

## Wide Storage Tanks on Piled Foundations

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**ABSTRACT:** Foundation designs benefit from correlation to well-documented case histories. However, for the design of wide tanks storing liquids, in particular those requiring piled foundations, only few well-documented case histories exist. The authors have found five papers reporting settlement of wide tanks or large groups and have reanalyzed the records. The reanalyses show that a large piled foundation can be modeled as a flexible raft placed at the pile toe level with the foundation load distributed according to Boussinesq stress distribution, and, for large piled foundations, that the capacity of an individual pile is not relevant to the foundation performance. The findings are used to address the analysis of a hypothetical large piled foundation for an 84 m diameter LNG tank at a site with a 60 m thick soil profile, consisting of clay, sand, and clay deposited over competent dense gravel. The differential settlement between the perimeter and interior piles and the effect of drag force and downdrag are discussed. The limitation of drag force as affected by the pile spacing and the weight of the soil in between the interior piles is addressed.

**KEYWORDS:** Wide storage tanks, Piled foundations, Settlement analysis

### 1. INTRODUCTION

Large tanks storing liquids, e.g., Liquid Natural Gas, LNG, typically have diameters ranging from about 60 m through 80 m. The loads are large, often necessitating placing the settlement-sensitive tanks on piled foundations, which then invariably require a very large number of piles; up to and in excess of 1,000 piles. The wide tank diameters cause difference in response between interior and perimeter, as well as between the main pile group and smaller groups of piles located adjacent to the tank to support connecting pipe racks and similar structures.

LNG tanks are often placed in coastal or near-shore areas with soil profiles containing thick layers of compressible soils, where site drainage and site preparation requirements frequently make it necessary to raise the area by placing a fill under and around the tanks. The fill causes the ground to settle, which develops drag force on the piles and downdrag (settlement) for the piled foundation. Depending on pile spacing, the drag force developing for the interior piles may be quite different to that for perimeter piles and piles outside or away from the tank.

Well-documented case histories reporting settlement observations on wide foundations are scarce. Only a handful are available that deal with wide tank foundations and, specifically, include results of settlement measurements across the tank footprint. This paper presents analyses of a few available case histories, verifying the use of conventional analytical methods for design that considers the observed settlements. The design of a typical large piled foundation, such as for an LNG tank, is then discussed in the light of the results of analysis of the case history foundations.

### 2. CALCULATION OF PILE GROUP SETTLEMENT

The settlement response of piles and piled foundations can be separated to three components.

Component 1 is the "immediate" downward movement of the pile cap when load from the supported structure is applied to the pile or piles. It is called "load-transfer movement". It is composed by shortening of the pile due to the axial load, of movement necessary for mobilizing the shaft resistance, and of movement of the pile toe, if the applied load is larger than the ultimate shaft resistance. In the latter case, the shaft resistance movement response ("t-z" function) for the pile element immediately above the pile toe and the load-movement response for the pile toe ("q-z" function) govern the process.

Most, if not all, shaft resistance develops at a very small relative movement, rarely more than 10 mm. Because a pile toe does not develop ultimate toe resistance, but responds by a continuous movement for increasing load, the magnitude of the pile head movement depends on the q-z function in response to the load reaching the pile toe, as applicable to each particular case.

Moreover, the shaft resistance is limited by the weight of the soil between the piles in the group. For close spacings and long piles, this means that most if not all the sustained load on the piles will be applied at the pile toe level (Fellenius 2015).

For toe bearing piles, the toe response could show to be so stiff that the shaft resistance near the pile toe is not fully mobilized.

Component 2 comes about for pile groups, usually groups of at least a few metre in width and length, e.g., four pile rows and columns or more. It is caused by the compression of the soil below the pile toe level as the individual pile loads overlap and increase the effective stress in the soil layers below the pile toe level. Settlement of the soils below the pile toe is rarely of concern for single piles, small pile groups, and toe bearing piles installed to dense non-compressible soils. It will be noteworthy for very large pile groups, however, where the overlapping stresses will influence a large soil volume of soil below the pile toe level, if these soil layers are compressible. It can easily be calculated by modeling the pile group as an "equivalent raft", as discussed below. Component 2 develops over time, but inasmuch the soils below the pile toe level are normally pervious, the compression due to consolidation usually occurs quickly.

Component 3 is independent of the load applied to the pile head from the structure and mostly affects single piles, small pile groups, and the perimeter piles of large pile groups installed in settling soil. It develops in the long-term and is due to (1) the 'elastic' shortening of the piles caused by the drag force, i.e., the increase of load in the pile with time as the upper soil layers move down relative to the pile and (2) the additional pile toe penetration imposed by the downdrag (pile settlement at the neutral plane). The shortening is in addition to the shortening from transferring the sustained load down the pile. Note that the drag force will not contribute to the soil settlement, only to pile shortening due to the increased load and to toe penetration. Component 3 develops over time and the process can take many years.

If the soil surrounding the piles displays minimal long-term settlement at the neutral plane, Component 3 is small and the zone of transition from the negative skin friction and positive shaft resistance will be long. If, however, the soil layers around the pile or piles settle appreciably due to, for example, significant lowering of the groundwater table, fill placed on the ground, regional settlement of surficial layers, etc., the transition zone is short. Essentially, the length of the transition from above, or below, the neutral plane is to where the relative settlement between the pile and the soil is about equal to the movement necessary to mobilize the shaft resistance.

The pile downdrag can become much larger than the settlement developed from the Components 1 and 2, and it is sometimes the most serious cause of settlement of a piled foundation. Fellenius (2006) has summarized a series of downdrag case histories reporting long-term observations of drag force and downdrag.

Estimating the settlement component due to downdrag involves determining the load distribution in the pile and analyzing the pile toe penetration and pile toe load, i.e., applying the pile stiffness response, as discussed by Fellenius (2004; 2015) and Fellenius and Ochoa (2009).

Terzaghi and Peck (1948) proposed to model the settlement of a group of essentially shaft-bearing piles supporting moderate structural loads as that of an equivalent raft with the same footprint as the pile group, placed at the lower third point of the pile length, and loaded to the same stress as the piled foundation, spreading the stress down into the soil below the raft. Later on, it became clear that the lower third point for the mentioned Terzaghi and Peck pile group coincides quite closely with the location of the force equilibrium (neutral plane) for the piles (Fellenius 1984; 1988). Indeed, the neutral plane is where the load applied to the pile group from the structure starts to be transferred to the soil. It is also the location of the settlement equilibrium, i.e., where the soil and pile settlement are equal and no relative movement occur between the pile and the soil.

When calculating settlement below the equivalent raft, one must take into account the significant stiffening of the soil due to the presence of the piles, and modify the values of soil compressibility accordingly for the combined two-component material. Furthermore, the changes of effective stress around the pile group due to other effects than the load applied to the particular pile group analyzed, must also be included in the analysis. Such changes come from fill, other foundations, excavations, lowering of the groundwater table, etc.

Starting at the neutral plane, some of the applied load and drag force are transferred—spread out—to the soil through the shaft resistance below the neutral plane. For a large pile group, the spreading makes very little difference to the calculated settlement—provided the mentioned stiffening of the soil due to the piles from the neutral plane to the pile toe level is considered. For a small pile group, however, the load transferred by shaft resistance to the soil affects the soil layers below the pile toe. The portion of the load that reaches the pile toe is a separate matter. Distributing all of the shaft resistance portion from an equivalent raft placed at the neutral plane will result in a distribution to a too large an area at the pile toe level.

Fellenius (2015) suggests that the transfer of shaft resistance to the pile toe level can be calculated as originating from a series of pile elements with the resistance from each element distributed, for example, according to the  $2(V) : 1(H)$  method considering each element to be a footing unit.

The mentioned distributions result in an average footprint area at the pile toe that has a stress level about equal to that of an equivalent raft at the neutral plane for which the load applied to the piled foundation is spread to the pile toe at  $5(V) : 1(H)$  from the neutral plane, as indicated in Figure 1.

Inasmuch the soil below the pile toe level is compressible, calculations for the settlement in the soils underneath the pile toe level of a flexible equivalent raft placed at the pile toe will provide

reasonable estimate of the settlement of the piled foundation. The stress distribution below the pile toe level can be calculated by means of the  $2(V) : 1(H)$  method, which will provide an average of the calculated settlement of the pile group. If the distribution across the pile group is desired, the stress calculation should be by means of the Boussinesq distribution.

