Abridgment

Side Shear on Piles Driven in Oversized Pilot Holes

Michael W. O'Neill, Department of Civil Engineering, University of Houston
M. Frederick Conlin, Jr., National Soil Services, Inc., Houston

The stratigraphy at the site of the new Texas Viaduct, in Texarkana, is shown in Figure 1. The variable conditions near the surface required the use of piles to support the structure, but the gravel and sandstone layers and the random zones of hard clay below did not permit driving the majority of the piles to grade without structural damage; therefore, pilot holes were used. These holes were augered with a lightweight bentonite-water slurry to maintain open boreholes through the waterbearing soils. Oversized holes were used to minimize the entrapment of slurry or cuttings beneath the pile tips, at which point the majority of load transfer to the soil was assumed to occur. It was anticipated that the slurry would escape in the spaces between the sides of the square piles and the oversized holes, and that there would be a small contact area between a pile and the natural soils where the corners of the pile extended beyond the diameter of the pilot hole.

The majority of the piles were 40.6-cm (16-in) square piles, typically penetrating about 12 m (40 ft) below grade, with design loads of about 310 kN (35 tons) based on only end bearing. While it was assumed that some side shear (skin friction) would develop against the piles, there was no rational basis for assigning values of it because of the method of installation. Therefore, in order to measure the actual side shear developed and to provide some insight into the mechanism controlling side shear in piles driven in oversized pilot holes, each of three concrete test piles was instrumented in the casting bed with full-bridge electronic-resistance, embedment strain-gauge circuits (transducers) located near its top and bottom (Figure 1).

By translating the auger in the finished pilot holes, the mean diameter of the holes was estimated to be 48 cm (19 in) [7.6 cm (3 in) greater than the width of the piles]. Each pilot hole was terminated about 48 cm (19 in) above the ultimate tip elevation, the auger withdrawn, the pile inserted, and the pile driven to grade. Each pile dropped to within 1.5 to 3.0 m (5 to 10 ft) of the bottom of the pilot hole under its own weight plus that of the hammer and appurtenances. Thereafter, about 125 blows of the hammer were applied to seat the piles. Each pile was allowed to stand without load for at least 1 week prior to testing to allow possible freezing to occur.

Two tests were conducted on each pile: (a) a maintained-load proof test to twice the design load and (b) a rapid-load test to approximately 3.5 times the design load. The proof test consisted of application of load in increments of half the design load every 6 h, maintenance of twice the design load for 30 h, and rapid rebound. Each rapid test was conducted about 1 h after the pile was rebounded from the proof test by applying small increments of load every 2.5 min until failure in side shear occurred. Load versus settlement diagrams for all of the load tests are given in Figure 2.

The mean pile movement was calculated for each butt settlement reading by subtracting half of the computed elastic compression of the pile from the measured butt settlement. The mean movement was then plotted against the measured mean side shear (load transfer) developed for each increment of load. The load-transfer relations for the rapid-load tests are shown in Figure 3. Relations developed from the subfailure proof tests followed the same trends as those shown in Figure 3.

The behavior of the piles can be analyzed on the basis of the type of soil into which they were driven. The only pile driven into a clearly cohesionless soil profile was TV3, which is expected to develop ultimate side shear according to the following equation:

$$f_u = \bar{\sigma}_v k \tan\delta \tag{1}$$

where

- f_u = mean unit ultimate side shear,
- $\bar{\sigma}_v$ = average vertical effective stress acting between the ground surface and the lower transducer,
- k = coefficient of lateral earth pressure, and
- δ = angle of friction between the pile wall and the soil.

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Figure 1. Soil and test pile profiles.

