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Measured end resistance of CFA and drilled displacement piles in San Francisco Area alluvial clay

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Continuous Flight Auger (CFA) and drilled displacement (DD) piles in the San Francisco (California, USA) area are typically designed using a combination of side- and end resistances. For moderately-sized buildings, these piles are typically 50 to 100 ft (about 15 to 30 m) in length and often bear in Pleistocene epoch alluvium consisting primarily of clay with interbedded sand seams. It can be unconservative to rely upon the higher consistency sand seams because their depth, thickness, and consistency can vary dramatically over short distances. A more robust design approach assigns an end resistance based on the strength of the clay. The fully mobilised end resistance from fifteen (15) high quality axial compression loading tests performed on cast-in-place piles are compared to the average net cone resistance for one diameter below the pile tip. The comparison suggests that direct estimation of the end resistance using local load testing data will result in higher end resistances than will the conventional bearing factor of 9 times the estimated undrained shear strength derived from the cone penetration test.

Keywords: Cast-in-place piles; End resistance; Pile loading tests

Introduction

Continuous Flight Auger (CFA) and Drilled Displacement (DD) cast-in-place piles in the San Francisco (California, USA) area are typically designed using a combination of side- and end resistances based on cone penetration test (CPT) measurements. For moderately sized buildings in locations where bedrock is prohibitively deep, these piles are typically 15 to 30 m (about 50 to 100 ft) in length and bear in alluvium consisting primarily of clay with interbedded sand seams. It can be unconservative to rely upon the sand seams because their depth, thickness, and density of the sand seams can vary dramatically over short distances resulting in uncertainty regarding the conditions at each pile location over the footprint of a structure. A more robust design approach assigns an end resistance based on the characteristics of the clay. In an effort to better quantify the fully mobilised end resistance for San Francisco Bay Area alluvial clay, the fully mobilised end resistance from fifteen (15) high quality axial compression loading tests performed on CFA and DD piles are compared to the net cone resistance averaged over one diameter below the pile tip.

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Literature review

Skempton (1959) concluded that a bearing factor of 9 applied to the undrained shear strength was appropriate for bored piles installed in London clay based on the results of 25 loading tests. Bustamante and Gianeselli (1982), using the data from 197 deep foundation load tests performed at 48 sites with varying soil conditions in France, proposed a direct correlation (colloquially referred to as the 'LCPC Method' or 'French Method') between an average cone tip resistance and the end resistance (R_{end}) by way of an empirical bearing capacity factor k_c:

$$R_{\text{end}} = q_{ca} * k_c * A_{\text{pile}} \tag{1}$$

where q_{ca} is the equivalent cone resistance at the pile tip; A_{pile} is the cross-sectional area of the pile.

The LCPC Method bearing capacity factor (k_c) for compact to stiff clay differs for non-displacement piles (designated Group I) and displacement piles (designated Group II) as 0.45 and 0.55, respectively. Recognising that k_c is essentially equivalent to $9/N_k$ (where N_k is the cone factor for undrained shear strength), the approximate equivalent cone factors are 20 and 16, respectively.

