A REFINERY EXPANSION PROJECT WITH PILES INSTALLED IN SETTING SOIL – A CASE STUDY.

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Abstract

A large refinery expansion was undertaken at a site reclaimed from a lake about 40 years ago (Midwest of United States). The heavily-loaded and movement-sensitive structures were to be supported on auger-cast-in-place piles. Anticipating that the piles would be subjected to negative skin friction (downdrag and drag force) from placing about 1.5-m (5 ft) of new areal fill, the original design was to tip the piles into rock located at depths of about 27 to 31 m (88 to 101 ft). This recommendation was based on “capacity” and did not recognize that the issue is not a pile “capacity” problem but downdrag (pile movement) caused by external factors such as fill placement. Tolunay-Wong Engineers, Inc. was retained to review the original pile recommendations, and to assess shortening of the piles. The results of instrumented bi-directional (O-cell) tests, high-strain dynamic testing and the principles of Fellenius’ Unified Design Method for downdrag, drag force, settlement and capacity for single piles and small (narrow) pile groups were used to demonstrate that the piles only needed to penetrate 1.5 m (5 ft) into the upper glacial till layer to control downdrag. The resulting shorter piles also had adequate “capacity” for the applied design load. The revised pile design resulted in a safe foundation, significant cost savings and faster construction time (from shortening the piles).

1. Introduction

A large refinery expansion was undertaken at a site reclaimed from a lake about 40 years ago (Midwest of United States). The project included heavily-loaded and movement-sensitive structures. Most of the proposed structures were to be supported on 457-mm (18-in.) diameter auger-cast-in-place piles (ACIPs). Typical unfactored dead (sustained) and live loads per pile were 1,300 kN (292 kips) and 300 kN (67 kips), respectively. The piles were to be installed prior to fill placement, with pile cut off at about existing grade. Compressive strength of the grout was 34,473 kPa (5,000 psi) at 28 days.

Anticipating that the piles would be subjected to negative skin friction (downdrag and drag force) from placing about 1.5-m (5 ft) of new areal fill, the original design was to tip the piles into rock located at depths of about 27 to 31 m (88 to 101 ft). Calculated settlement from the new fill was about 55 mm (2.2 in.).

The above recommendation was based on “capacity”, a (old) method where the induced drag force is subtracted from the pile bearing “capacity”. Notice that the analysis of piles installed in settling soil has evolved to recognize that the issue is not a “capacity” problem but “downdrag” (pile movement) caused by external factors [such as fill placement (this case), dewatering, post-liquefaction settlement, etc.].

2. Objectives

Tolunay-Wong Engineers, Inc. (TWEI) was retained to review the original pile recommendation by others, and to assess shortening of the piles. Dr. Bengt H. Fellenius provided valuable technical support and advice to TWEI.

3. Soil Stratigraphy

The soil stratigraphy consisted of about 10.3 m (34 ft) of loose to dense sands, followed by about 14 m (46 ft) of compressible clays. Below the compressible clay was a hard glacial till about 7-m 23.1-ft) thick in Area B and 2.9-m (9.6-ft) thick in Area G, deposited on top of rock (limestone).

4. Pile Testing

The field investigation for final design included instrumented bi-directional (BD) tests (O-Cell), high-strain (HS) dynamic testing, lateral pile load tests, as well as new soil borings and cone penetration tests (CPTs). The pile testing program was performed at two areas (Areas "B" and "G") within the project layout. Three companion piles were installed per area: one pile for the static BD load test, one pile for the HS dynamic testing, and one pile for the static lateral load test (not covered in this document):

Area B: Pile B1 (BD static load test)
Pile B2 (HS dynamic testing)
Pile B3 (Lateral static load test)

Area G: Pile G1 (BD static load test)
Pile G2 (HS dynamic testing)
Pile G3 (Lateral static load test)

LoadTest performed the BD tests; GRL performed the HS dynamic tests. Dr. Fellenius also looked into the test data for the purpose of the engineering recommendations.

