Pile Foundations as Settlement Reducer for Large Ms Storage Tanks

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ABSTRACT
This paper presents the performance of large diameter MS tanks supported on relatively short driven cast-in-situ piles. The settlement predicted for these 28m to 33m diameter tanks carrying a product load of 100000kN was of the order of 105mm at the centre and 80mm along the periphery. RCC common pile caps were designed for the expected differential settlements. Tanks were monitored during product loading as well as during hydro-tests and recorded 40mm to 110mm settlement. Performance of these tanks on piles was compared with that of large diameter storage tanks supported on ring wall foundation. These tanks on ring wall recorded about 155mm settlement under 60% of the load carried by tanks on pile foundation. Under equivalent load intensity, this was almost five times the settlement of the tank on pile foundation. Paper concludes that large diameter MS tanks can be supported on relatively short piles reducing the settlement significantly. There was significant saving by using 10m to 12m long piles instead of 26 to 30m piles.

1. INTRODUCTION
A soil profile comprising soft marine clay up to 2.50 m, a loose silty sand layer for another 2.50 m, then an interlace of medium dense to dense sand layer, layers of weak and moderately compact sandy clay and clayey sand layers for a large depth posed great challenge to the foundation designer of a gigantic fertiliser plant to be built in the early 1980s. A necessary fill and the weak clay at shallow depth ruled out the adequacy of shallow footings. While the medium dense to dense sand consistently available between 8.00 m and 13.00 m below the finished ground level offered a bearing stratum for pile foundations, very inconsistent profile comprising weak and moderately strong soil layers up to about 26.00 m was forcing the designer to select long pre-cast driven or bored piles for all the major units. It was decided to adopt relatively short driven cast-in-situ piles of 10 m to 12 m bearing them in the medium dense to dense sand after giving due considerations to the consequence of weak layers below. Here, large group of piles is acting as settlement reducer.

Detailed settlement estimations made for medium to large size pile groups resting in this sand layer concluded that even heavy structures and storage tanks could be supported on short piles. Pile foundation for 28 m to 33 m diameter storage tanks were designed for an expected settlement of 85 mm to 110 mm at the centre. These large pile groups were connected by RCC common raft for supporting the storage tanks.

This paper presents the performance of seven such large storage tanks. The performance of large pile group foundation is compared with the performance of the tanks supported on ring wall foundation directly on the soil. These tanks of relatively smaller diameter and with lesser load intensity and supported on ring wall foundation settled 50 to 70% more than those supported on piles.

2. IN THE LITERATURE
One of the oldest examples of pile foundations as settlement reducer was designed by Zeevaert (1957) for La Azteca office Building in Mexico City, constructed during 1954-55. A partly compensated foundation system recorded about 200 mm settlement at the end of construction, very close to the predicted values. The corresponding dished settlement profile was on the expected lines with about 30mm differential settlement. The expected settlement of foundation without friction piles was more than 500 mm.

Another important example is the Island Block, London, England, that was constructed in 1971-73 (Hooper and Levy, 1981). The structure had very large plan area, seven stories above ground and one basement. Large diameter piles placed directly below the columns and connected by a 225 mm thick common raft formed the foundation. The measured settlements were of the order of 15 mm to 30 mm with dished shape. The settlements were successfully predicted by approximate methods.

The reinforced concrete grain storage silos in Ghent, Belgium, constructed in 1976-1978 and reported by...
3. DESIGN OF FOUNDATION

The compressibility and shear strength profiles in the ammonia tank locations are represented by static cone resistance from standard Dutch cone of 37.5 mm diameter as shown in Figure 1.

![Cone Resistance Profile – Ammonia Tank 3](image)

**Fig. 1: Cone Resistance Profile – Ammonia Tank 3**

The variations in the cone resistance values between (−)3.0 m and (−)12.0 m levels made the selection of pile length extremely difficult. Three options were considered, viz., (i) terminating the piles between (−)5.0 m and (−)6.5 m, allowing the deeper weak layers to compress under incremental loads, (ii) terminating the piles in a depth range (−)9.0 m to (−)11.0 m allowing the remaining weaker layers to compress under the incremental loads and (iii) to terminate the piles at deeper levels such as (−)20.0 m to avoid the influence of weaker layers. By detailed settlement analysis for small and medium size pile groups, piles resting between (−)5.0 m and (−)11.0 m were adopted for various structures in the plant.