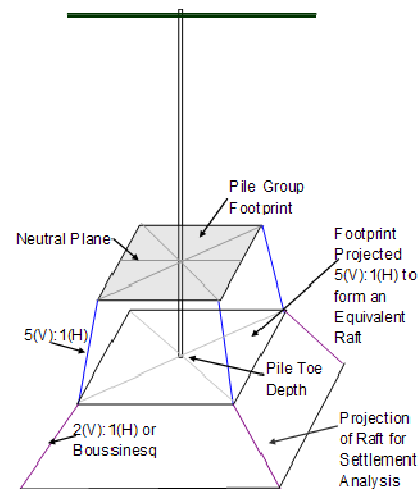


Figure 1 Distribution of stress between the neutral plane and the pile toe level for a small group of piles according to Fellenius (2015). Only one pile is shown.

For other than small pile groups, the effective stress distribution inside the pile group is limited to the buoyant weight of the soil between the piles, which means that along the upper length of the piles the shaft shear is smaller than for a single pile. Therefore, the unit negative skin friction and the drag force are smaller than in the free field. Along the lower length, the combined effect of the drag force and the sustained load produce a larger unit shaft resistance, which combined with the lower shear forces along the upper length results in a neutral plane very near the pile toe. For large pile groups, therefore, the spreading of load below the neutral plane to the pile toe is negligible and can be omitted.

### 3. CASE HISTORIES

#### 3.1 Case 1 — QIT Plant, Quebec

Golder and Osler (1968) presented a case history of twelve years of settlement measurements of a bank of five furnaces placed with long sides in parallel next to each other at a depth of 1.5 m and about 6 m apart over a total footprint of about 16 m by 54 m. Each furnace had a 16 m by 10 m footprint and was supported on a group of thirty-two, about 6 m long, 600 mm diameter expanded-base piles (Franki piles) installed to a depth of 8.5 m at center-to-center spacings ranging from 2.1 m through 3.2 m. The average footprint ratio (total cross sectional area of the piles over total foundation footprint) was about 6 %. The total furnace load was 21 MN/unit, that is, 660 kN per pile and an average stress of 130 kPa over each furnace footprint. The soil profile consisted of an upper 24 m thick, compact to dense sand deposit on a more than 50 m thick layer of soft compressible clay. The groundwater table was at 4 m depth. A static loading test to 1,800 kN performed before constructing the furnaces showed a maximum pile head movement of 3 mm.

The furnaces were built in early 1951. Settlement of the furnaces was monitored until January 1962 at six benchmarks placed between the furnaces. Figure 2 shows the settlements measured for the furnaces from April 1951 (when all five furnaces were completed) through January 1962 for time in both linear and logarithmic scales. The straight-line development of the settlements versus log of time diagram implies that consolidation settlement was continuing when the last (1962) readings were taken.

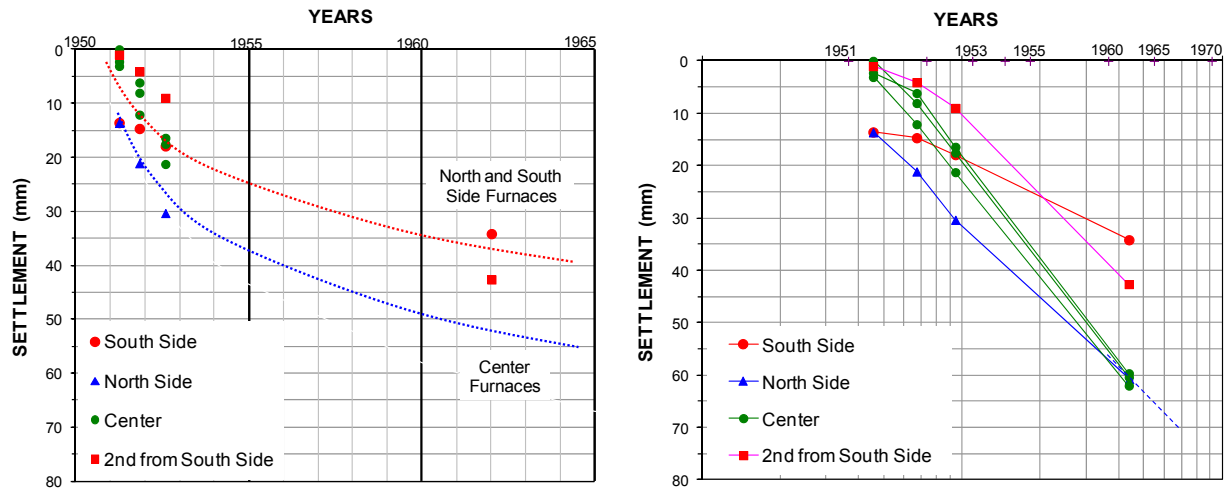


Figure 2 Settlements versus time in linear and logarithmic axes (Data from Golder and Osler 1968)

Figure 3 shows the settlement measured along the center of the furnaces and the settlement calculated using Boussinesq stress distribution, compressibility parameters, settlement, and conventional consolidation approach, as fitted to the January 1962 settlement for the center of Furnace 3. The parameters obtained by the fitting were used to calculate the settlements for a flexible equivalent raft placed at the pile toe level. As indicated in the figure, the calculated and measured values agree well.

The settlement calculations shown in Figure 3 and other figures in this paper are prepared using the UniSettle software (Goudreault and Fellenius 2011).

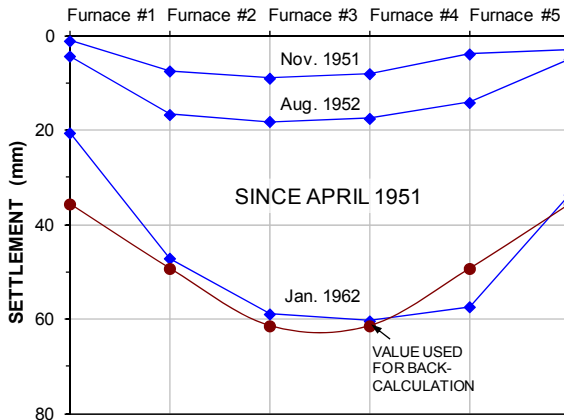


Figure 3 Measured and calculated settlements along the center of the furnaces (Fellenius 2011)

### 3.2 Case 2 — Ghent Silos, Belgium

Goossens and VanImpe (1991) presented results of ten years of monitoring settlement along the side of a tightly-spaced group of 40 grain silos, 52 m in height, founded on a 1.2 m thick concrete raft with an 84 by 34 m footprint. The raft was supported on 697 piles, consisting of 520 mm diameter, 13.4 m long, cast-in-place concrete piles with expanded base (Franki piles) with a working load of about 1,200 kN. Two static loading tests to 2,250 kN performed before constructing the furnaces showed a maximum pile head movement of 7 mm. The average footprint ratio was 5 %. The soil profile consisted of sand alternating with clay. The groundwater table was at 3.0 m depth. For fully loaded silos, the total load distributed evenly across the footprint corresponded to a stress of about 300 kPa.

Based on the results of the static loading test, the settlement of the piled foundation was expected to be small. Still, to investigate the long-term development, a programme of settlement monitoring at five bench marks affixed to the raft a long one side was implemented. Figure 4 shows the measured settlement (solid line) and as calculated using Boussinesq stress distribution, compressibility parameters, and conventional consolidation approach (dashed lines) for a flexible raft placed at the pile toe level. The fit of calculated value to the measured value at Benchmark BM2A calibrated the input to the analysis.

The figure also shows a settlement curve back-calculated for the center line of the piled raft foundation using the so-calibrated soil parameters. The settlements calculated for the center line indicate that the differential settlement between the center and the corner would have been about 200 mm over 40 m, about 1:200. However, Goossens and VanImpe (1991) reported no sign of distress for the silo structure.

Again, the settlement measured at the benchmarks and the values calculated using the parameters matched the settlement at the BM2A benchmark. This indicates that the settlement of the piled foundation can be correctly modeled by a conventional Boussinesq stress distribution analysis of a flexible raft foundation located at the pile toe level.