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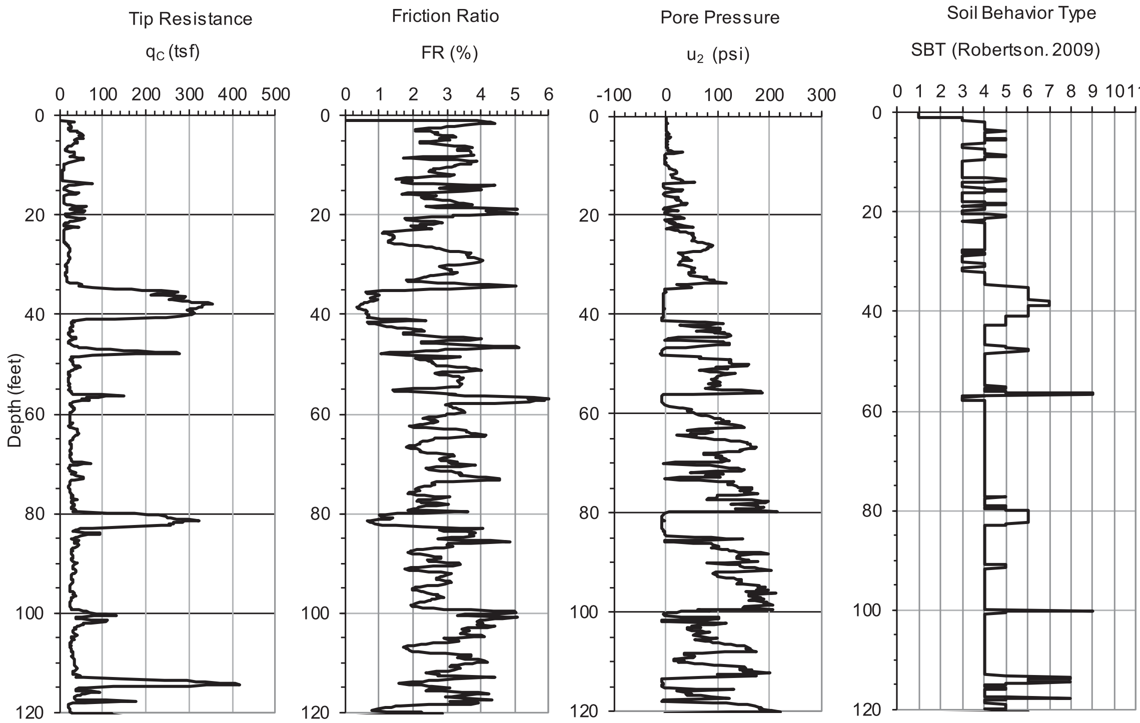


1 CPT sounding in San Francisco Bay Area clay (Pleistocene clay sediments below 40 feet)

The LCPC Method and other similar CPT based methods (Nottingham 1975; Schmertmann 1978; Turnay and Fakhroo 1981; Eslami and Fellenius 1997) calculate the equivalent cone resistance at the pile tip using various forms of data filtering and smoothing to find the representative average cone resistance at and below the pile tip. The filtering is intended to avoid the disproportionate influence of data 'spikes'.

O'Neill, Vipulanandan, Ata and Tan (1999) evaluated the behaviour of 43 continuous flight auger (CFA) test piles in the Houston-Gulf Coast area and Florida and determined the LCPC Method prediction was the most accurate for prediction method for piles in clay. The LCPC method has been incorporated into commercially available CPT analysis software (Geologismiki 2007).

Because the test data for this study were selected to represent clay sites with negligible interbedded sands, it was possible and considered appropriate to rely on the mean tip resistance of the clay only extending one diameter below the pile tip. Measured resistances can then be compared to the LCPC Method predictions to evaluate the appropriateness of typical k_c factors for San Francisco Bay Area alluvial clays.

Subsurface conditions

The subject load tests are all located in the San Francisco Bay Area which is recognised as part of Coastal Ranges geomorphic province. The Coastal Ranges are a series of alternating parallel mountain ranges and valleys trending from the southeast to the northwest along the coast of California that were formed when the Pacific Plate collided with the North American Continental Plate. During the last Ice age (Pleistocene epoch), the sea level was much lower than today, and the current San Francisco Bay Area was a series of broad valleys between the Coastal Ranges. At the beginning of the Holocene epoch, glaciers melted causing a rapid rise in sea level. The resulting influx of seawater through the Golden Gate filled the current San Francisco Bay. As inflow of sea water slowed, the rivers draining into the San Francisco Bay Area continued to deposit material derived from the Coastal Ranges. These youngest Holocene marine sediments continue to accumulate and are recognised as the Bay Muds. The clay that is the subject of this paper is Pleistocene alluvium deposited in the Coastal Range valleys outside the extent of the current San Francisco Bay before the end of the last Ice age. A detailed description of the geologic history of the San Francisco Bay Area is presented by Helley, Lajoie, Spangle and Blair (1979).

