma'_v(avg) \tag{14}$$

where β is a correlation factor given in Figure 5 (*13*) and $\sigma'_v(avg)$ is the average vertical effective stress in the soil between the pile head and base. It follows, then, that $f_{max}(avg)$ is the average unit-shaft resistance along the pile.

Neely, through analysis of the same data base, also proposed that the unit end-bearing resistance of CFA piles in sand, q_b, could be related to the uncorrected SPT value (N) in blows/0.3 m at the pile base, as follows:

$$q_b(\text{tsf}) = 1.9\ N \le 75\ \text{tsf, or} \tag{15a}$$

$$q_b(\text{MPa}) = 0.19\ N \le 7.5\ \text{MPa} \tag{15b}$$

ESTIMATION OF AXIAL CAPACITY OF SCREW PILES

Bustamante and Gianeselli (*14*) summarize procedures for calculating axial capacities of screw piles from in situ soil tests based on numerous correlations with field tests on such piles.

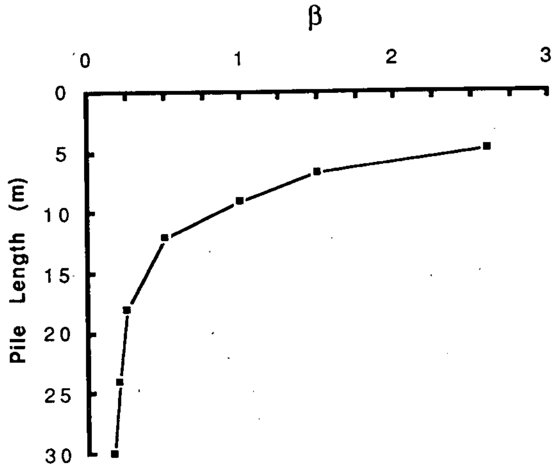

FIGURE 5 Correlation factor β versus pile length (*13*).

TABLE 1 *K* Values for Various Geomaterials

Type of Geomaterial	Type of *In Situ* Test		
	Menard PMT	CPT	SPT
Clay	1.6 - 1.8	0.55 - 0.65	0.9 - 1.2
Sand	3.6 - 4.2	0.50 - 0.75	1.8 - 2.1
Gravels*	≥ 3.6	≥ 0.5	unknown
Chalk	≥ 2.4	≥ 0.6	≥ 2.6
Marl	≥ 2.4	≥ 0.7	≥ 1.2

* CPT and SPT results are questionable

Base Capacity

The base capacity Q_b is computed in Equation 16:

$$Q_b(MN) = K\,A_b\,\alpha' \quad (16)$$

where

A_b = base area of the pile, conservatively estimated using diameter as 0.9 d, where d is the outside diameter of the flanges (m^2);
α' = adjusted ultimate base pressure factor [limit pressure p_L at pile base (MPa) for Menard-type pressuremeter, q_c at pile base for CPT (MPa), or SPT N value at pile base (blows/0.3 m)], and
K = dimensionless correlation factor from Table 1.

In each case an adjusted value of the in situ test parameter is taken for the computation of α', as follows:

Menard-Type Pressuremeter

$$p_L(\text{adjusted}) = [(p_L + a)(p_L)(p_L - a)]^{0.333} \quad (17)$$

where

$p_L + a$ = ultimate limit pressure at 0.5 m below base of pile,
$p_L - a$ = ultimate limit pressure at 0.5 m above base of pile, and
p_L = ultimate limit pressure at elevation of base of pile.

Cone Penetration Test

The following procedure is used to compute q_c(adjusted):

1. Smooth the q_c versus depth curve to eliminate local irregularities;
2. From the smoothed curve, determine mean q_c ($q_{c\,\text{mean}}$) from 1.5 pile diameters above the base of the pile to 1.5 diameters below the base of the pile;
3. Clip all q_c values < 0.7 $q_{c\,\text{mean}}$ and > 1.3 $q_{c\,\text{mean}}$; and
4. Compute q_c(adjusted) as the mean q_c value obtained from the clipped, smoothed q_c-depth curves within the depth interval indicated in Step 2.

Note that it is assumed that the M1 mechanical cone based on the International Society of Soil Mechanics and Foundation Engineering (ISSMFE) Standard TC-16 has been used to the obtain q_c values. However, if the electronic cone has been used, the values must be corrected according to Equation 18:

$$q_c(\text{M1}) = \beta\, q_c(\text{electronic}), \quad (18)$$

where β is 1.4 to 1.7 for cohesive soils and 1.3 for saturated sands.