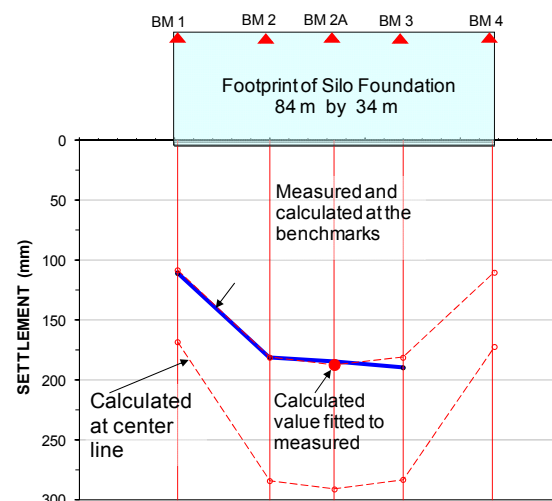


Figure 4 Settlements along the side of the silo foundation raft (data from Goossens and VanImpe 1991)

### 3.3 Case 3 — Oil tank in Egypt

It is of interest to compare the settlement for wide tanks supported on ground, as opposed to a piled foundation. El Far and Davie (2008) presented a case history of settlements of the perimeter of a 30 m diameter oil storage tank in Damietta, Egypt, placed at a depth of about 1.5 m on natural soil consisting of "interbedded sand and clay". Figure 5 shows the size of the tank, details of the soil profile, and compressibility parameters. The settlements at the tank center and perimeter predicted for a hydrotest were about 550 mm and 300 mm, respectively. The modulus numbers stand for the compressibility by the Janbu modulus number, which is mathematically equal to expressing the soil compressibility by an E modulus or a  $C_c$ - $e_0$  pair (Janbu 1967; 1998, CFEM 1992, Fellenius 2015). The modulus numbers,  $m$ , shown in the figure were determined from the compressibilities indicated in the original paper. (The magnitude of the modulus numbers for the sand layers suggest that they represent reloading conditions as opposed to virgin conditions).

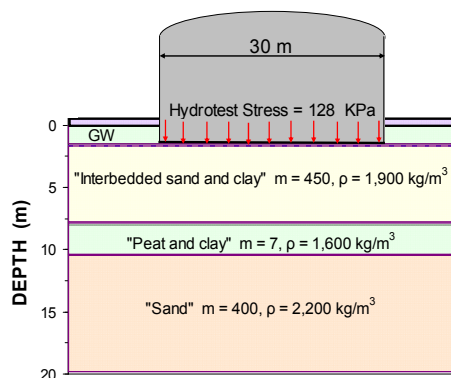


Figure 5 Tank at Damietta, Egypt (El Far and Davie 2008)

The area was raised by placing a 0.5 m of fill around and below the tank footprint before construction. The tank was hydrotested to a maximum stress of 128 kPa (estimated to be 125 % of the stress when fully loaded with oil). Filling the tank with water took 40 days, the load was kept on for 125 days, and emptying the tank took 60 days. The settlement was monitored at four benchmarks equally spaced along the tank perimeter.

Figure 6 shows the loading schedule and the settlements measured at the four benchmarks. No benchmark was placed at the tank center. The plotted dots represent the average settlement at the four benchmarks for each measurement occasion.

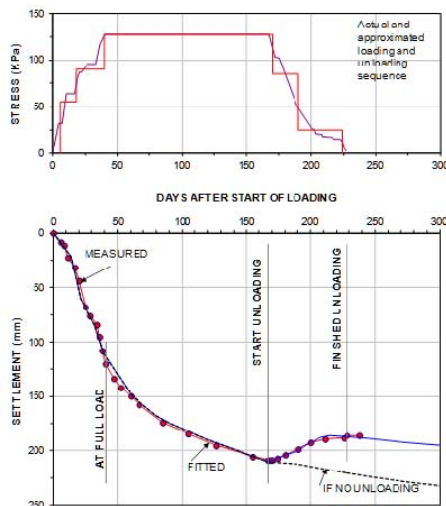


Figure 6 Hydrotest loading sequence and measured settlements (measurement data from El Far and Davie 2008)

Figure 7 shows a plot in logarithmic time-scale of the measured settlements. The lines are approximately straight, implying that the consolidation in the peat and clay layer might not yet have been completed, i.e., have yet to reach an about 90 % degree of consolidation.

The average of the four settlement curves was used to fit a calculation of settlement versus time. The dominant layer for settlement is the 2.5 m thick, very compressible layer of "Peat and clay" between depths 8.0 m and 10.5 m. The loading was modeled as a three-step increase of the stress and the unloading as a three-step removal of the stress.

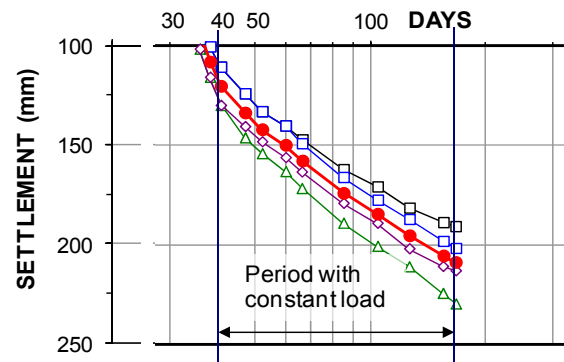


Figure 7 Development during the time for constant load plotted to logarithmic time scale (measurement data from El Far and Davie 2008)

The results of the best-fit calculation to the measured settlements are shown again in Figure 8. The fit was achieved by adjusting by trial-and-error the input of coefficients of consolidation for the peat and clay layer and by the re-compression modulus numbers (determined in the calculation of heave due to the unloading of the tank) for the soil layers with regard to immediate compression and consolidation. The figure also includes the calculated settlement for the center of the tank (which was not measured) indicating an about 200-mm differential settlement between the tank perimeter and center.

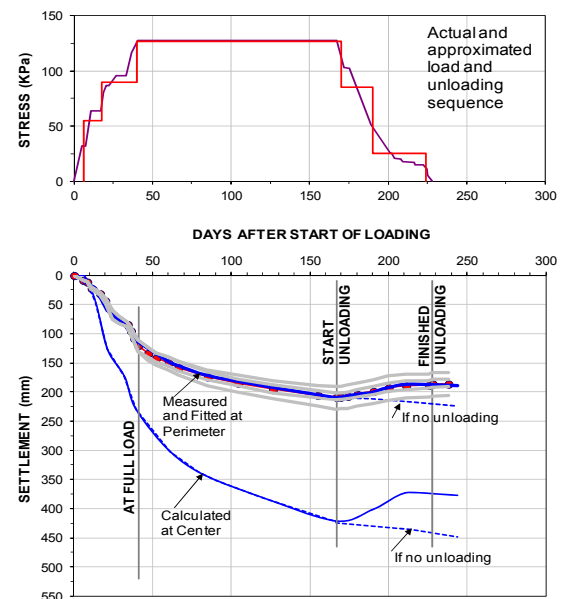


Figure 8 Hydrotest loading sequence and average measured perimeter settlement curve, calculated settlement fitted to the perimeter values, and calculated settlement for the center of the tank using the parameters obtained through the fitting procedure (measurement data from El Far and Davie 2008).

The hydrotest served as a preloading of the soil. The settlements during later use of the tank for oil storage can be expected to follow the reloading moduli of the soil determined from the unloading of the tank. The settlement of the tank in actual use was not monitored. However, modeling of the re-filling of the tank to full height using the parameters established in the modeling of the hydrotest results indicates that the settlement would have been about 30 mm along the perimeter and about 60 mm at the tank center.

### 3.4 Case 4 — LNG tanks in Barcelona, Spain

Leira Velasco and Lobato Kropnick (2007) reported a case history of settlement for two, about 80 m diameter, 150,000 m<sup>3</sup> storage volume, LNG tanks at the Port of Barcelona, labeled TK-3000 and TK-3001. As shown in Figure 9, the soil profile at the tank location consisted of about 6 m of loose non-engineered fill and debris, which was removed and replaced with sand and gravel at Tank TK-3000; containing cobbles at Tank TK-3001. In lieu of piles, the sand and gravel replacement fill at Tank TK-3000 was compacted using dynamic consolidation (dynamic tamping).

