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A case study of large screw pile groups behaviour

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ABSTRACT An instrumented pile test loading and the monitoring during operation of three large pile group foundations for oil tanks in Ostend (Belgium) is presented. The case study gives a comprehensive example of all the critical issues related to the assessment of pile group settlement behaviour, based in the tertiary sand underlain by a 100m thick O.C. clay layer. A fully instrumented single pile test load, which was performed at the site, will be discussed here. Soil characterization out of the CPT testing campaign does show a very heterogeneous fill consisting in very soft clayey/silty material up to depth of 20m and underlain by a layer of about 5m of very dense tertiary sand as the actual bearing layer. It will be shown how the local heterogeneity of the fill may give rise to slightly differences in the deformations and direction of the tilt of the tanks. The results in terms of "immediate" settlement out of the short period of the hydro-test preceding the start of tanks' operation are reviewed. Long term settlement prediction is done assuming deformation parameters derived from the CPT data and using the equivalent raft method. A more accurate estimate of the long term behavior of the construction will go out from the additional settlement-monitoring data during the operation's phase and using a simplified 3D elastic soil model.

1 THE OIL TANKS' FOUNDATION ENGINEERING PROBLEM AT OSTEND

1.1 General soil conditions at the site

At the Ostend oil tank site (Fig. 1), the soil conditions, up to depths of roughly 12 to 15 m can be described as very heterogeneous fill. The whole area has indeed been excavated and hydraulically refilled over decades before reaching the today's new destination of the site as an oil tank plant.

It means that below some thin or weaker quaternary soil lenses (from about 12–15 m up to depths of about 18 m) the first natural bearing layer would be the tertiary sand. This sand layer is about 4 to 5 m thick and it is underlain by a silty clay of the Tielt formation which reaches down to the depth of 45 m as suggested by geological data. Underneath this silty clay layer a large overconsolidated clay layer (Kortrijk formation) is appearing, reaching depths of 170m.

The large number of CPT tests which have been performed at the site confirms the heterogeneity of the fill and the quite regular location and thickness of the tertiary sand. Unfortunately none of these CPTs was reaching large depths. This makes impossible to define the actual thickness of the silty clay layer and furthermore to have any data available on the O.C. clay layer of the Kortrijk formation.

Typical q_c profile on the site and corresponding soil behaviour type classification SBTn out of CPT testing (Robertson, 2010) are shown in Figure 2, respectively with a white and a yellow line.

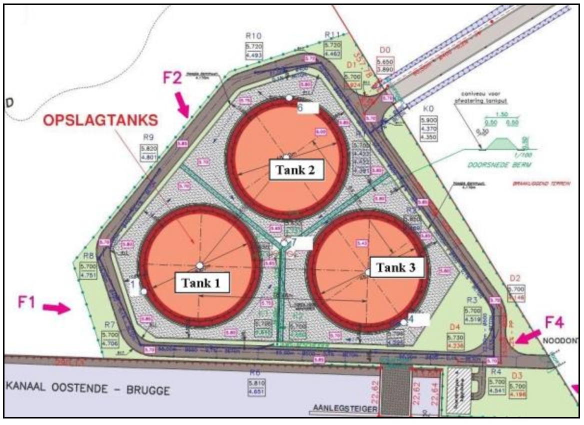

Figure 1. The site and location of the three oil tanks at Ostend (Belgium).

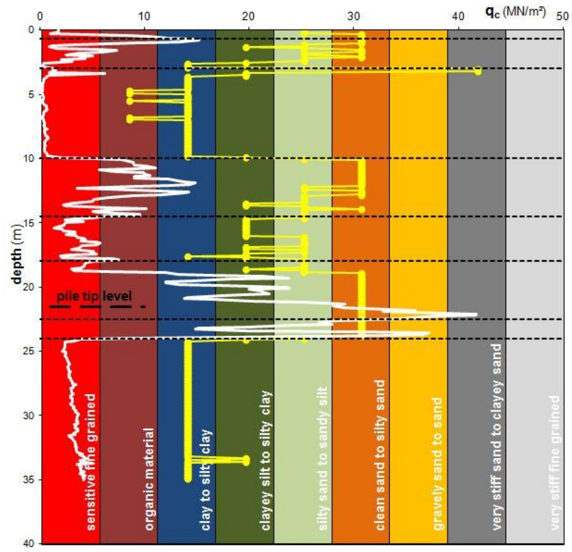

Figure 2. CPT based soil type classification at the location of the test pile axis (q_c profile : white line ; SBTn : yellow line, Robertson 2010).