The piles for the BD tests were instrumented with five vibrating wire strain gages (one strain gage placed below the O-cell, and four strain gages above the O-Cell). For both O-cell piles, the O-cell locations were at about the interface between the compressible clay and the hard glacial till. The O-Cell strain gages were attached to a 10HP42 steel beam which was inserted into the pile after the completion of grouting and removal of the auger. Pile instrumentation also included two pairs of telltales. The O-Cell tests were performed in accordance with ASTM D1143 ("Quick" load test method).

3.1 - BD (O-Cell) Test Results

The results of the tests from Piles B1 and G1 are presented in Figure 1. The load tests were performed following ASTM D1143 Standard ("Quick" Method). The points shown correspond to pile displacement after holding the load for a duration of 8 minutes.

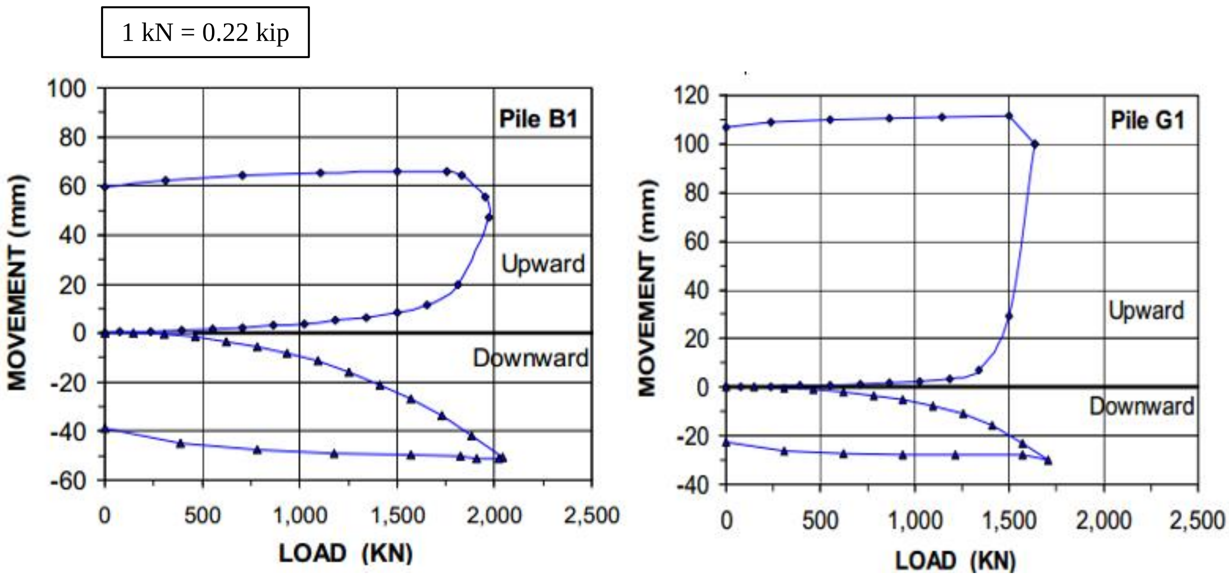


Figure 1. Load-Movement Curves from BD Tests – Piles B1 and G1

It is customary to combine the measured upward and downward movements into an equivalent head-down pile settlement curve. The simulation (trial-and-error) consisted on matching the upward and downward curves from the O-Cell load tests assuming load transfer t-z (upward movement) and q-z (downward movement) functions, and using the software UniPile. The combined maximum simulated loads in the tests were 4,010 kN (901 kips) at 72 mm (2.8 in.) pile head movement and 3,350 kN (753 kips) at 47 mm (2.2 in.) pile head movement for piles B1 and G2, respectively.

The recorded strain changes are converted to load by multiplying strain (ε), area (A), and "elastic" modulus (E). While the steel area is well defined, the concrete area due to unavoidable variation of the diameter of drilled piles is not. Pile grout volume per five-foot increment was recorded by a PIR (Pile Installation Recorder) system during pile installation. However, the largest uncertainty is with the modulus, which not only can vary between concrete or grout compositions, it is also not a constant but a variable due to changes with stress level. This can be overcome by applying the "tangent modulus" method (Fellenius 1989, 2001) in which the change of stress over change of strain is plotted versus strain. Combined elastic modulus values of 29 GPa (4,206 ksi) and 30 GPa (4,351 ksi) were computed for Piles B1 and G1, respectively, using the tangent method, .