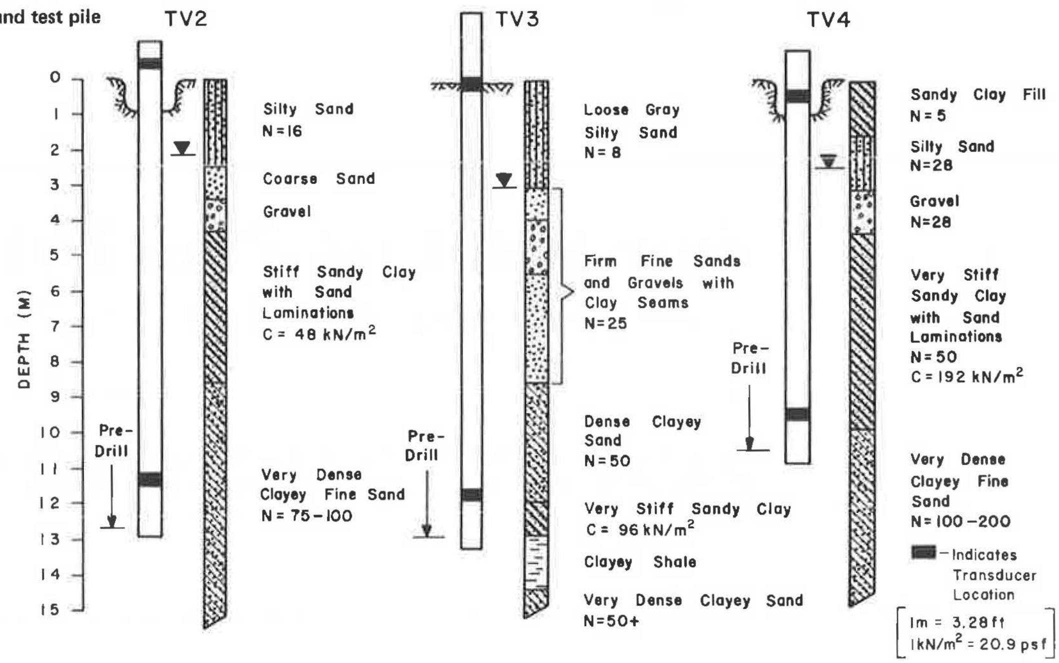


Figure 2. Load versus settlement relations.

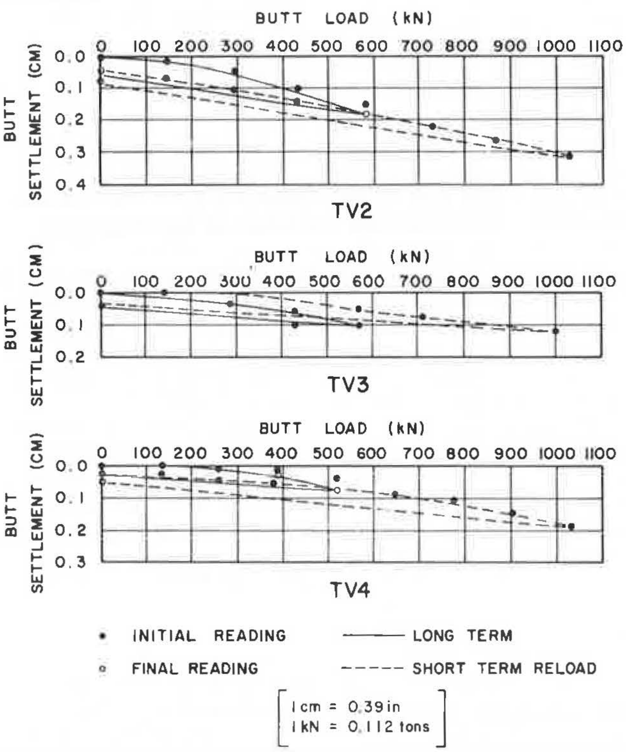
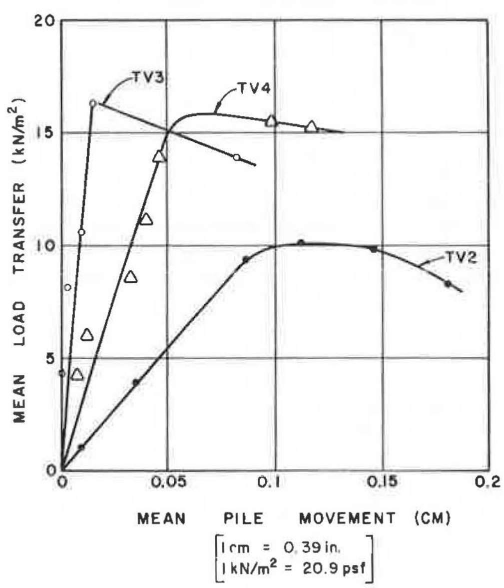


Figure 3. Mean unit load transfer relations.



By standard penetration testing, the mean angle of internal friction (ϕ) of the soil from the surface to the lower transducer level was estimated to be 37 deg, and $\bar{\sigma}_v$ was estimated to be 74 kPa (1545 lb/ft²). From Figure 3 it is seen that f_u = 16.3 kPa (340 lb/ft²), which is approximately 49 percent of the typical value observed locally in an undersized pilot hole. Thus, k tan δ can be computed from equation 1 to be 0.22.