1.2 The oil tank foundation design

The three oil tanks (Fig. 1) each containing 33000 m^3 are steel structures of 48 m in diameter and 19 m height, positioned in a triangular shape at a center to center inter-distance of about 65 m. They are founded on a 48.8m diameter, 60 cm thick reinforced slab, supported by 422 displacement screw piles.

The 460 mm diameter displacement screw piles of the Omega pile type reach to a depth to of 21.5 m and are placed at an inter-distance of 2.2 m (centre-to-centre), so with a spacing higher than 3 times the diameter of the pile as recommended and well known from literature. They have been designed, with a global safety of about 2, according to the Van Impe-De Beer method (1986) to each take a maximum design load of 960 kN, including some 180 kN of negative skin friction load which originates from the reconsolidation of the soft layers along the pile shaft (Van Impe et al., 2013).

The foundation design of the oil tanks can actually be considered as a foundation on a large group of 422 end bearing displacement screw piles, cast in situ, until about 22 m of depth.

2 INSTRUMENTED SINGLE PILE TEST LOAD

2.1 Measured load-settlement data of the test pile and corresponding capacity discussion

In order to optimize the design method, a fully (extensometer) instrumented test pile was installed to be test loaded up to a pile base settlement of 10% of the pile base diameter, which in accordance to the Belgian practice corresponds to the required deformation at “failure load” of a soil displacement pile.

The measured data show that the large deformations (>10% of the pile tip diameter) of the pile tip, with subsequent ongoing pile tip displacements do appear at pile head load levels in between 3300 kN and 3600 kN, which was found to fairly well correspond to the CPT-predicted “failure” load of about 3.5 MN according to the Van Impe-De Beer method.

However, even with the ultimate test pile load corresponding quite well to the predicted values, the measured distribution of the ultimate pile tip load versus the ultimate pile shaft load appeared to be quite different from the predicted one.

The lower tip bearing capacity mobilized during the pile test load points to a bad execution of the test pile and to the presence of residual stresses (Fig. 3). The weak soil-pile tip interaction at the base is due to the long reinforcement cage with inner extensometer cage, both fitting only narrowly inside the rather small diameter opening left open by the re-screwed auger head during the casting of the concrete. However such unsatisfying soil-pile tip interaction is not of concern when coming to the installation of the real tank piles, where the reinforcement cage is never reaching the pile tip level and there is of course no supplementary cage for the instrumentation unlike the pile test.

The zero readings of the instrumentation in the pile, after the residual strains were developed, do obviously influence the interpretation of the load-settlement curves of a pile assumed to start from zero initial stress levels at zero pile head loading.

In Fig. 3 the full blue line indicates the test pile load distribution during the test loading at a level of 3005 kN pile head load (of the assumed not pre-stressed pile). The violet dashed line indicates the estimated negative skin friction developed along the pile shaft due to reconsolidation of the soil layer in between 1 m and 17 m depth. This is, consequently, the new zero-reference line for the extensometer readings. Therefore the actual corrected pile load distribution curve would be the full green line in Fig. 3.

However, the dashed violet line is calculated starting from the assumption of a perfect soil-pile tip interaction, which is not necessarily the case for this instrumented test pile. So, the dotted lines in Fig.3 suggest other possible less perfect soil- pile tip interaction. Only lab testing derived stiffness of the soil underneath the already installed pile tip, would lead to more reliable outcome of the pile tip interaction stiffness required to optimize the interpretation of the measured load-distribution curve out of the pile load test.

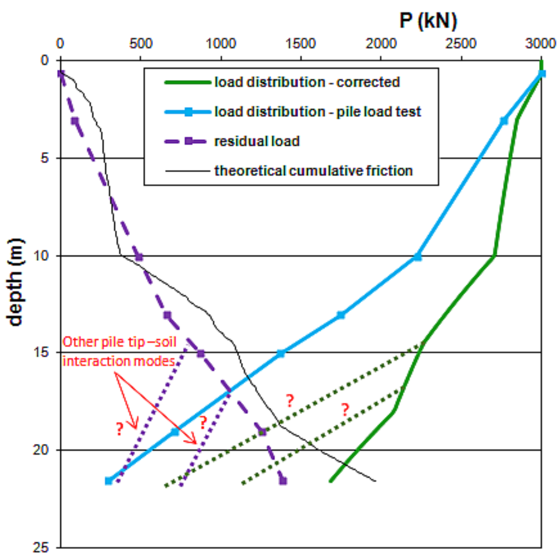

Figure 3. Corrected test pile load distribution implementing the residual stress induced loading (Van Impe et al.,2013).

