The Increase in Capacity with Time for Heavily Overconsolidated Clays

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ABSTRACT

Results are presented for the restarting of 48-in. (1.22-m)-diameter open-ended pipe piles driven in heavily overconsolidated clays for the Ruby Platform off the coast of Trinidad and Tobago. The piles were restarted after delays of about 1, 5, 13 and 15 days at a penetration of about 78 ft (24 m). The soil resistance to driving was computed using the Case-Goble bearing capacity formulation. The total skin friction and total end bearing were determined versus pile penetration. The CAse Pile Wave Analysis Program (CAPWAP) was used to determine the distribution of the soil resistance along the length of the pile and at the pile toe.

Keywords: overconsolidated clay , pile monitoring, soil sensitivity

1 INTRODUCTION

The increase in the capacity of piles driven in clays is well documented. During continuous driving, the clay surrounding the pile is remolded and large excess pore water pressures are generated. Because the excess pore pressures decrease rapidly with radial distance from the pile, water will begin to flow laterally out of the disturbed zone and the clay will consolidate. As pore pressures dissipate, pile capacity increases. Field measurements have shown that the time required for driven piles to regain ultimate capacity can be relatively long.

According to Skempton and Northey (1953), the sensitivity is close to 1.0 for heavily overconsolidated clays, with water contents close to the plastic limit as encountered in the soil boring. Karlsrud et al. (2014) verified significant ageing effects in sand and clay by performing pile load tests. Six 16- to 20-in. (400- to 500-mm)-diameter 66-ft (20-m)-long test piles were driven at six sites, and loaded to failure 1, 3, 6, 12 and 24 months after driving. The four clay sites consisted of low and medium plasticity normally consolidated clays, and low and highly plastic overconsolidated clays. The shaft resistance increased the most in low plasticity normally consolidated clays and increased the least in highly plastic overconsolidated clays.

2 SOIL STRATIGRAPHY

The Ruby Platform is about 27 miles (43 km) off the northeast coast of Trinidad. The geology is complex, with intense deformation over the past 30 million years as the Caribbean Plate converged on the South American Plate. The soil stratigraphy is based on our observations during drilling of a single deep boring, classification of soil samples recovered from the soil boring, in situ testing, and laboratory test results. A summary of the major soil strata at the boring location is tabulated below.

Table 1. Soil Stratigraphy.

Stratum	From (ft)	To (ft)	Description
I	0.0	75.5	Very stiff to very hard clay
II	75.5	266.4	Very hard clay

3 UNDRAINED SHEAR STRENGTH

The undrained shear strength of the clay soils in the soil boring was evaluated using published correlations by Ladd and Foott (1974), Quirós et al. (2000), and Casey and Gemaine (2012), and from Piezocone Penetration Tests (PCPT) using a cone factor proposed by Mayne et al. (2015). Additional details are presented by Stevens et al. (2022). The undisturbed and remolded unconsolidated-undrained (UU) triaxial compression tests are presented in Fig. 1. The soils in the upper 5 ft (1.5 m) have significantly lower strengths than the underlying soils, ranging between about 1.7 and 3.8 ksf 80 and 180 kPa). Below about 5 ft (1.5 m), the shear strength increases from about 5.2 ksf (250 kPa) at 11.5 ft (3.5 m) to about 21.9 ksf (1050 kPa) at 160 ft (49 m).