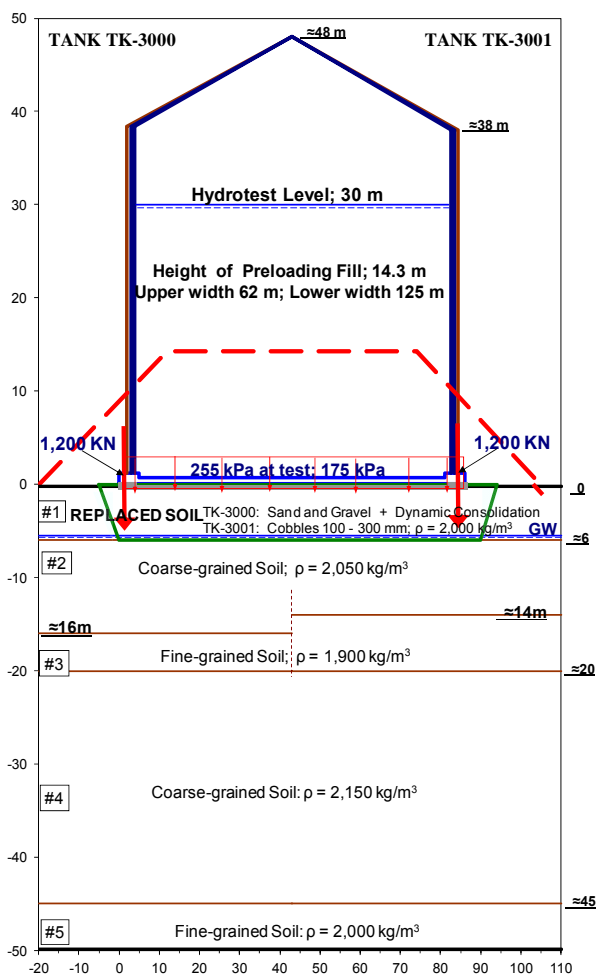


Figure 9 LNG tank at Port of Barcelona, Spain. Tank and soil profile (Data from Leira Velasco and Lobato Kropnick 2007)

The natural soil below 6 m depth consisted of dense coarse-grained soil—to about 14 m depth at Tank TK-3001 and to about 16 m at Tank TK-3000. Between this layer and a main deposit of very dense coarse-grained soil (extending to at least 50 m depth) was a layer of compressible, fine-grained soil, about 4 m thick at Tank TK-3000 and about 6 m thick at Tank TK-3001. The left side of Figure 9 shows conditions for Tank TK-3000. The right side shows those for Tank TK-3001. The figure also shows geometric details of the excavation and back-filling volume and the extent of a

preloading surcharge of the tank area. Note that the excavation and back-filling was essentially only carried out under the tank footprints.

The effect of the dynamic consolidation at Tank TK-3000 of the replacement sand and gravel was investigated by means of two SPT-borings and two CPTU soundings. Figure 10 shows the SPT N-Indices to about 5 m depth (through the replacement soil layer) before and after the dynamic consolidation (tamping).

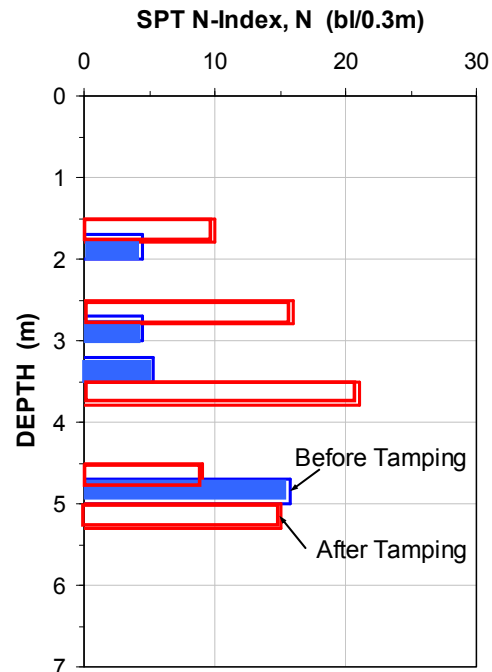


Figure 10 SPT Indices in the upper 6 m thick replacement soil before and after dynamic consolidation (Data from Leira Velasco and Lobato Kropnick 2007)

Figure 11 shows the cone stress,  $q_b$ , of a CPTU-sounding pushed through the replacement soil and about 4 m into the coarse-grained soil before and after the dynamic consolidation. The figure also shows a diagram of the soil compressibility (Janbu virgin modulus numbers) calculated from the cone stress values according to Massarsch (1994) and Fellenius (2015). The diagrams indicate that the tamping was successful in densifying the replacement soil.

After the dynamic consolidation treatment and before the construction of the two LNG tanks, both areas were subjected to a preloading during 110 days (Tank TK-3000) and 250 days (Tank TK-3001). The preloading consisted of placing a 14 m high fill over the footprint of the tanks and about 20 m beyond the tank footprints.

During the preloading, settlements were monitored along the perimeter and center of each tank as well as at the edge of each fill. Figure 12 shows the measured settlements versus time (days) in linear scale for the ground level of the two tanks at tank centers, perimeters, and outer edges of fill.

Figure 13 shows the settlements plotted during the days of constant height of fill and after removal of the fill. In Figure 14 the settlements are plotted versus time (days) in a logarithmic scale. The trend is toward a more horizontal curve toward the end of the period of constant fill height after initial linearity of the curves indicating that consolidation of the compressible layer had ceased. Note that for Tanks TK-3000 and TK-3001 with 4 and 6 m thick compressible layers, respectively, the length of time for achieving the full consolidation appears to have been 55 days and 120 days, respectively. That is, the relative consolidation time was approximately proportional to the square of the thickness ratio of the two compressible layers, which is in agreement with consolidation theory.



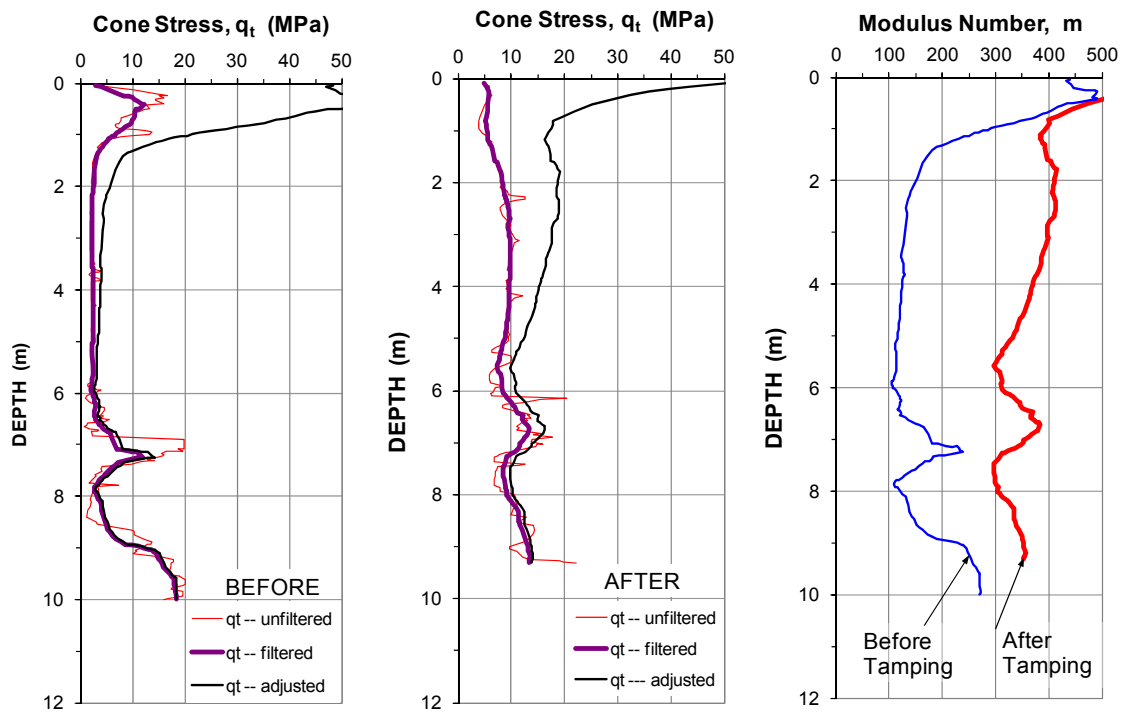


Figure 11 Cone stress before and after dynamic consolidation (two left diagrams) and Janbu modulus numbers (right diagram) determined from the cone stress values before and after tamping (Date from Leira Velasco and Lobato Kropnick 2007)