Anyhow, the pile load test shows clearly that the total ultimate pile load is satisfying largely the design load and also confirms that the expected load settlement stiffness at design load satisfies the criterion of pile tip deformations of about 3 mm at 1.5 times the design load.

2.2 *Settlement estimation*

An initial settlement estimation was done for a single loaded tank using the method of the equivalent raft. According to Tomlinson (1986) the pile group should be replaced by an equivalent raft located at a depth which depends on the nature of the soil profile and ranges from 2⁄3L for friction pile groups to L for groups bearing on rock, where L is the pile length. The load is spread at an angle which varies from 1 in 4 for the former case, to zero for the latter case. We considered however an equivalent raft in between these two cases, as a function of the average ratio of pile shaft to pile tip capacity: we assumed an equivalent raft at 18.20 m depth, with an equivalent outspread diameter of 52.26 m and under an equivalent load at that depth of about 152 kPa. Once the equivalent raft has been established, the settlement can be computed by considering a normal shallow foundation analysis.

The values of the constrained modulus (M) which have been used within this analysis (Table 1) were derived from CPT results (Fig.2), using the following empirical relationship by Robertson (2009):

$M = \alpha_M (q_t - \sigma_{v0})$;

where α_M varies with the cone penetration resistance Q_t and the Soil Behaviour Type Index I_c (for $I_c > 2.2$: $\alpha_M = Q_t$ if $Q_t < 14$ and $\alpha_M = 14$ if $Q_t > 14$; while for $I_c < 2.2$: $\alpha_M = 0.0188\ [\ 10^{(0.55Ic + 1.68)}]$).

Table 1. Estimated stiffness parameters from the CPT result at the test pile location (Robertson, 2009).

Depth (m)	q_c (MPa)	M (MPa)
0.66	4.60	40
3	0.66	5
10	10.07	97
14.5	5.28	66
18	15.80	143
21.56	35.68	282
22.48	26.18	209
24	3.3	27
35	5.23	51

For the OC clayey layers below 35m depth, for which we had no data, we assumed a typical constant value of the constrained modulus of about 150 MPa.

The equivalent raft method with the hypotheses described so far, predicted 91 mm of final settlement under the centerline. However, such prediction is very much depending on the actual compressibility of the unknown clayey layers below 35 m depth and settlement monitoring of the tanks was therefore deemed to be essential.

In addition the interaction of the 3 tanks' loading will lead unavoidably also to a settlement trough.

3 OBSERVED TANK SETTLEMENT BEHAVIOUR

3.1 Tank settlement during hydro-testing

Each tank is being monitored on 16 points along its perimeter, equally divided at an center angle of 22.5° from each other.
The settlement measurements along the perimeter have been analyzed in terms of an average settlement (average of the 16 points), a best fit plane, the size and direction of the rotation of the tank (tilt), out-of-plane settlements (deviation from the best fit plane) and out-of-plane deflections (distortion).

During the hydro-test, all tanks were filled with water to a height of 18m. The filling of each tank took about 3 days and the water level in the tanks was then kept constant for about 4 days. The emptying procedure of the tanks took then again a period of 3 days.

The measured tank settlement is the combination of the settlement of the foundation (raft and pile group) and the compression of the asphalt layer. The latter was established to be about 3mm, while the settlement of the foundation was 20 to 21mm under a load of 180 kPa.

As each tank was tested separately and for a very short period, the impact of the load is presumably limited to the immediate response of the stiff sand layer in which the pile group is resting and the upper part of the underlying silt clay layer.

The tilt of the tanks during this procedure (2-3mm) was as well quite limited because there was no real interaction between the tanks.

Analyzing the response of the separate tanks under hydro-test loading allows to estimate the immediate compressibility parameters of the sublayers. We considered each pile group foundation below each tank as equivalent to a raft foundation of same diameter of the tank and with an embedment equal to the pile length. The SteinP 3DT (simplified 3D elastic model) program by Geologismiki, allows to calculate the deformation according to Boussinesq stress distribution for a flexible raft: from a back analysis of the measured settlement during hydro-test we could estimate then a value of 200 MPa for the sand layer and 230MPa for both the underlying silty clay and the thick OC clay layer (although the impact of the hydrotest of a single tank probably does not reach deep enough to really influence this layer).