The measured load distribution with depth and equivalent head-down load distribution for the maximum cell loads on Piles B1 and G1 are presented in Figure 2. From this information

Figure 3 was developed to back-calculate the beta-coefficients (effective stress "proportionality" coefficient) that will later be used to compute the long-term response of the pile.

1 kN = 0.22 kip; 1 m = 3.3 ft

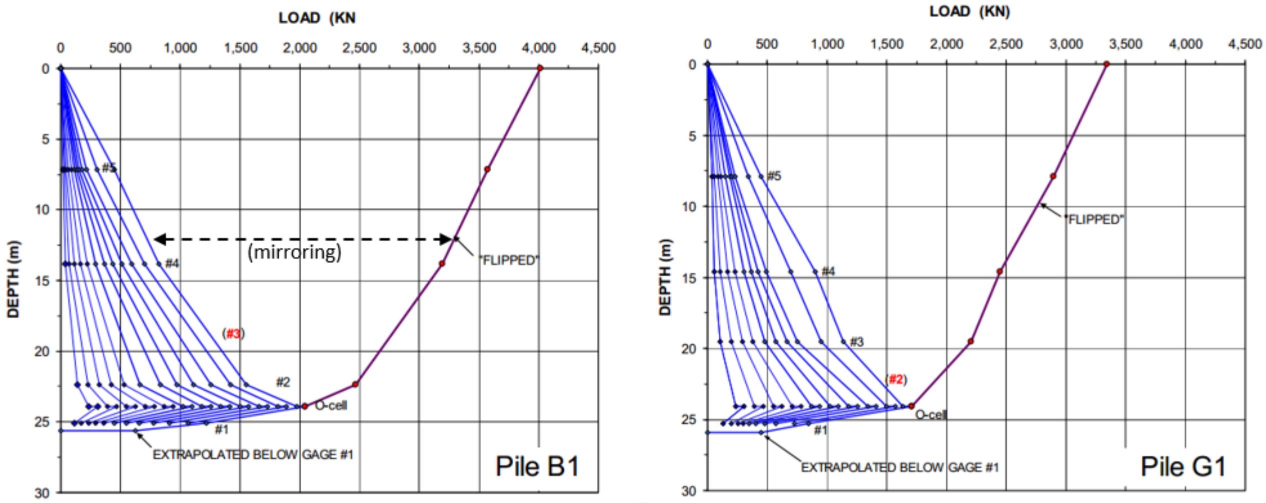
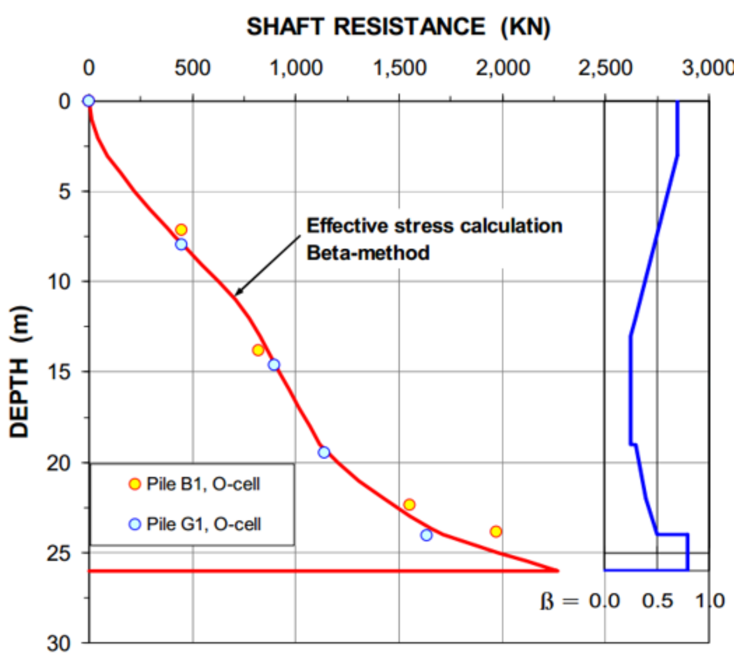


Figure 2. Load-Distribution at Gage Levels from BD Tests and Equivalent Head-Down Load Distribution for Maximum Cell Load