The result of a typical CPT sounding for the subject soil conditions is illustrated in Fig. 1. The clay of interest in this sounding exhibit a cone tip resistance of approximately 30 tsf (2.9 MPa). It is the authors' local experience that the interbedded sand layers are typically 2 to 5 ft (0.6 to 1.5 m) thick and rarely greater than 10 ft (3 m) thick. Tip resistance in interbedded sand layers for the example shown in Fig. 1 range from 200 tsf (19.2 MPa) to over 400 tsf (38.4 MPa), which is typical for the area. Ground water is typically 5 to 15 ft (1.5 m to 4.5 m) from the ground surface.

As illustrated by Fig. 1, cone tip resistance of the clay is relatively uniform with depth except for the localised spikes at interbedded sand seams. This is characteristic of the

Table 1. Test pile information

Location	Auger Dia (in)	Length (ft)	Overbreak (%)	CFA/DD Pile	Soil Conditions	Q_b* (ksf)
Redwood City	16	70.0	35	CFA	Clay	26.5
San Jose	16	65.8	23	CFA	Clay	34.9
San Jose	16	60.1	23	CFA	Clay w/ sand seams	52.4
San Jose	16	60.0	26	CFA	Clay	65.3
Mountain View	16	80.1	47	DD	Clay	43.7
Oakland	16	36.1	21	CFA	Clay w/ sand seams	35.6
Burlingame	16	62.1	33	CFA	Clay w/sand seams	86.3
Redwood City	16	58.7	15	DD	Clay	49.7
Redwood City	16	59.0	14	DD	Clay	50.4
Rohnert Park	16	50.2	34	CFA	Clay	21.5
Redwood City	18	65.0	19	DD	Clay	15.0
Redwood City	16	66.0	20	DD	Clay	42.4
Belmont	16	60.1	28	DD	Clay	33.7
Belmont	16	60.0	26	DD	Clay	22.7
Sunnyvale	18	70.0	41	CFA	Clay	28.2

1 in = 25.4 mm; 1 ft = .3048 m; 1 ksf = 47.88 kPa

San Francisco Bay Area sites used for the current study. The depth and thickness of the interbedded sand seams can vary significantly between CPT soundings. Thus, the significant uncertainly associated with the competency of the sand seems can be problematic for design.

CFA and DD cast-in-place piles

The cast-in-place piles included in this study were installed using both conventional continuous flight auger (CFA) and drilled displacement (specifically the Omega pile® tool) platforms. Conventional CFA piles (also known as augered cast-in-place piles, augered pressure-grouted piles, and auger cast piles) are installed by the excavating soil by the rotating a continuously flighted auger into the ground and then placing fluid cement grout into the excavated volume as the auger is extracted. Drilled displacement (DD) piles, as the name implies, are installed by first creating a hole in the ground by advancing a displacement tool into the ground with a combination of substantial torque and crowd. Then, cement grout is pumped through the drill stem and exits the displacement tool as the drill stem and displacement tool are extracted.

The primary installation difference between CFA and drilled displacement piles is that the former creates a hole by excavation and the latter by displacement. There can be a measurable increase in cone tip resistance as a result of the displacement process (Siegel, Nesmith, Nesmith and Cargill 2007); however, the difference between the two installation methods is intuitively less prominent beneath the pile tip than along the pile length. Regardless, this study differentiates between the two installation methods to allow for evaluation of installation method effects.