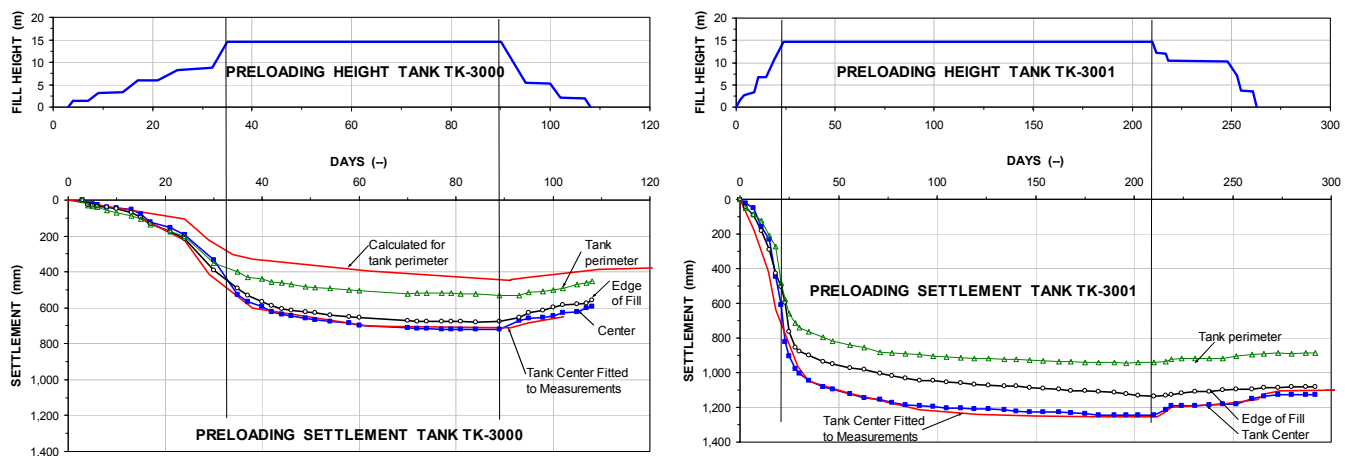


Figure 12 Preloading schedule and settlement versus time for Tanks TK-3000 and TK-3001

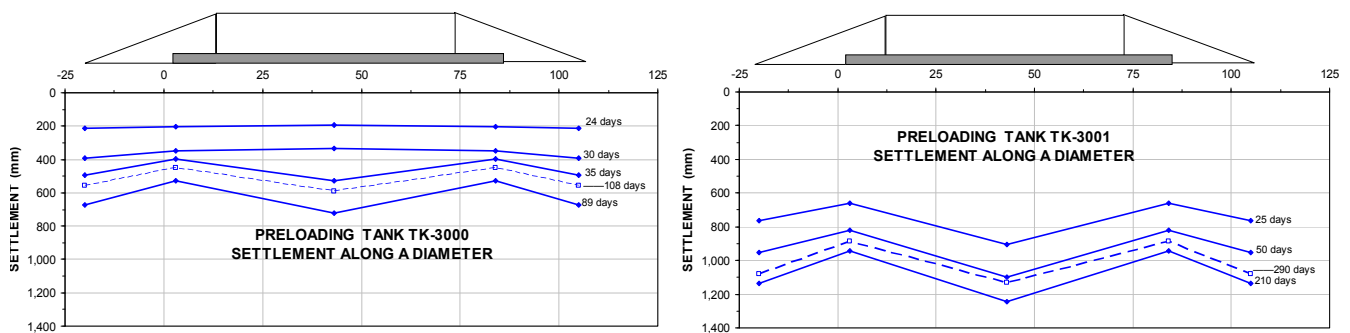


Figure 13 Preloading settlements along a diameter for Tanks TK-3000 and TK-3001

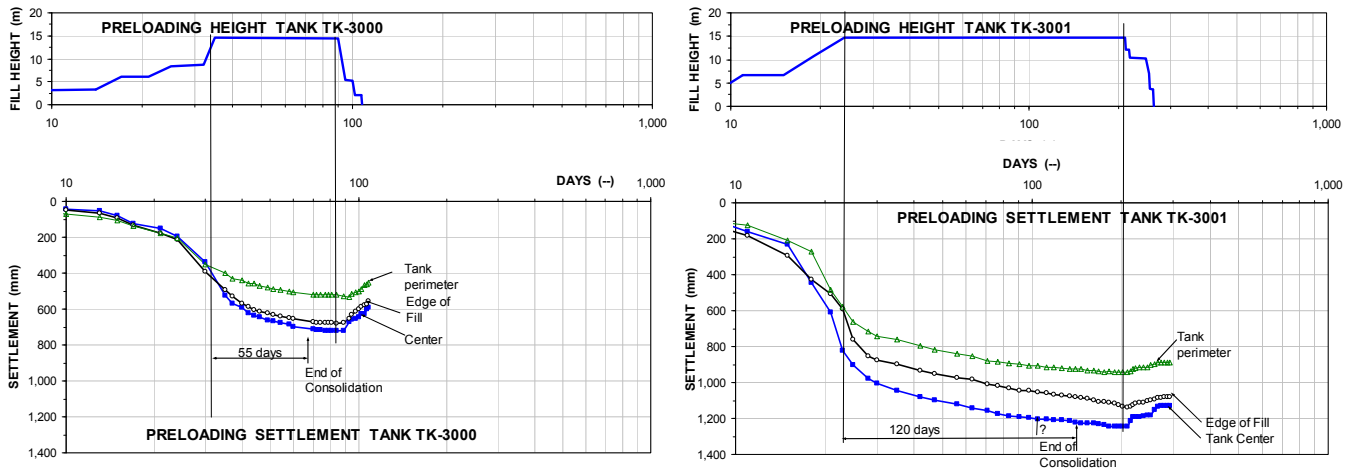


Figure 14 Preloading settlements along the tank diameters in log scale

A best-fit settlement calculation was applied to the measured center settlement. Other than the thickness of the compressible layer and the duration of the fill at constant height, the same parameters were used for both tank calculations. The values used are shown in Table 1. When these best-fit values were used to calculate the settlement for the perimeter of the tank and the "edge of fill", there was little agreement because lateral spreading had occurred under the perimeter and, in particular at the edge of the fill: the soil had "flowed" into the soft/loose original debris layer outside the excavation and back-filling zone. This increased the settlement at the tank perimeter and edge of the fill.

Table 1 Best-fit parameters determined for the Barcelona case history (Leira Velasco and Lobato Kropnick 2007)

Layer (m <sup>2</sup> /year	Depth (m)	Immediate		Consolidation		
		m <sub>i</sub>	m <sub>ir</sub>	m	m <sub>r</sub>	c <sub>v</sub>
Sand & Gravel	6	100	1,000			
Cobbles	6	40	400			
Coarse sand	16 & 14	200	2,000			
Fine-grained	4 & 6	100	600	8	80	40
Coarse-grained	30	200	2,000			

where:  $m_i$  = immediate-compression virgin modulus  
 $m_{ir}$  = immediate-compression re-compression modulus  
 $m$  = consolidation virgin modulus  
 $m_r$  = consolidation re-compression modulus  
 $c_v$  = consolidation coefficient

When the preloading fill had been removed, the tanks were constructed. Before they were put to use, they were hydrotested. Figure 15 shows the measured settlements for the center and the perimeter of Tank TK-3000. The settlements for Tank TK-3001 were very similar. The figure also shows the calculated curves for the center and the perimeter obtained using the best fit parameters developed for the preloading case, Table 1.

The compacted fill distributed the applied stress much the same way as would a piled foundation.

### 3.5 Case 5 — Liquid Ammonia Storage Tank in Port of Thessaloniki, Greece

Badellas et al. (1988) and Savvaadis (2003) presented a case history of settlement measurements for a 38 m diameter, liquid storage tank

in Greece supported on a piled foundation. The soil profile consisted of 40 m of soft compressible soil followed by dense coarse-grained soil. The groundwater table was at about 1.5 m depth.