$r_s = ß\,\sigma'_0$

r_s = Ultimate unit shaft resistance

ß = Beta Coefficient ("effective-stress proportionality coefficient")

σ'_0 = Effective overburden stress

1 kN = 0.22 kip; 1 m = 3.3 ft

Figure 3. Back-Calculated Beta Coefficients from Measured BD Load Test Shaft Resistance Values

3.2 High-Strain Dynamic Test Results

High-strain dynamic testing was performed in companion Piles B2 and G2 by dropping a 130-kN (30-kip) ram weight from different heights. The piles were instrumented with accelerometers and strain gages located near the pile head. The results indicated that neither shaft resistance nor toe resistance was fully mobilized in the dynamic test on pile B2 because it was difficult to maintain concentric blows (which generated uneven stresses and resulting bending at the four strain gages attached to the pile) despite major grinding of the pile head. This limited the height-of-fall of the ram to no more than 450 mm (18 in.) [versus 620 mm (25 in.) and 920 mm (36 in.) in Pile G2].

Better results were obtained in the dynamic test in Pile G2. The results from Pile G2 measured ultimate pile resistance of 3,140 kN (706 kips) for blow #3 and 3,800 kN (854 kips) for blow #4, respectively, with an average value of 3,470 kN (763 kips). This average value is in reasonable agreement with the result of the BD test performed on Pile G1 which resulted in an ultimate pile resistance of 3,350 kN (737 kips).

5.Method of Assessment the Pile Response Under Soil Settling Conditions

The principles of the Unified Design Method (UDM) (Fellenius 1984, 1988, 2019) for single piles and small (narrow) pile groups were followed to assess the response of the piles under settling soil. The three principles of the UDM are presented below.

1. The settlement (differential and total) of a piled foundation must be smaller than the maximum acceptable (or permissible) for the supported structure.
2. The sum of sustained load and drag force must be smaller than the axial structural strength of the pile with an acceptable margin (factor of safety). Drag force is only of importance to the structural strength of the pile.
3. The sum of sustained (dead) and transient (live) loads must be smaller than the pile axial "capacity" with an acceptable margin (factor of safety).

Notice that many structures on piled foundations have failed even though the design incorporated a factor of safety on "capacity" larger than 2 and 3. No structure supported on piled foundation designed for acceptable settlement has ever failed (Fellenius, 2019, personal communication).

The Unified Design Method is accepted by several standards and codes around world; more recently by the US Corps of Engineers. The UDM concept has also been validated by finite-element method (FEM) model studies of piles installed in settling soil (Tan and Fellenius, 2016).

Using the computed ultimate shaft resistance and back-calculated beta-coefficients from Figure 3, and principles of the UDM, force and settlement equilibrium diagrams were developed as presented in Figure 4 to find the depth where toe resistance – neutral plane equilibrium occurs. Based on Figure 4, the expected pile head settlement is 22 mm (0.86 in.) which is less than 25 mm (1 in.) design requirement (Principle 1 of UDM → OK).