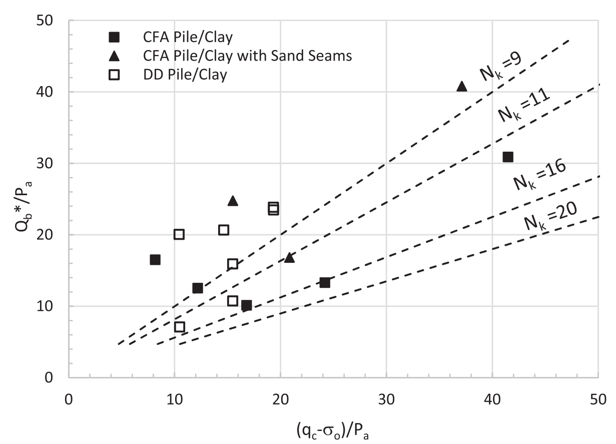
There is uncertainty in the actual cross-sectional area of CFA and DD piles because of the influence of drilling/augering and grouting. For the purposes of this study, the tip area is assumed to be the actual volume of grout divided by the pile length (i.e. average cross-sectional area), which is greater than the tip area based on the nominal auger or displacement tool diameter, due to positive grout factors greater than 1.0 (positive overbreak). Therefore, calculated end resistances in the current study, based on estimated as-built tip area, will be lower than calculated end resistance based on the nominal tool diameter (Table 1).

Loading test procedures

The subject piles were tested by the authors in accordance with ASTM D1143 (2013) within 7 to 14 days after installation. Interpretation of end resistance was accomplished using strain gage data based on procedures described in Siegel (2010). The top load application was terminated after the load-displacement curve achieved the definition of geotechnical failure according to the 90% Brinch Hansen criteria (Brinch Hansen 1963) and corresponding end resistance was considered as fully mobilised.

End resistance

Presented in Fig. 2 is the unit end resistance (Q_b*) determined from 15 pile loading tests plotted versus the



2 Unit end resistance in San Francisco Bay Area clay (Pleistocene Alluvium)

calculated mean net cone resistance (q_c-σ_o) one diameter below the pile tip. Both the unit end resistance and the average net cone resistance have been normalised to atmospheric pressure ($P_a = 1$ atm = 2116 psf = 101 kPa). The asterisk (*) in Q_b* indicates that an adjustment was made to account for the actual grout volume as discussed above. Shown for reference are dashed lines representing the relationship between tip resistance, undrained shear strength (for three different cone factors) and the theoretical end resistance using a bearing capacity factor of 9. The trendlines for N_k=16 and N_k=20 represent the relationship used in the LCPC Method to estimate tip resistance based on CPT measurements.

As may be expected, there is a general trend of increasing end resistance with increasing average net cone resistance. There is no indication that drilled displacement (DD) piles exhibit a higher end resistance than CFA piles; suggesting that, in general, clays below the pile tip are not amenable to improvement by the displacement pile system. Similarly, there is no indication that the presence of thin sand seams near the pile tip increased the fully mobilised end resistance. The interpretations of these tests indicate that the accepted approach using a bearing capacity factor of 9 coupled with the CPT-based undrained shear strengths tends to underpredict the fully mobilised end resistance in these San Francisco Bay area clays.

Conclusions

The following conclusions are presented based on the pile loading test data and the associated interpretations presented herein:

1. The trend is that higher fully mobilised end resistances for cast-in-place piles corresponds with greater net cone tip resistances for San Francisco Bay Area clays (Pleistocene alluvial sediments) represented by the project sites included in the database.
2. There is no indication that drilled displacement (DD) piles exhibit a higher end resistance than CFA piles; however, it is probable that, in general, clays are not amenable to improvement by DD pile installation.
3. There is no indication that the presence of thin sand seams increased the fully mobilised end resistance. It is intuitive that substantial sand seams near the pile tip will generally increase the fully mobilised end resistance in these Bay area clays. However, this study intentionally selected data to represent clay sites with negligible interbedded sands near the tip of the piles.
4. The interpretations of these test data indicate that the accepted approach using a bearing capacity factor of 9 coupled with the CPT-based undrained shear strengths tends to underpredict the fully mobilised end resistance in these San Francisco Bay Area clays. This is consistent with the authors' experience of San Francisco Bay area soils of performing better during pile axial loading tests than expected based on prediction using the LCPC Method.
5. For these and other projects, pile loading tests can provide value to engineers and owners provided they are designed to fully mobilise pile resistance. The results of these high-quality load tests advance understanding of local conditions and provide value to deep foundation projects.
6. The data represented in Fig 2 is limited and it is possible that a greater population of data may show trends that were not evident in this study.

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