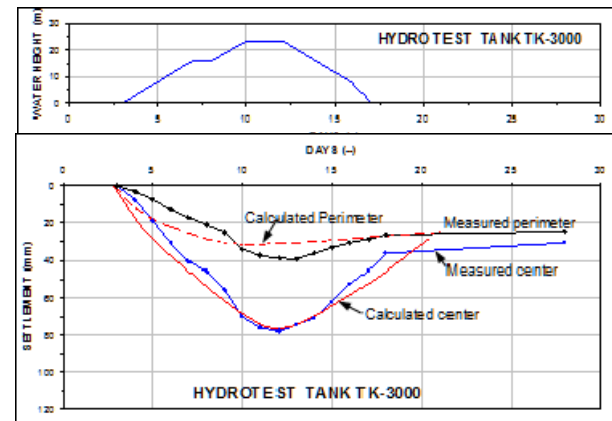


Figure 15 Hydrotest for Tank TK-3000 showing water height in tank and settlements at center and along perimeter as measured and as calculated using the best-fit parameters of Table 1

The tank bottom consisted of an 800 mm thick concrete raft and the total dead weight of the empty tank is 70,000 kN (about 60 kPa stress). The foundation comprised a total of 112, 1,000 mm diameter, 42 m long bored piles. The footprint ratio was about 8 % and the average spacing was about 3.6 pile diameters c/c. Figure 16a shows a view of the tank and Figure 16b the layout of the piles. Fourteen piles were monitored for settlement during a hydrotest. The location of three of these, Piles 7, 11, and 16, are indicated in the figure.

A 30-day hydrotest to a height of about 17 m was performed scheduling ten days of loading, ten days of holding the height, and ten days of removing the water. The settlement of the mentioned three pile heads was monitored. Figure 17, upper diagram, shows the sequence of water loading. Figure 17, lower diagram, shows the settlements measured during the hydrotest for the three monitored piles.

The cases show that conventional soil compressibility parameters obtained from the best-fit between measured and calculated settlements for the preloading event gave settlement values for the hydrotest event that were in a good agreement between measured and calculated settlements of the tank center and perimeter.

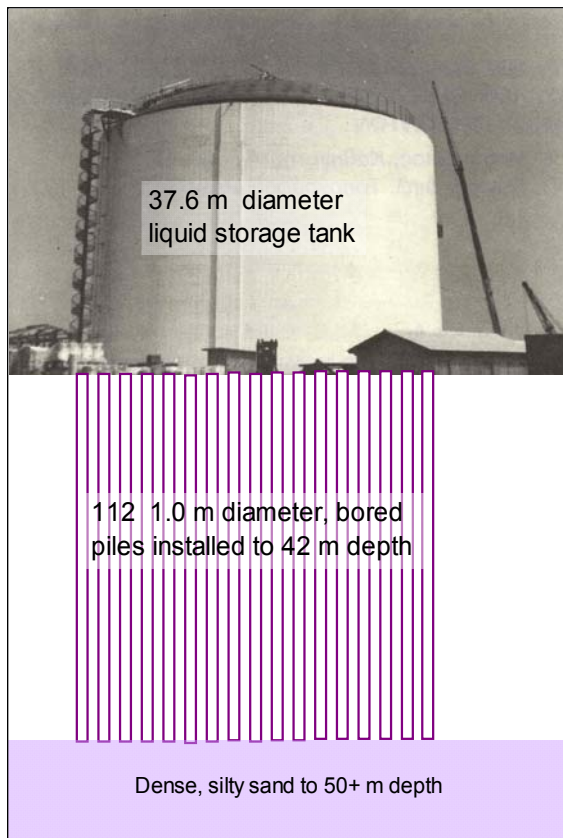


Figure 16a The Thessaloniki tank

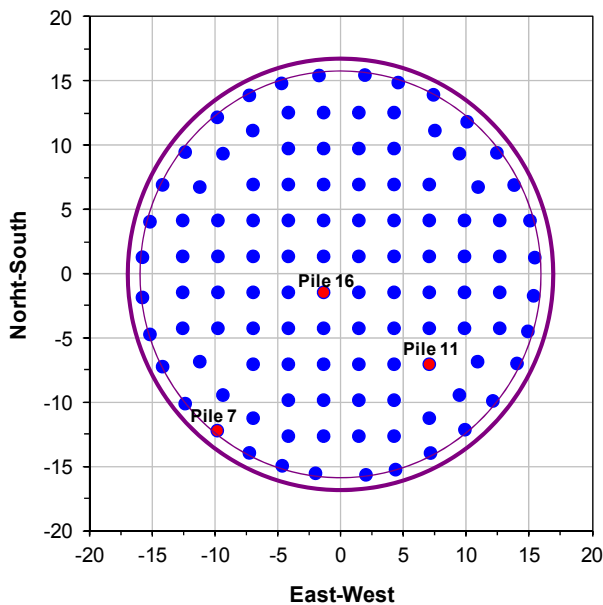


Figure 16b Layout of the piles for the Thessaloniki tank

Figure 18 shows the settlement measured along a diameter of the tank settlement at the end of the hydrotest, as extracted from a contour line graph in the original paper. The distribution clearly indicates that also the piled foundation responded to the loading and settlement as a flexible raft. Back-calculations were made for an Equivalent Raft placed at the pile toe depth with the load-spreading to the raft per the mentioned method. The fitted conditions were used to calculate the settlement along the full diameter. The calculations assumed negligible compression of the piles and that, in

calculating the settlements, the pile group could be modeled as a raft loaded uniformly with the weight of the tank and its stored liquid. The resulting soil parameters indicated a 100-kPa preconsolidation margin ( $\sigma'_p - \sigma'_{v0}$ ), a virgin elastic modulus of 25 MPa ( $m = 250$ ), and a re-compression elastic modulus of 50 MPa ( $m_r = 500$ ). The stress below the raft was per Boussinesq distribution.

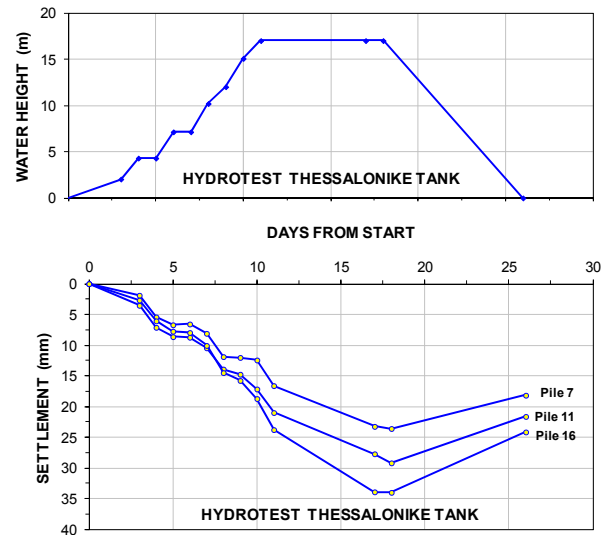


Figure 17 Hydrotest for the Thessaloniki tank showing water height in tank and settlements measured at three piles (Data from Badellas et al. 1988, Savvaidis 2003)

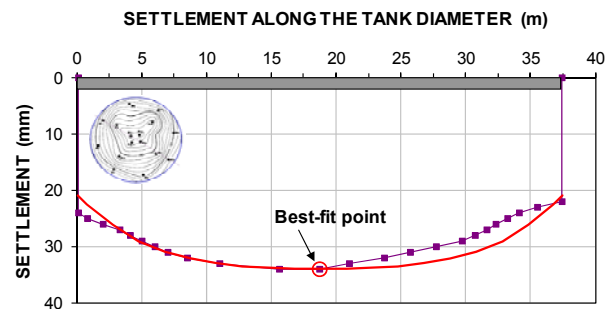


Figure 18 Measured and calculated settlements for the Thessaloniki hydro tested tank

#### 4. THE ANALYSIS METHOD APPLIED TO A WIDE TANK ON A PILED FOUNDATION

We have applied the foregoing analysis approach to a hypothetical, but typical, liquid storage tank; an 84 m diameter LNG tank, placed on a piled foundation. The tank base consists of a 0.6 m thick concrete raft.

Typical for a coastal LNG site, the soil at the tank site is assumed composed of 25 m of moderately compressible, normally consolidated clay on 10 m of dense sand followed by 25 m of moderately compressible, slightly preconsolidated clay on very dense gravel at 60 m depth. The groundwater table lies at the 2-m depth. An upward water gradient exists in the clay, corresponding to a 2.0 m artesian head (2.0 m above ground) in the sand. The pore pressure distribution in the lower clay layer is hydrostatic. To prepare the site for construction, a 1.0 m thick fill will be placed over a large area of the site.

