Beneficial use of Pressuremeter Tests to optimize the Slab foundation design of a Biogas Plant in Germany

Utilisation Préférentielle du Pressiomètre pour optimiser la conception des fondations d'une usine de biogaz en Allemagne

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ABSTRACT

This paper presents a case study of a biogas plant in Germany. Based on the original soils report, the ground was considered marginal based on DPM and later CPT. With the estimated soil parameters, a shallow foundation design for the tanks was not feasible. The settlement prediction for the high loaded tanks was in the intolerable range of 40 to 50 cm. Thus, the typical solution was an expensive thick reinforced slab supported by deep piles.
To optimize the foundation design, the Menard pressuremeter (PMT) was carried out with the aim of using a direct design procedure for predicting the bearing capacity and settlements of the tanks. Based on the PMT results a shallow foundation on Controlled Modulus Columns (CMC), a rigid inclusion network (RI) with load transfer platform, could be designed to reduce the settlement on an acceptable limit. The additional benefits of the shallow foundation approach were an optimised slab thickness of 25 cm and lower costs for the foundation works. Shallow foundations provide adequate support for steel tanks, and they are cost effective compared with conventional deep foundations using piles. However, to realise that kind of optimization, appropriate and reliable soil parameters for the design are needed. The RI design in this case was done with load transfer curves to model the rigid inclusion behaviour, as suggested by the Frank and Zhao (1982). The input parameters for this model are determined from Menard pressuremeter tests. The prediction of settlements considering the RI treatment shows good agreement with measured settlements obtained during hydrostatic (full scale) load tests after completion of the works.

RESUME

Cet article présente une étude de cas d'une usine de biogaz en Allemagne. Dans le rapport géotechnique, le sol a été considéré comme compressible sur la base du DPM et, plus tard, du CPT. Avec les paramètres estimés du sol, il n'était pas possible de concevoir des fondations superficielles pour les réservoirs. Les prévisions de tassement pour les réservoirs fortement chargés se situaient dans une fourchette inacceptable de 40 à 50 cm. La solution de base était donc une dalle armée épaisse et coûteuse soutenue par des pieux profonds.
Afin d'optimiser la conception des fondations, le pressiomètre Menard (PMT) a été réalisé dans le but d'utiliser une procédure de conception directe pour prédire la capacité portante et les tassements des réservoirs. Sur la base des résultats du PMT, une fondation superficielle sur sol renforcé par des colonnes à module contrôlé (CMC), un réseau d'inclusions rigides (RI) avec une plate-forme de transfert de charge, a pu être conçue pour réduire le tassement dans une limite acceptable. Les autres avantages de cette approche sont une épaisseur de dalle optimisée de 25 cm et des coûts réduits pour les travaux de fondation. Les fondations superficielles fournissent un support adéquat pour les réservoirs en acier et sont plus économiques que les fondations profondes conventionnelles utilisant des pieux. Cependant, pour réaliser ce type d'optimisation, des paramètres de sol appropriés et fiables sont nécessaires pour la conception. Dans ce cas, la conception de l'IR a été réalisée avec les courbes de transfert de charge de Frank et Zhao (1982) pour modéliser le comportement de l'inclusion rigide. Les paramètres d'entrée de ce modèle sont déterminés à partir d'essais pressiométriques Menard. La prédiction des tassements en tenant compte du traitement RI montre une bonne concordance avec les tassements mesurés obtenus lors d'essais de charge hydrostatique (à l'échelle réelle) après la réalisation des travaux.

Keywords: Rigid inclusions; CMC; load transfer curves; tank foundation; PMT testing, interpretation and design.

1. Introduction

Industrial tanks are usually large and heavily loaded structures that are used for storing fluids. Given that tank storage structures are constructed using relatively thin walls, floors and roofs, unforeseen ground deformations can have undesired impacts on the performance of tanks.

Steel tanks of biogas plants are generally high and relatively simple structures. They are flexible and ductile. They can tolerate settlements without loss of structural integrity. These tolerable settlements, however, are not without limits. Excessive settlements may affect the serviceability of the tanks. Steel tanks are comparatively sensitive to edge failure and differential settlements around their shell.

In cases where predicted settlements are considered intolerable, the basis solution is usually an expensive and CO_2-consuming thick reinforced slab supported by deep piles. The use of such foundations is usually based on overdesign due to poor or inaccurate soil parameters, together with the use of rough estimations or multiple correlations. With more accurate soil parameters, it is possible to optimize the tank foundation design, leading to a cost-effective foundation solution.

Pressuremeter tests could be used for the design, for optimization and for quality control of various ground improvement techniques under several tank projects (Yee and Varaksin 2007; Hamidi and Varaksin 2017). For soft soils and high loads, rigid inclusions using Controlled Modulus Columns (CMC) is a well-established technique for ground improvement. This type of ground improvement will be assigned to class BII of the new Eurocode 7-3 and can be reliably designed with pressuremeter data (Racinais et al. 2017).