There is no direct means of separating the factors k and tan δ. However, experiments have shown that δ/ϕ for smooth concrete against cohesionless soils varies from 0.86 to 1.00. If a reasonable value of 0.90 is chosen, δ is 33 deg, and k is 0.34. The Rankine active earth-pressure coefficient for a soil with a ϕ of 37 deg is 0.25. The coefficient of earth pressure at rest is approximately (1 - sin ϕ) or 0.40. Therefore the lateral earth pressure for TV3 appears to be somewhat lower than that in an at-rest state but higher than that in a fully active state.

The mean ultimate skin friction for TV3 can thus be approximated by the expression

$$f_u = (0.94 - \sin\phi)\bar{\sigma}_v \tan 0.9\phi \tag{2}$$

The validity of this equation cannot be assumed at depths greater than about 25 equivalent pile diameters. Displacements in the order of 0.02 cm (0.01 in) were sufficient to mobilize f_u.

Pile TV4 was driven into a predominantly clay profile. Pile TV2 was driven into a stratified clay and sand profile in which a significant portion of the sand was overlain by medium stiff clay. During installation, the soil from this clay stratum was most likely forced by the pile down into the lower sand stratum for a significant distance so that remolded clay was present in a smear zone between the pile and the sand below the clay stratum. Thus, the remolded clay rather than the existing sand provided much of the side shear below the clay stratum. Therefore, piles TV2 and TV4 were analyzed on the assumption that all of the side shear was provided by clay in undrained shear.

When a pile is driven without a pilot hole into soft to stiff clays, the soil around the pile is fully remolded immediately after the pile is driven, but the high lateral stress created by the driving promotes consolidation of the remolded soil so that, with time, the original shear strength (c_u) is essentially regained and approximately equals the ultimate side shear.

When a pile is driven into an oversized pilot hole in clay, however, the partial annular gap between the pile and the soil restricts the development of lateral stresses around the pile to an amount that may be insufficient to affect consolidation. Thus, the ultimate side shear developed in such a case can be approximated by

$$f_u = (c_u/S_t)A_{cr} \tag{3}$$

where

c_u = undrained cohesion of the undisturbed clay,
S_t = sensitivity of the clay, and
A_{cr} = ratio of the idealized lateral contact area between the soil and the periphery of the pile to the gross peripheral area of the pile.

The sensitivity of the soil at the test locations was approximately 1.55 and the idealized contact area ratio for a 40.6-cm (16-in) square pile in a 48-cm (19-in) diameter pilot hole is 0.33. The undrained cohesion of the undisturbed clay was 48 and 192 kPa (1000 and 4000 lb/ft^2) at the locations of TV2 and TV4 respectively. The values of mean ultimate skin friction computed with equation 3 are 10 kPa (210 lb/ft^2) at TV2 and 40.7 kPa (850 lb/ft^2) at TV4.

Thus, equation 3 predicted f_u quite accurately for TV2, but overestimated f_u by a factor of about 2.5 for TV4. Such behavior is consistent with the corresponding comparisons of computed and observed developed side shear in piles driven without pilot holes into medium stiff to very stiff clays when the original shear strength is assumed to equal the ultimate skin friction. Above a certain value of c_u, typically 48 to 72 kPa (1000 to 1500 lb/ft^2), the full undrained cohesion is not converted into pile skin friction. This phenomenon may be due to the fact that the relatively minor lateral movements that occur during driving tend to force the soil away from the pile permanently unless the overburden stresses are high enough to cause it to flow back against the sides of the pile.

Because of this phenomenon for piles driven without pilot holes and the test results reported, it is reasonable to modify equation 3 as follows:

$$f_u = (c_u/S_t)A_{cr} \qquad [c_u \leqslant 72\ \text{kPa}^2(1500\ \text{lb/ft}^2)] \tag{4a}$$

and

$$f_u = (72\ \text{kPa}^2/S_t)A_{cr} \qquad [c_u \geqslant 72\ \text{kPa}^2(1500\ \text{lb/ft}^2)] \tag{4b}$$

in which the value of the computed mean ultimate skin friction remains 10 kPa (210 lb/ft^2) for TV2, but is reduced to 15 kPa (315 lb/ft^2) for TV4. The values computed from equations 4a and 4b are 20 to 25 percent of the values observed locally in undersized pilot holes.

The mean displacement required to develop full skin friction for piles TV2 and TV4 was somewhat less than 0.125 cm (0.05 in). This is on the order of one-quarter of the value observed for piles driven without pilot holes. The lower displacement may be related to the reduced lateral pressure and consequent narrow zone of shear straining adjacent to the pile.