The tank foundation is placed on 1,400, square, 400 mm side concrete piles driven to 30 m depth below original ground surface, i.e., to about the mid-point of the sand layer. The footprint ratio of the piled foundation is 4.0 %, and the pile spacing, c/c, is 2 m, which corresponds to 5.0 pile diameters. The maximum load from the tank in service is 200 kPa, corresponding to an average of 800



kN per pile. Figure 19 shows the approximate layout of the tank and piles. The project will also include several small pile groups spread out across the site made up of two to about ten piles, supporting machinery, connecting pipe racks, and other units with concentrated loads.

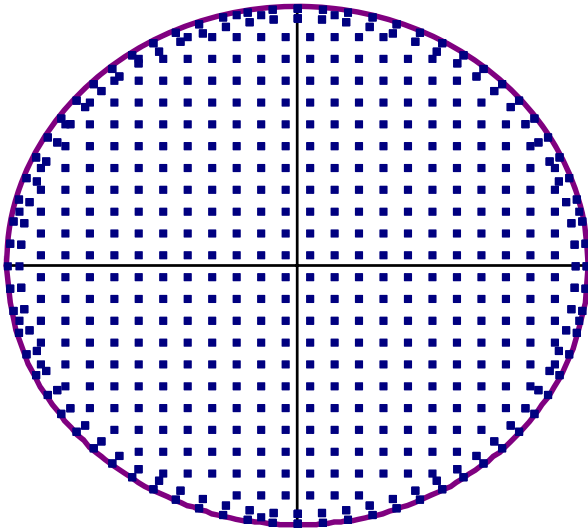


Figure 19 Plan view and pile layout of the simulated hypothetical tank

Figure 20 shows the results of a simulation of a static loading test for the typical pile. The pile is assumed instrumented so as to provide full information on the pile shaft and toe resistance response. The loading test simulation is made using the UniPile program (Goudreau and Fellenius 2013) for effective stress method of analysis. The simulation includes the effect of residual load. Shaft and toe resistances at the Offset limit “capacity” is indicated.

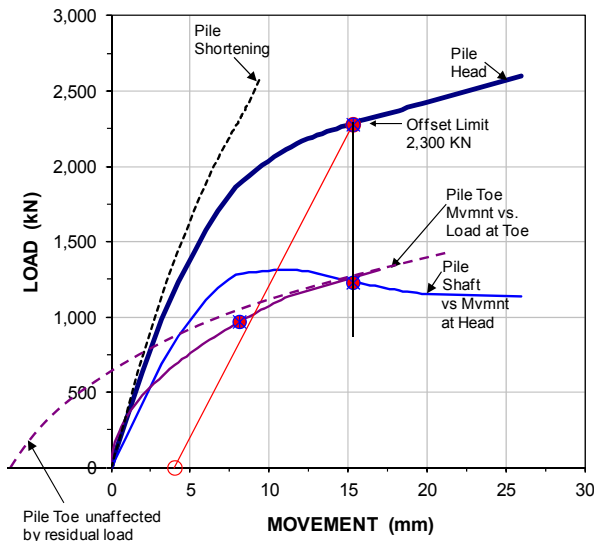


Figure 20 Simulated load-movement curves

For the small pile groups at the site away from the tank, conventional design rules require the pile capacity to be larger than the maximum working load to be supported by the pile. In North America, it is common to require a factor-of-safety ranging from 2.0

through 2.5 on the capacity established in a static loading test with the capacity determined according to the offset limit as illustrated in Figure 20. The simulated test indicates that the mentioned 800 kN working load is well within this condition. The shaft and toe resistances mobilized due to an applied pile head load equal to the offset-limit load are indicated on the respective curves.

Figure 21 shows the distribution of load and resistance for a single pile. Two curves are shown for the load distribution: one for the test condition and one for the long-term condition, when the soils will have consolidated by the fill placed on the ground.

Piles used as single piles or in small groups will be subjected to drag force from accumulated negative skin friction. The load from the structure plus the drag force will impose a maximum load on the pile, which for the long-term condition will amount to about 2,000 kN at the neutral plane—the force equilibrium. This is of no consequence for the subject piles, as the structural strength of the pile section is more than adequate to accept this load. Moreover, beyond a small load-transfer movement, no significant settlement will develop from the drag force. The long-term settlement is therefore only the downdrag caused by the area fill.

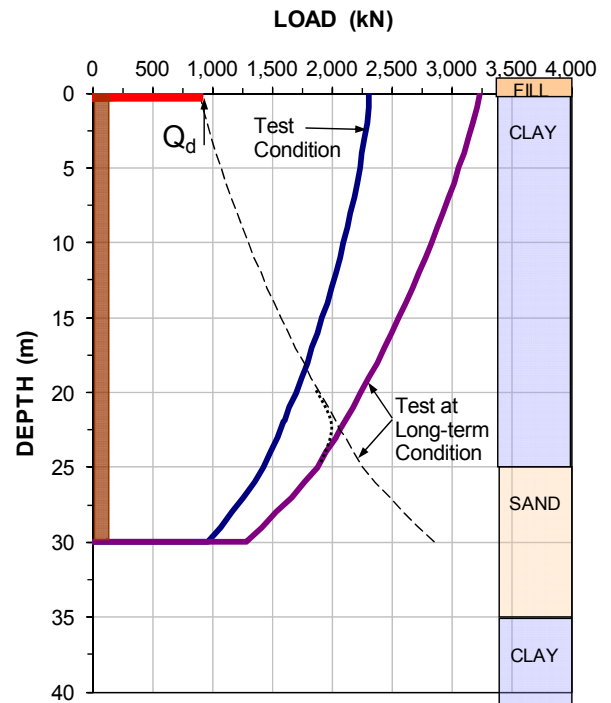


Figure 21 Load and resistance distribution for the single pile ( $Q_d$  = Dead Load)

Single piles and small pile groups will experience downdrag, because the soil at the neutral plane, the settlement equilibrium, is settling, as illustrated in Figure 22 showing the long-term load and resistance distributions and the distribution of settlement. The figure illustrates the basics of the "Unified Pile Design Method" (Fellenius 1984; 1988; 2004; 2015 and Fellenius and Ochoa 2009).

For most cases similar to the simulated case, the 50 mm long-term settlement caused by the fill would be acceptable. However, the assessment of the tank foundation will have to consider also the pile group effect. The capacity of a single pile in the group is of little relevance. As discussed by Fellenius (2015), for the interior piles in the group, the maximum shaft resistance and for that matter, the maximum drag force, is limited to the weight of the soil in-between the piles. That weight is simply the effective stress

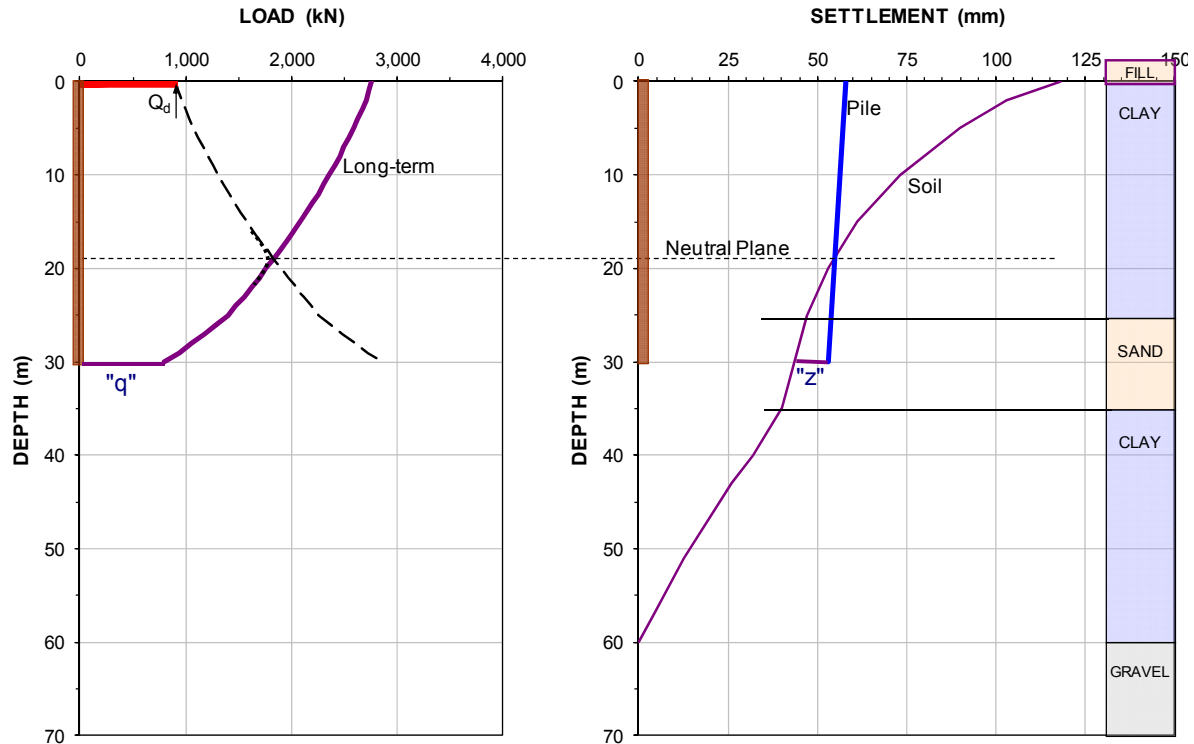


Figure 22 Load and resistance distribution supplemented with settlement distribution