At the end of the hydro-test, after fully emptying the tanks, the residual average deformation was about 8mm for all tanks.

3.2 Tank settlement during operation

The tanks have been in operation since July 15th 2013. The filling of each tank took about 2 months: in a timeframe of 6 months, all tanks have been filled with diesel, increasing the load on the foundation to 145 kPa.

A first measurement of the settlements occurred right after the filling of tank 3 (January 2014) and a second measurement took place 8 months later (September 2014). The loading sequence of the tanks and the average settlement as a function of time are shown in Figure 4. The additional average settlement during the operational phase at this point has reached values of 34 to 40mm. As we would expect this settlement is higher than the one during the hydro-test, because of the increased stress field and the onset of consolidation.

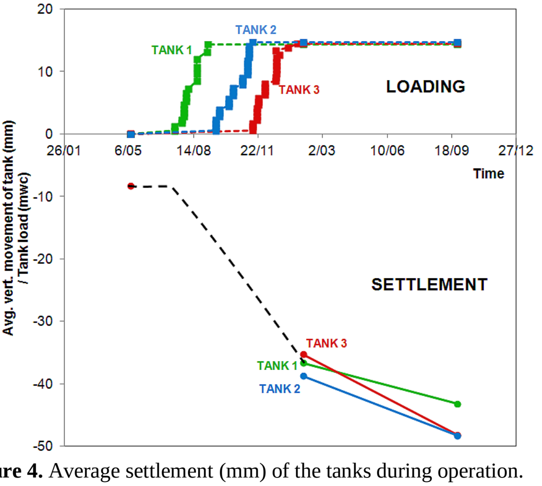

Figure 4. Average settlement (mm) of the tanks during operation.

Figure 5 shows the vertical deviation from the average value for each measurement point. Points with negative values (outside the smaller circle) indicate points which have settled more than the average. The shape of the curves therefore indicate the size and the direction of the tilt of the tanks.

A higher value of the tilt compared to the situation of the hydro-test is also to be expected due to the interaction of the different loads. Both tank 1 and 2 exhibit a 12-13mm of nearly perfectly planar tilt (0.00026 m/m) towards the central area in-between the tanks, while tank 3 tilts clearly slightly distorted and almost directly north for about 20 mm (0.0004 m/m). The local subsoil heterogeneities below tank 3 are probably the reason for such uneven tilt of tank 3.

Akhavan-Zanjani (2009) reviewed a large number of case studies and references related to allowable settlement criteria for steel tanks: for large tanks, as in Ostend, Klepikov (1989) would recommend for example a maximum allowable differential settlement of 0.004D and an ultimate tilt of 0.007H (where D and H are respectively the diameter and the height of the tank).

The values of average settlement, tilt and distortion measured up to now are therefore still far below critical values.

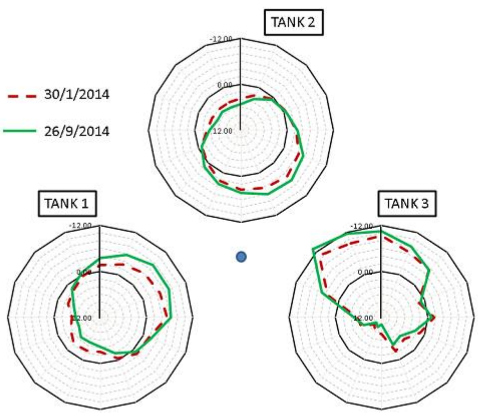

Figure 5. Vertical deviation from average settlement (mm) during operation.

4 ANALYSIS OF THE TIME SETTLEMENT BEHAVIOUR

The settlement of the full pile group is only marginally influenced by the fill material. The important characteristics are those from the dense sand layer, the silty clay and, in the long term, the thick OC clay layer. Unfortunately, as mentioned before, the data on the two latter are very limited. Authors therefore have attempted to analyze the current settlement data in order to predict the long term behaviour of the construction.

Based on on the compressibility parameters out of the CPT data and the immediate compressibility parameters derived from the fast hydro-test, a single value of the vertical consolidation coefficient c_v for the silty clay was calculated to obtain the best fit between predicted and measured average settlements.