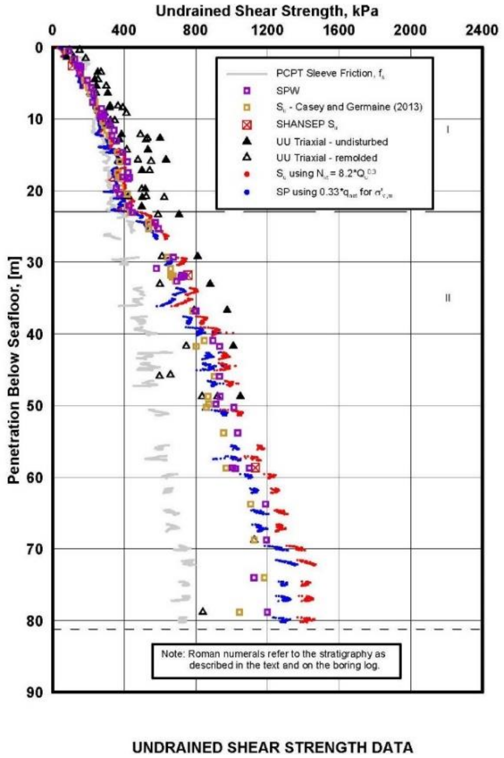

Fig. 1-Undrained Shear Strength Data

4 OVERCONSOLIDATION RATIO

Fig 2 presents the overconsolidation ratio (OCR) computed using the Casagrande (1936) and Oikawa (1987) methods, as well as values computed from the PCPT data using procedures proposed by Mayne (2007). The clay is heavily overconsolidated, with an OCR greater than 60 near the seafloor, decreasing to about 15 near the bottom of Stratum I, and generally decreasing from about 20 at the top of Stratum II to 11 at the bottom of the boring.

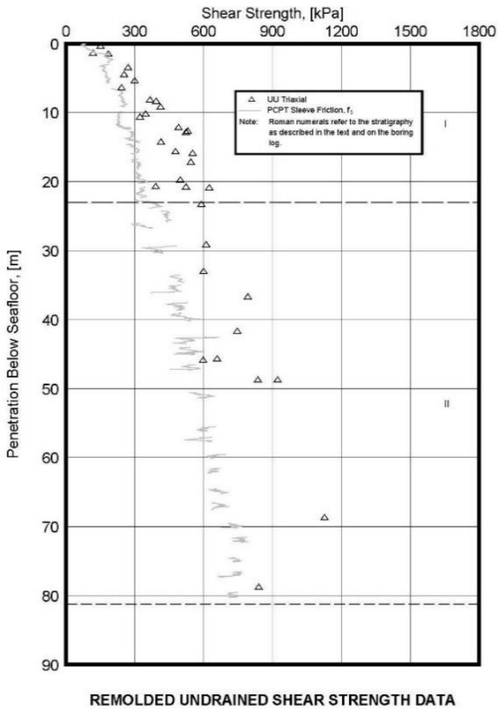

Fig. 2- Remolded Undrained Shear Strength Data

5 REMOLDED SHEAR STRENGTH

Results of the remolded UU triaxial compression tests are plotted in Fig. 3, together with the PCPT sleeve friction, which have been observed to approximate the remolded shear strength. The measured remolded UU shear strength data are about 1.0 to 2.0 times higher than the PCPT sleeve friction in Stratum I and about 1.1 to 1.6 times higher in Stratum II.

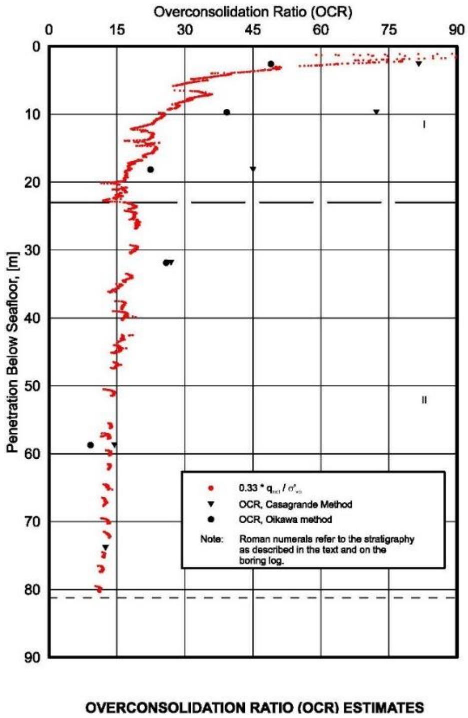

Fig. 3- Overconsolidation Ratio (OCR) Estimates

6 SOIL SENSITIVITY

The soil sensitivity was determined from the ratio of the undisturbed to the remolded shear strength from UU triaxial compression tests performed at the same water content, and from the cone sleeve friction ratio using an equation proposed by Schmertmann (1978). As shown in Fig. 4, the soil sensitivity was close to 1.0 in Stratum I, and between 1.1 and 1.4 in Stratum II.