Standard Penetration Test

$$N(\text{adjusted}) = 1000[(N_{+a})(N)(N_{-a})]^{0.333} \quad (19)$$

where

N = N in blows per 0.3 m, uncorrected, at elevation of pile base,
N_{+a} = N in blows per 0.3 m, uncorrected, at 0.5 m below pile base, and
N_{-a} = N in blows per 0.3 m, uncorrected, at 0.5 m above pile base.

Shaft Capacity

The ultimate shaft capacity, Q_s, is computed from Equation 20:

$$Q_{u,s} = \sum_{i=1}^{N} q_{si} S_{li} \quad (20)$$

where

q_{si} = ultimate unit shaft resistance (equivalent to $f_{\max}$) in MPa in Segment i,
i = Segment number,
S_{li} = lateral or perimeter area of Segment i, using shaft diameter = 0.9d, and
N = total number of segments.

The value for q_{si} is chosen from Figure 6 (*14*), based on curve selection indicated in Table 2.

The expressions on the right-hand sides of Equations 16 and 20 can be added to give the ultimate pile capacity. The author

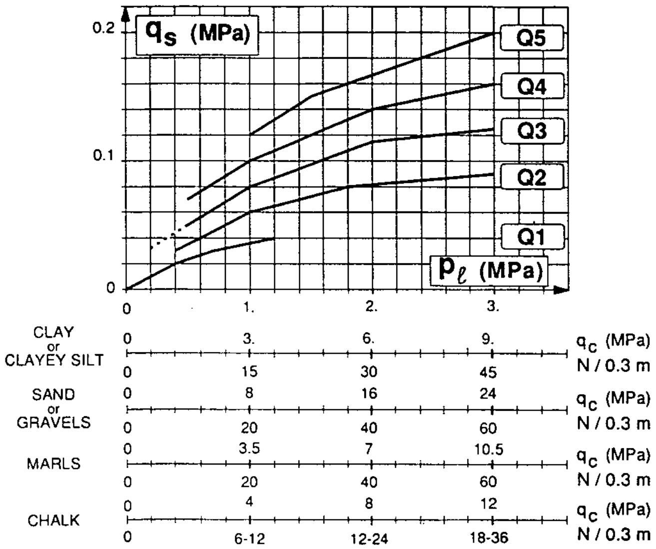

FIGURE 6 q_s **(MPa) based on ultimate limit pressure from Menard-type pressuremeter,** q_c **from cone penetration test, and N (uncorrected) from standard penetration test (*14*).**

TABLE 2 Suggested Curves To Be Used from Figure 6

Geomaterial	Range of Values for p_L and q_c		Applicable Curve (Fig. 6)
	p_L (MPa)	q_c (MPa)	
Clay, clayey silt	< 0.3	< 1.0	Q1
or sandy clay	≥ 0.5	≥ 1.5	Q3
	≥ 1.0	≥3.0	Q4
Sand or gravel	< 0.3	< 1.0	Q1
	> 0.5	> 3.5	Q4
	> 1.2	> 8.0	Q5
Chalk	≥ 0.5	≥ 1.5	Q4
	> 1.2	> 4.5	Q5
Marl	< 1.2	< 4.0	Q4
	≥ 1.5	≥ 4.0	Q5

Note that Table 2 refers only to cast-in-place screw piles, and not for piles that are cased.

suggests that a factor of safety of 2 be used on the result to assign allowable pile capacity. Where piles are used under settlement-sensitive structures, settlement at working load should be checked.

PILE SETTLEMENT

Recent studies focus on how to predict the settlement of augered piles and thereby estimate service-limit loads for the structures they support. One such method unique to CFA piles is described by Fleming (*15*), who uses hyperbolic functions to represent load-movement behavior of the shaft and base and considers elastic shortening of the pile. The reader is referred to that reference for further details.

CONCLUSION

Augered piles have been used successfully in Europe and elsewhere for transportation engineering. With the application of modern monitoring devices for concrete pressure, volume, and other parameters, and equations for static capacity, based on numerous correlations between values given by in situ testing tools and observed behavior of test piles, it is possible to use augered piles. A number of methods have been documented in this paper. Applications of foreign CFA practice by the U.S. transportation-engineering community should produce increased confidence in the use of augered piles for the construction of bridge and wall foundations.

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