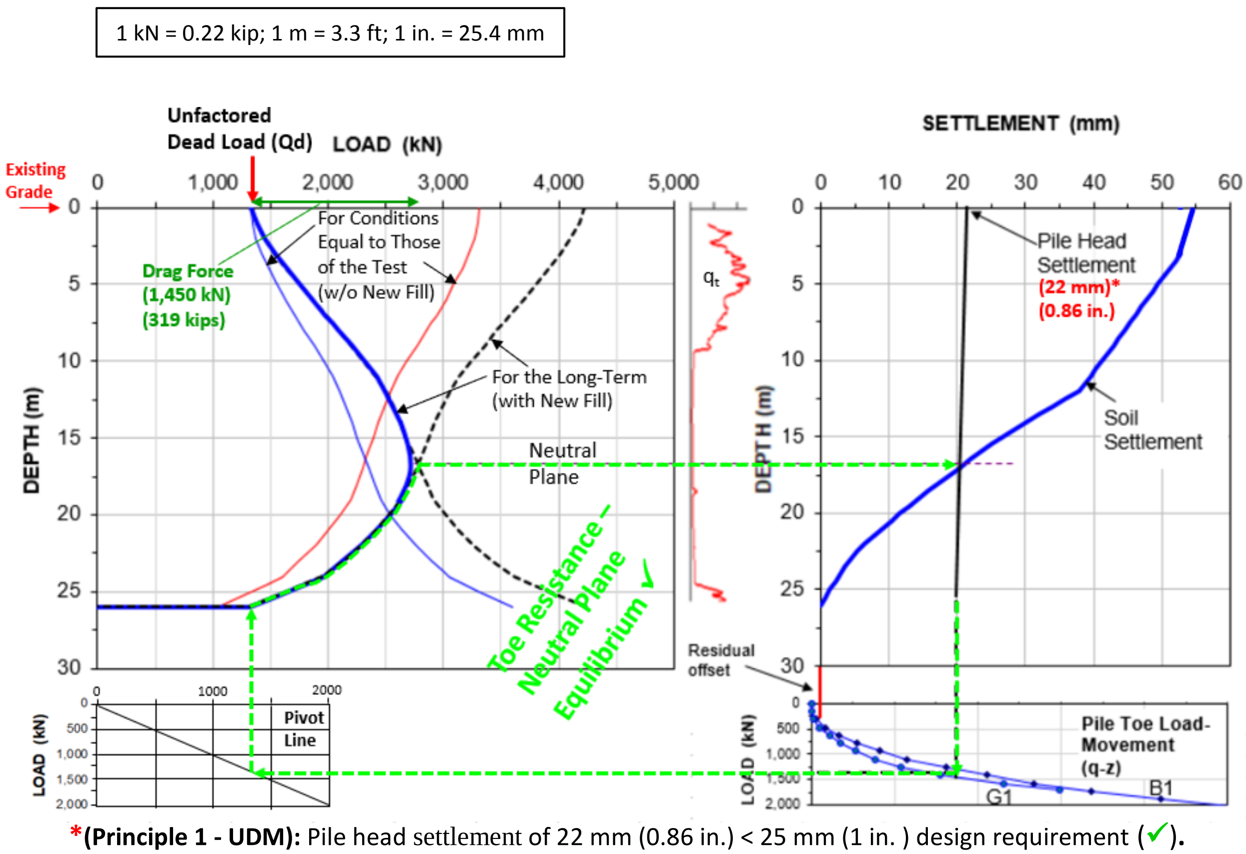


Figure 4. Toe Resistance – Neutral Plane Equilibrium Determination (Fellenius and Ochoa, 2009)

Note that there is only one intersection depth that is simultaneously a force and settlement equilibrium: the neutral plane. A simple trial-and-error procedure will let us find it as shown on Figures 5a and 5b (Fellenius 2016).

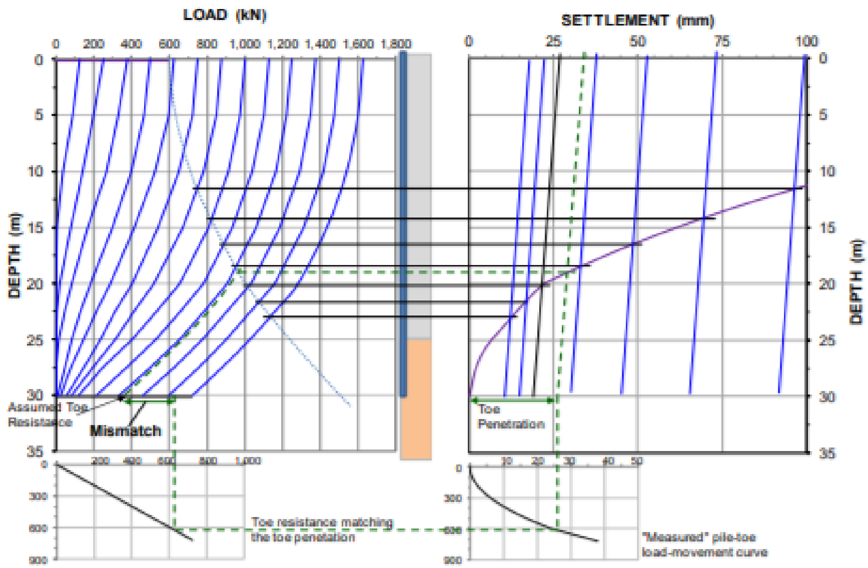


Figure 5a. First Attempt to Find the True Neutral Plane (Fellenius, 2016)