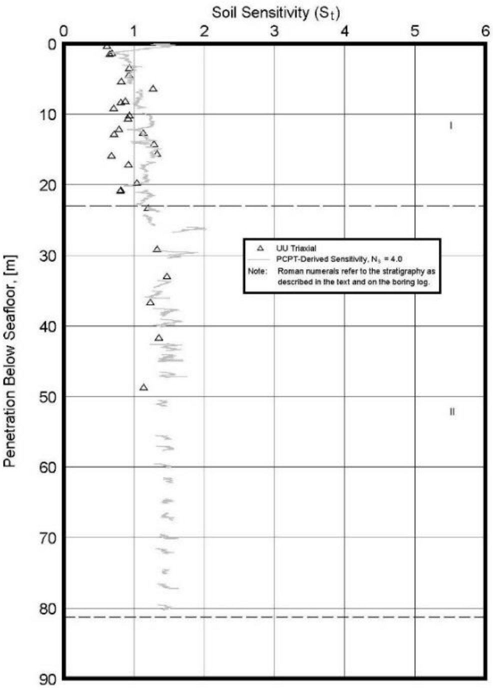

Fig. 4- Soil Sensitivity Data

7 THIXOTROPY

Thixotropy is a time-dependent reversible process. Thixotropic soils under constant moisture and density soften when remolded and the loss of strength is gradually regained with time when the soils are allowed to rest. For normally to lightly consolidated clays, the thixotropic characteristics are usually evaluated by miniature vane tests but for these heavily overconsolidated clays a series of UU triaxial compression tests were performed. The UU shear strengths over a 60-day period are plotted in Fig. 5 along with the measured water content.

The results in Fig. 5 indicate that the UU shear strength between 15 and 60 days is about 85 percent to 89 percent of the strength immediately after remolding. Since the sensitivity of the Stratum I soil is close to 1.0, these results are not unexpected.

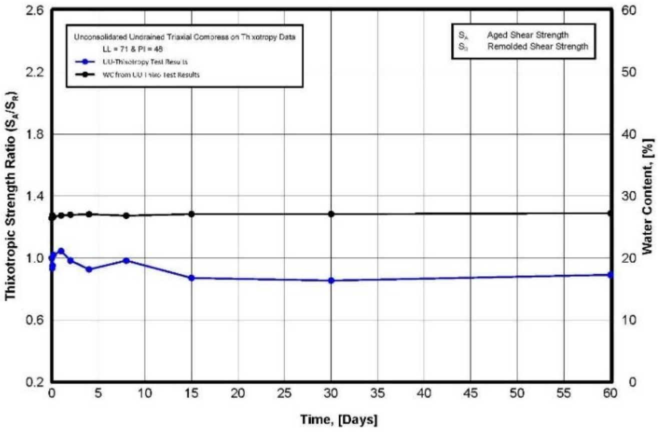

Fig. 5- UU-Thixotropic Shear Strength Ratio

8 DRIVING SEQUENCE

The driving sequence is presented in Table 2, with installation dates and driving times for the P2 and P3 sections of each jacket pile. The time from the end of driving the P2 section to the start of driving the P3 section determines the delay. Delays greater than 24 hours were due to weather delays. Blow counts and hammer energies for the last five feet of driving (or the first five feet of redriving) are presented in Table 3.