Typically, after detailed analysis, it was concluded that piles deeper than (−)6.50 m was not beneficial in the case of large storage tanks as the estimated settlement with piles resting between (−)5.0 m and (−)11.00 m were more or less the same. When the piles were shorter, the incremental stress in the shallow weaker layer was more and when the piles were longer, the deep seated weaker layers contributed more settlement.

Initially based on the cone resistance profiles, the pile depths in these tank locations were finalised and detailed settlement estimations were carried out. The settlement estimations were carried out using the compressibility parameter for sand layers suggested by De Beer (1948) as

\[ C = 1.5q / p_2^2 \]

where \( q \) is the cone resistance and \( p_2 \) is the effective overburden pressure at mid depth of the layer under consideration. Several researchers have reported that the compressibility parameter suggested by De Beer is very conservative and Meyerhof (1964) suggested a correction factor of 0.50 to the estimated value.

Conventional procedure of an equivalent flexible raft assumed at a depth equal to 2/3 pile length below the pile cap is adopted in the estimations. The initial load tests conducted on individual piles in the early stages of the project concluded that almost 60% of the design load was friction component. This justifies the load dispersion along the pile length and an equivalent raft at 2/3 pile length. Subsequent load distribution is obtained using Boussinesq’s circular area load solutions. Summary of such exercises is presented below in Table 1.

**Table 1: Settlement Estimation for Ammonia Tanks 1, 2 & 3 Supported on Driven Cast-In-Situ Piles**

<table>
<thead>
<tr>
<th>Tank</th>
<th>Estimated*</th>
<th>Most Probable with Meyerhof Correction</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>C</td>
<td>P</td>
</tr>
<tr>
<td>Ammonia Tank 1 (A)</td>
<td>139</td>
<td>89</td>
</tr>
<tr>
<td>Ammonia Tank 2 (A)</td>
<td>145</td>
<td>95</td>
</tr>
<tr>
<td>Ammonia Tank 3 (A)</td>
<td>167</td>
<td>102</td>
</tr>
<tr>
<td>Ammonia Tank 1 (B)</td>
<td>68</td>
<td>48</td>
</tr>
<tr>
<td>Ammonia Tank 2 (B)</td>
<td>80</td>
<td>51</td>
</tr>
<tr>
<td>Ammonia Tank 3 (B)</td>
<td>101</td>
<td>58</td>
</tr>
<tr>
<td>Ammonia Tank 3 (C)</td>
<td>343</td>
<td>202</td>
</tr>
</tbody>
</table>

* Using De Beer procedure. C and P, at centre and at periphery respectively

P(M) is periphery settlement considering pile raft interaction (A) with pile up to about (−)6.00 m, (B) with pile up to about (−)10.0 m, (C) with replacement of top clay and ring wall foundation

The above analysis does not take into account the possible load re-distribution by the interaction between the piles and the common pile cap (as a raft). The actual settlement along the periphery is expected to be closer to that at the centre reducing the difference between the settlements at the periphery and at the centre by about 50 percent. An estimate of periphery settlements as suggested by the designer is reported in the last column of Table 1. Similar estimations for a foundation with piles taken to (−)10.0 m level are also made and reported in the same table. As mentioned earlier, there is no significant improvement in the settlement conditions with longer piles. The above estimations are reproduced from the original foundation design note (Datye, 1983).

The ammonia Tanks were of 33 m diameter MS tanks supported on 437 piles equally spaced and connected by a common pile cap in the form of a thin raft. 450 mm diameter driven cast-in-situ piles with square spacing of 1.50 m c/c in both ways and resting in medium dense sand available...
on the piles was taken for two reasons. The first reason was to confirm more capacity to take care of the product load. The piles recorded 4mm to 5mm settlement where the load test was conducted to a load of 1253 kN, slightly more than twice the design load for 600 kN, the routine static load tests were conducted to a load of 1253 kN, slightly more than twice the design capacity. This decision to apply more load is also plotted in this figure. Even though the individual load test results were marginally less in the case of Tank 1.