times the height of the soil above the considered point times the soil area per pile (about  $4.0 \text{ m}^2$ ). For the subject case, fully mobilized shaft resistance for a single pile is about 2.5 times larger than the in-between weight. In contrast, the piles along the tank perimeter will have drag forces equivalent to fully mobilized shaft resistance.

Because the piles are connected to a common slab or pile cap—the tank base—the larger drag force acting on the perimeter piles will cause the loads from the structure to be directed to the interior piles. For the assumed typical case, the drag force acting on the perimeter piles will be smaller than the sustained pile load (the dead load). Therefore, this redistribution of load is of negligible consequence for the simulated typical case. The main question for the assessment is the settlement of the tank foundation.

The settlement response of a piled foundation similar to the simulated typical case is best modeled as the settlement of a flexible "Equivalent Raft" placed at the pile toe depth, as was assumed in the analysis of the case records of the tanks with piled foundations presented in the foregoing. It is often assumed that the hydrotest will provide information applicable to the long-term settlement of the tank foundation, as was indicated in the case histories quoted in this paper. However, the settlement response of the foundation of the assumed typical case is much more affected by consolidation than were the responses for the case histories. The calculated settlement response at the tank center of an assumed 30 day hydrotest involving ten days of filling the tank, ten days of full height, and ten days of emptying the tank is shown in Figure 23.

It would indeed be easy to believe that the hydrotest has indicated also the long-term settlement of the tank. That is, that the long-term settlement would be about equal to the settlement measured in the hydrotest. This could however be misleading, because the consolidation in the clay below takes a long time to develop and it would have had very little time to develop during the test.

Figure 24 shows the results of a simulation of the long-term settlement (combining "immediate", "consolidation", and "secondary") for the center and the perimeter of the tank foundation, as well as the settlement of the general area "away from the tank" as

affected by the fill only. Whether or not the about 100 mm differential settlement between the perimeter and the center of the tank is of concern can now be rationally assessed. The key point is that the response over short-term and long-term can be rationally and readily assessed using conventional soil mechanics principles.

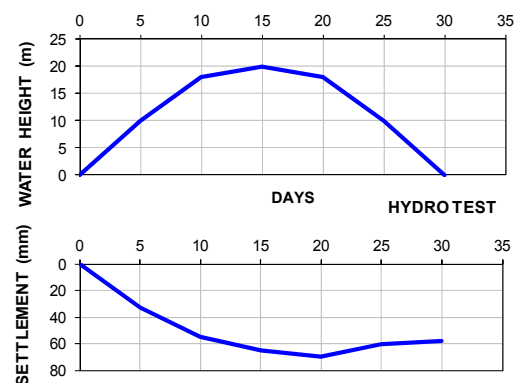


Figure 23 Simulated settlement response for an assumed hydrotest

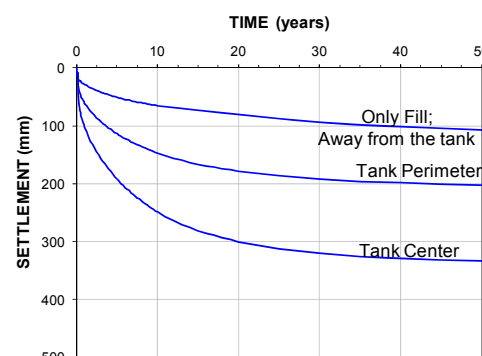


Figure 24 Simulated long-term settlements

It should be noted that the settlement calculations do not account for the fact that the downdrag, i.e., settlement due to the causes other than the pile load, will affect the perimeter piles more than the center piles, thus, offsetting to some degree the differential settlement between the perimeter and the center tank area due to the load from the tank.

Often the center piles in a large pile group are made longer than the perimeter pile ostensibly to minimize differential settlement by offsetting the larger settlement expected for the central portion of the foundation. However, for LNG tanks and similarly large-area piled-foundations at sites containing layers of compressible soil undergoing settlement, to offset the influence of the downdrag on the perimeter piles, a better approach may be to lengthen the perimeter piles (pile rows) to offset the downdrag effect. Settlement analysis of the foundations that include all relevant aspects will point toward the best approach.

## 5. CONCLUSIONS

The five case histories of settlement of foundations with a wide footprint presented in this paper show that, whether or not the foundations are supported by piles, differential settlement develops between perimeter and interior parts of the foundation, indicating a flexible response to the applied load.

The best-fit back-analyses of the histories show that the settlements can be consistently modeled by assuming the foundation raft to be flexible and applying Boussinesq stress distribution even for a slab (pile cap) thickness of 1.2 m, as used for the silo structure (Case 2, above). The slab thickness of tanks for storage of liquids is normally thinner than 0.7 m. However, be the thickness more than 1 metre, a wide footprint slab might still act as a flexible raft with respect to transferring forces to the soil.

Design of a foundation supported on a single pile or a small pile group needs to appraise both axial pile bearing capacity and downdrag aspects. Settlement for the load applied from the supported structure is limited to load transfer response. For pile groups in-between being "small" or "large", potential magnitude of settlement due to the load applied to the piles can be established by analyzing an equivalent raft placed at the neutral plane with the load from the structure distributed from that raft with due inclusion of the stiffening effect of the piles.

In contrast to the design of single piles and small pile groups, design of large pile groups is dominated by consolidation settlement due to the load applied to the pile group.

The analyses of the case history records show that the settlements can be calculated by modeling the foundation as an equivalent raft.

Theoretically, the raft should be placed at the neutral plane and the soil compressibility must include the stiffening effect of the piles. However, placing the equivalent raft at the pile toe level is conservative and makes for faster calculations as time-consuming iterations become unnecessary. For small groups, it is necessary to include the effect that the load is also distributed to the soil from the neutral plane. A 5(V):1(H) is a realistic approximation of the stress distribution below the neutral plane to the pile toe. The analysis must, of course, also include other causes of change of stress in the soil level, such as fill and lowering of groundwater table.

Depending on pile spacing, large pile groups are normally less affected by the drag force developing in settling soil above the pile toe level, because neither the drag force nor the shaft resistance along a pile inside a group of piles can be larger than the weight of the soil in between the piles. Therefore, the interior piles in a group will receive a correspondingly smaller drag force as opposed to the perimeter piles. (The smaller drag force is often misguidedly termed "shielding" due to the perimeter piles. To achieve a "shielding" effect, the perimeter piles must not be connected to the foundation slab, or the drag force would become an added foundation load; likely an expensive and impractical solution to the "drag force issue").

Large pile groups where the soil is settling due to other causes than the pile supported loads will be affected by downdrag along the perimeter piles that will not affect the interior piles. This difference is beneficial as it will reduce the differential settlement between tank perimeter and center portion.

The average total settlement of a tank foundation and the distribution of settlement between the tank center and the perimeter is a function of the settlement caused by the tank supported load and that caused by area fill, groundwater table lowering and adjacent structures, e.g., the next tank over. The particular development for a specific case can be addressed by conventional settlement analysis applying the specific soil compressibilities, consolidation characteristics, and Boussinesq stress distribution.

It is unfortunate that so few well documented case histories are available in the literature with regard to long-term settlement monitoring of wide pile supported structures, such as LNG tanks. The geotechnical community can certainly learn and benefit from more well documented information. We hope that this situation will improve.

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