9 COMPUTED QUANTITES

Strain transducers and accelerometers were bolted on opposite sides of the pile. The elastic modulus of the pile was used to obtain force-time histories, and the acceleration was integrated to obtain velocity-time histories. The soil resistance to driving was computed using the Case Method with a damping coefficient of 0.5, appropriate for clay. To help interpret these results, the total skin friction SFT and the total computed force at the bottom of the pile CFB were computed. Total means that these values were computed assuming zero damping.

Although there was no difficulty restarting the piles after delays of 312 and 365 hours for Piles A1 and B2, respectively, the average soil resistance (RX5) for ten blows before and after the delay required to make the last add-on increased from 2653 to 5066 kips (11.8 to 22.5 MN) for Pile A-1, and from 2885 to 5880 kips (12.8 to 26.2 MN) for Pile B2. For Pile B1, the delay was 121 hours, and RX5 increased from 2862 to 5093 kips (12.7 to 22.7 MN). For Pile A2, the delay was only 23 hours, but RX5 increased from 2777 to 4845 kips (12.4 to 21.6 MN). Stevens (2004) showed that the soil resistance typically decreases with every hammer blow applied, but for these piles, the soil resistance is constant for the first ten blows during the redriving.

When the last add-on is made, SFT and CFB generally increase. The exception is Pile A2. SFT does not increase after the short delay of 23 hours. CFB does increase. Another observation is that SFT is constant or decreasing below 125-ft (38-m) penetration, and CFB is constant or increasing below 125-ft (38-m) penetration. SFT and CFB are plotted versus penetration for Piles A1, A2, B1 and B2 in Figs. 6 through 9, respectively.