The load settlement responses of five piles under routine pile load tests are presented in Figure 2. These piles are from the groups provided for three ammonia tanks.

The average elastic rebound after every load increment is also plotted in this figure. Even though the individual piles were designed for 600 kN, the routine static load tests were conducted to a load of 1253 kN, slightly more than twice the design capacity. This decision to apply more load on the piles was taken for two reasons. The first reason being very small settlements at 1.50 times the design load and loading up to twice the capacity will ensure adequate factor of safety for the friction component alone. The second reason was to confirm more capacity to take care of the hydro test load. The unit weight of liquid ammonia is 6.5 kN/m³ while the hydro-test load is about 1.60 times the product load. The piles recorded 4mm to 5mm settlement under the test loads and almost 40% of the settlement was reversed while unloading.

Five numbers of 28 m diameter phosphoric acid storage tanks and three numbers of sulphuric acid storage tanks, each of 100000 kN capacity, were also supported on short driven cast-in-situ piles. These piles also recorded similar sets while driving and the individual load test results were also comparable.

### 3. HYDRO TEST AND PRODUCT LOADING

The total product load from the liquid ammonia is only about 62% of the hydro-test load. Hence, the tanks will experience very small post hydro-test settlement during its product handing period. The outcome of the hydro test on three ammonia tanks is presented in Figure 3 in the form of load settlement response. Some inconsistencies in the load settlement responses presented in Figure 3 are due to unscheduled unloading and loading during hydro test that was necessitated by water requirements at the construction front. Typical settlement pattern along the tank periphery under different water load is presented in Figure 4. The tank was placed on a slab supported on series of columns in turn supported by the common pile cap raft. The space between the common raft and the base slab facilitated the measurement of settlement at the centre.
The 28m diameter phosphoric acid tanks are of 10.8 m height. The total product load is 1,00,000 kN resulting an average load equal to 17.9 m water column. This load intensity is equal to the hydro test load of ammonia tanks. Here, in the case of phosphoric acid tanks, the hydro test load is about 57.5% of the final product load. Initially the tanks were hydro tested to 10.3 m water column (57.5% of the total product load) and the results were compared with the settlements observed for the ammonia tanks. The load settlement response compared reasonably well and the tanks were allowed to be loaded with acid by incremental filling. The settlements were measured and the final load settlement plots for four tanks are presented in Figure 6.

![Fig. 6: Load – Settlement Response of Phosphoric Acid Tanks 1, 3, 4 and 5 under Hydro-test](image)

The tanks were emptied after the hydro-test and the acid filling was not possible with uniform increments as the product inflow was not uniform. However, the settlements were recorded after every incremental loading. The inconsistency in the load settlement response curves is because of the non-uniform loading and intermittent unloading cycles.

Comparison with Tanks on Ring Wall Foundation

Meanwhile two fuel oil tanks, both 22 m diameter and 10 m high and one filtered water tank, also of similar size, were constructed on ring wall foundation after replacing the top clay up to about 3.60 m below FGL. These tanks were hydro-tested and the settlements were measured at the centre and periphery. The settlement at the centre of the tanks was measured using three tube settlement marker locally fabricated for the purpose. The settlements at the centre and the periphery of the fuel oil tanks under 10.0 m water load are plotted in Figure 7 along with the settlement between the periphery and centre.

![Fig. 7: Settlements at Center and Periphery of Tanks on Pile Foundations and Fuel Oil Tank on Ring Wall Under Hydro-Test](image)

4. CONCLUSIONS

Estimations using conventional simple procedures are adequate for fairly accurate predictions of settlement of large pile groups supporting storage tanks. The interaction between piles and the common raft reduces the differential settlement between the periphery and centre.

Significant reduction in settlement can be achieved by adopting short piles overcoming the shallow weak layers, but by carefully deciding the pile lengths based on the subsequent weaker layers.

The subsoil condition in the present case is typical of sedimentary deposits along the coastal regions and the design approach has wide scope for application.

REFERENCES