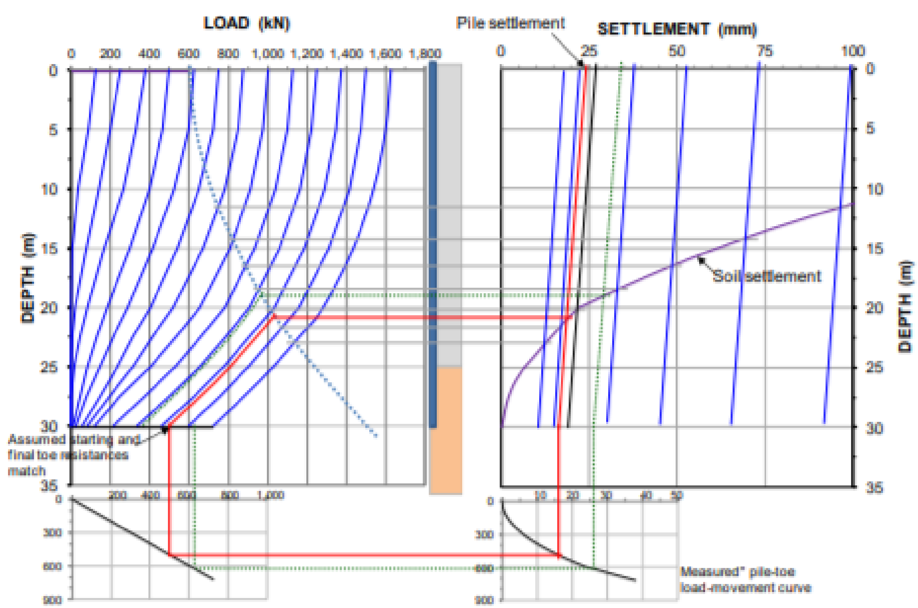


Figure 5b. Final Match Between Starting and Finished Toe Resistance and Determining the Pile Resistance (Fellenius, 2016)

Principle 2 – UDM: From the force-equilibrium diagram the estimated drag force (Qn) is about 1,450 kN (319 kips). Drag force is only of importance for the structural strength of the pile. Drag force should be added to dead load of 1,300 kN (286 kips) at the neutral plane to assess the structural strength of the pile. The Structural Engineer indicated no structural strength for pile under these forces (✓).

Principle 3 – UDM: The computed the factors of safety (FS) in terms of "capacity" can be computed from the following equation: Qu/(Qd + Ql), where Qu is the ultimate pile resistance from the load test, Qd is the unfactored sustained (dead) load, and Ql is the unfactored live (transient - wind, seismic) load.

-Pile B1: FS = 4,010 KN / (1,300 kN + 300 kN)= 2.5 (✓)
-Pie G1: FS = 3,350 kN / (1,300 kN + 300 kN)= 2.1 (✓)

Note that drag force does not come into the equation since at ultimate strength condition drag force is zero (i.e., positive skin resistance from pile head to pile toe).

6.Conclusions

- Performed pile testing provided valuable data used in the assessment of the pile response for piles installed in settling soil (i.e., under negative skin friction).
- The pile analysis was performed using the principles of the Unified Design Method for downdrag, drag force, settlement, and "capacity" for single piles and small (narrow) pile groups.
- The pile response analysis (considering equilibrium between toe resistance and neutral plane) indicated that the auger cast-in-place pile only needed to penetrate 1.5 m (5 ft) into the hard glacial till (i.e., piles did not have to tip into rock) to control downdrag. The resulting shorter piles also had adequate "capacity" for the applied design load.
- Notice that the above results are *site-specific* and resulted in shorter piles from what was originally planned. For other project sites, the results could be different (for instance, the piles may have to be longer to control downdrag, etc.).
- For the presented case study, the engineers (and owner) understood the potential benefit of performing the *instrumented* pile load tests, and approved the additional cost for performing the instrumented pile testing. At the end, the extra expense paid off and resulted in a safe foundation design, several millions of dollars in foundation cost savings, and faster construction time (from shortening the piles).

7.References

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