The prediction was done using a simplified 3D elastic stress model (again using the SteinP 3DT program). Referring to some representative grid-points, we have first considered the deformation with time from the loading of each separate tank and then we have added together the three different distributions in accordance to the superposition effect. Of course, when adding together the settlement distribution of each tank, we took into account the fact that the three tanks have not been loaded all at the same time but in a timeframe of 6 months. The model takes also into account the slight lower thickness of the sand layer below tank 3. The ultimate average settlement for the tanks ranges from 87 to 90 mm and the centers of the tank settle 132 to 136 mm (Figure 6). The long-term tilt ranges from 19 to 21 mm.

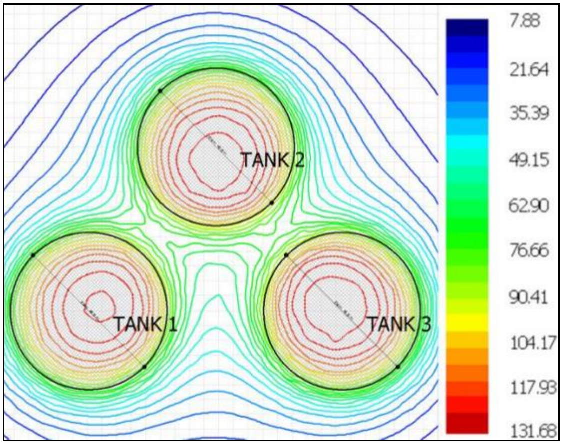

Figure 6. Simplified settlement analysis of the tanks under operational load.

As the actual size of the consolidating layer is unknown, fitting was done on the basis of the combined c_v/d^2 parameter (where d is the drainage path length).

This leads to a value of the time factor c_v/d^2 of 0.0022 $month^{-1}$.

Figure 7 shows the predicted versus measured values for the average settlement. The predicted shape will be initially different for the three tanks because they have been filled at a different time. The time range which is considered is approximately 20 years.

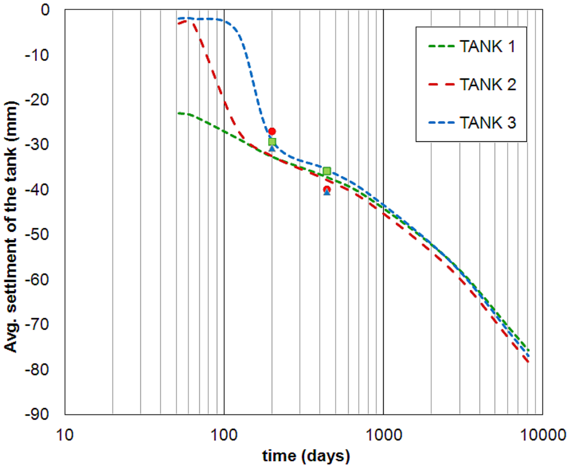

Figure 7. Predicted (dashed lines) versus measured (dots) average settlement of the tanks under operational load.

Such prediction underestimates however the amount of tilt which is occurring, especially for tank 3. This could be due to the existence of some stiffness heterogeneity in the region of this tank.

Further modelling would require additional soil data to increase the accuracy of the soil model and higher-level software capable of taking into account complex soil layering.

5 CONCLUSIONS

A fully instrumented screw pile test load, performed within the design stage of the oil tanks' foundation at Ostend, is presented. The pre-stressing of the test pile during its installation has been highlighted to explain the difference between predicted and measured single pile capacity.

The data of the hydro tested oil tanks are reviewed to illustrate that the end bearing displacement screw pile group underneath each tank, although in an almost 18 m of very heterogeneous soil layering can guarantee a quite uniform settlement for each tank to a very similar level.

During the hydro-test, the settlement trough starts already to develop by the mutual interaction of the 3 tanks at short distance from each other. The monitoring data during operation show anyhow such interaction much more clearly as the combined loading of the three groups gives rise to larger settlements than those measured during the separate loading of the hydro-tested tanks. Moreover, a significant increase of the tilt of the tanks occurs as well during operation.

Due to the large scale of the combined constructions, the influence depth is considerably larger than the extent of the soil investigation. Authors have made an attempt to analyze the data obtained during the hydro-test and the current operational stage to make quite reliable estimate on compressibility and consolidation coefficients of the OC clay layers (35 m below tank's foundation) which will govern the long term settlement.

The obtained values lie within the normal range for these type of soils, and allow further extrapolation of the current measurements. Additional measurements campaigns will take place in the next months to allow further optimization of the model.

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