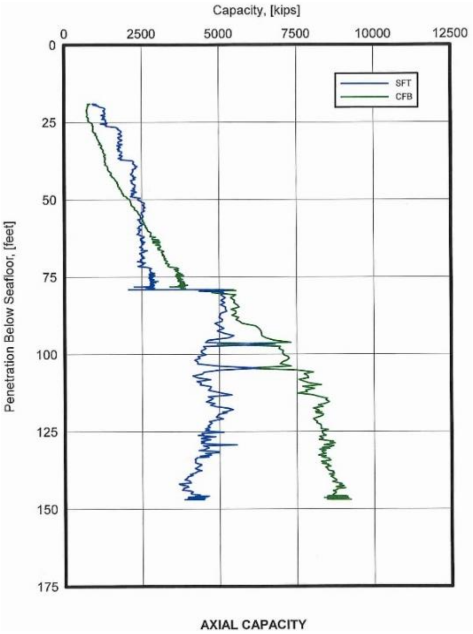

Fig. 6 Axial Capacity

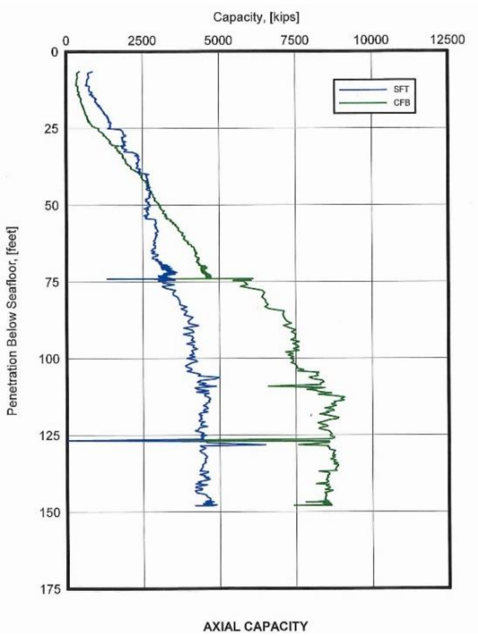

Fig. 7 Axial Capacity

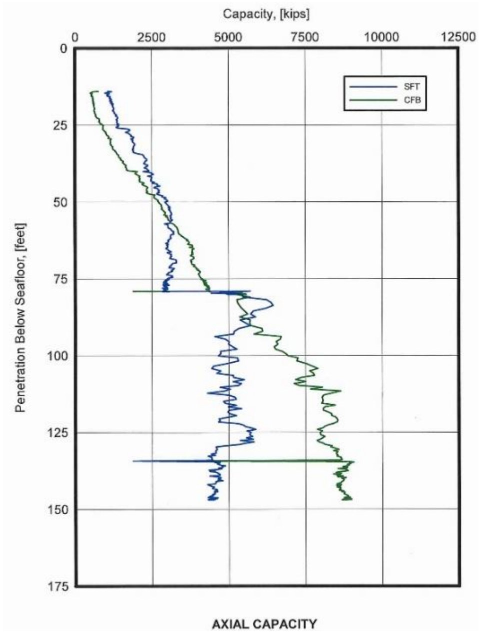

Fig. 8 Axial Capacity

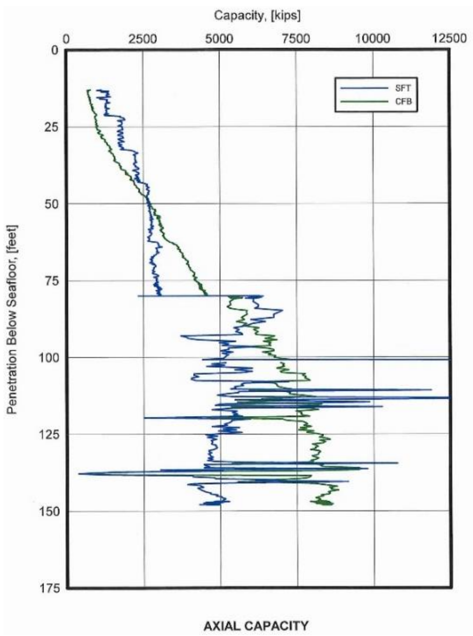

Fig. 9 Axial Capacity

10 CAPWAP ANALYSES

The computer program CAse Pile Wave Analysis Program (CAPWAP) was used to estimate soil quake and damping parameters, and the distribution of the soil resistance to driving along the length and at the toe of the pile. The pile is divided into continuous segments and calculations are made using a traveling wave algorithm. Either the measured pile top force or velocity is used as a boundary condition, and the complementary quantity is computed and compared to the measured curve. The set of soil parameters is varied until a best match between the measured and computed pile top force or velocity is obtained.

The skin friction distribution is plotted versus penetration in Fig. 10 for the average of the three deep piles at the end of initial driving (EID) and individually for each pile at the beginning of redriving (BoR). The shaft resistance does not increase for Pile A2 with the shortest delay of 23 hours. The shaft resistance does increase for the other piles, from 43 percent for Pile B1 (121 hours), to 54 percent for Pile A1 (312 hours), and 67 percent for Pile B2 (365 hours).

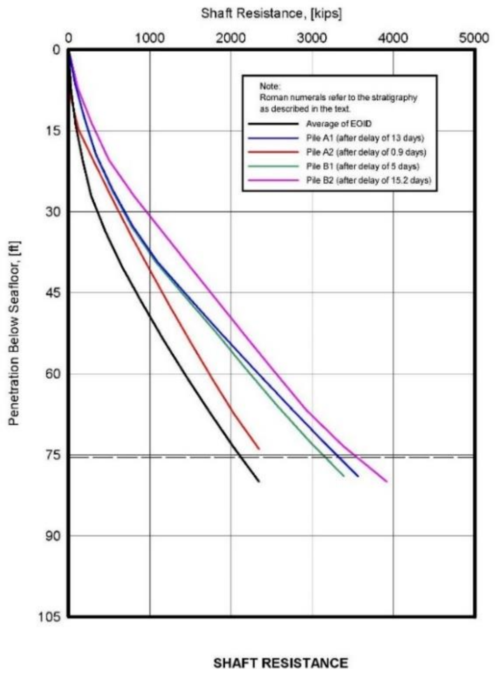

Fig. 10- Shaft Resistance

11 SUMMARY

- The undisturbed shear strength increases from about 5.2 ksf (250 kPa) at 11.5-ft (3.5 m) penetration to about 21.9 ksf (1050 kPa) at 65-ft 49 m)penetration.
- The soils are heavily overconsolidated, with an OCR of greater than 60 near the seafloor decreasing to about 15 near the bottom of Stratum I, and generally decreasing from about 20 at the top of Stratum II to 11 at the bottom of the boring.
- The laboratory measured soil sensitivity data are generally close to 1.0 within Stratum I and between about 1.1 and 1.4 in Stratum II.
- Thixotrophy testing show that the UU strength between 15 to 60 days is about 85 to 89 percent of the strength immediately after remolding.

12 CONCLUSIONS

- Although there was no difficulty restarting the piles after delays of 312 and 365 hours for Pile A1 and B2, respectively, the average soil resistance to driving (RX5) increased about 100 percent for these piles, and about 75 percent for the other piles.
- The average soil resistance (RX5) for 10 blows before and 10 blows after the delay required to make the last add-on increased from 2653 to 5066 kips (11.8 to 22.5 MN) for Pile A1, and from 2885 to 5880 kips (12.8 to 26.2 MN) for Pile B2. For Pile B1, the delay was 121 hours, and RX5 increased from 2862 to 5093 kips (12.7 to 22.7 MN). For Pile A2, the delay was only 23 hours, but RX5 increased from 2777 to 4845 kips (12.4 to 21.6 MN).
- When the last add-on was made, SFT and CFB generally increased. The exception is Pile A2. SFT does not increase. Another observation is that SFT is constant or decreasing below 125-ft penetration, and CFB is constant or increasing below 125-ft penetration.
- The shaft resistance computed in the CAPWAP analyses does not increase for Pile A2 with the shortest delay. The shaft resistance does increase for the other piles, from 43 percent for Pile B1 (121 hours), to 54 percent for Pile A1 (312 hours), and 67 percent for Pile B2 (365 hours).

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Table 2: Driving Sequence

Pile	Pile Section	Date	Driving Time		Penetration (ft)		Delay (hours)
			From	To	From	To	
A1	P2	9/30/2020	14:26	15:33	19	79	
	P3	10/13/2020	15:23	21:56	79	147	311.83
A2	P2	10/12/2020	03:35	04:39	6	74	
	P3	10/13/2020	3:16	11:44	74	148	22.62
B1	P2	10/9/2020	18:34	19:57	14	79	
	P3	10/15/2020	12:49	19:46	79	148	120.87
B2	P2	9/30/2020	15:52	17:10	13	80	
	P3	10/15-16/2020	22:10	05:42	80	148	365.00

Table 3: Blow Counts and Hammer Energies

Pile	Pile Section	Blow Counts (bpf)	Hammer Energy (kJ)
A1	P2	36, 36, 39, 37 and 47 to 79 ft	251, 253, 252, 252 and 235
	P3	145, 131, 32, 71 and 71 from 79 ft	306, 364, 398, 398 and 398
A2	P2	49, 48, 53, 48 and 40 to 74 ft	249, 252, 248, 252 and 244
	P3	117, 82, 90, 70 and 67 from 74 ft	313, 319, 319, 382 and 398
B1	P2	42, 46, 43, 49 and 50 to 79 ft	254, 249, 249, 241 and 248
	P3	89, 58, 59, 54 and 49 from 79 ft	328, 437, 475, 486 and 482
B2	P2	45, 49, 64, 40 and 49 to 80 ft	249, 251, 250, 250 and 250
	P3	157, 53, 45, 85 and 148 from 80 ft	395, 399, 403, 402 and 476