1.1. Project Overview

The biogas plant in lower Saxony, Germany, is one of the largest of its kind in Europe. It consists of five steel tanks, five concrete tanks and a 5,000 m² hall for storing the biodegradable substrate before the fermentation process.

Figure 1. Project picture, before final installations.

The main method used to improve the poor ground was the CMC method under all high loaded areas (tanks and foundations of the storage hall). Menard Rapid Impact Compaction (M-RIC) was used under areas with low surface loads (traffic and light loaded areas).

The focus of this paper are three steel tanks with dimensions of 24 m in diameter and a height of 19 m. These are fixed-roof tanks with a design distributed load of 190 kPa. The settlement criteria defined by the Client was 100 mm and 1/100 of differential settlement.

Hydrotests were done up to full fluid hight for two reasons: to check for leaks within the shell and to induce or preload the settlement before the final installation of the piping system. Thus, contrary to most structures whose actual performances under full loadings are only when the structure is put into service, the tanks are fully loaded with water before making the final connections of the piping system.

1.2. Ground Conditions

In the first step, the soil was investigated with 6 m deep core drilling, dynamic penetration with a drop hammer of 30 and 50 kg (DPM: dynamic probing medium, DPH: dynamic probing heavy) and CPT (Cone Penetration Test).

The soil below the organic topsoil is characterised by alternating layers of compressible alluvial loess (sa'cl'Si) and fluvial sands (siSa), which are underlaid by boulder sands and clays with varying proportions of silt and clay. At depths of 11 to 19 metres, dense glacial sands and gravels with high bearing capacity were found.

However, the excavation depths of the core drilling and the DPM could not reach this load-bearing sand. The DPH reached the sand. Nevertheless, the results of the DPH are highly influenced by friction along the rod string. The results show an increasing number of blows in the compressible soil layers. This is illustrated by the comparison of the three types of soundings in Fig. 2. In comparison, the CPT has a cone resistance of q_c = 1-3 MPa. N10 and q_c do not correlate in your fine-grained soils. The use of DPM and DPH are therefore severely limited to determine appropriate soil parameters and layers for the ground model.

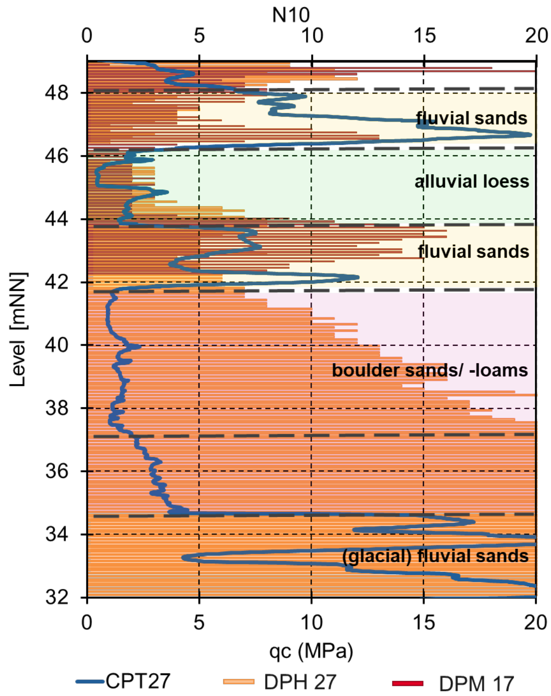

Figure 2. Comparison of DPM, DPH and CPT.

2. Pressuremeter results

The Menard pressuremeter was carried out under each tank for two reasons:

- The measured data could be used to model the soil model under each tank accurately
- Determination of skin friction and tip resistance for the RI design using Eurocode 7-2 Appendix E or NF-P 94-262 (2012), see section 3.

2.1. Comparison of deformation modulus

The deformation modulus of the soil can be determined from pressuremeter test parameters using various approaches. A common method is the correlation of the vertical deformation modulus from the Menard modulus E_M, linked by the structural coefficient α according to NF P94-261, see Eq. (1) and Tab.1. This relationship is valid only for an ideal raft or slab application case with no stress distribution under the foundation.

$$E_{oed} = \frac{E_M}{\alpha} \tag{1}$$

Table 1. Structural coefficient α (NF P 94-261)

Soil type	**Peat**	**Clay**	**Silt**	**Sand**	**Gravel**
overconsolidated or very dense	-	1	⅔	½	⅓
normally consolidated or normally dense	1	⅔	½	⅓	¼
overconsolidated weathered or loose	1	⅔	½	⅓	-

Another approach is described by Hoang et al. 2018, finetuned and validated in Hoang et al. 2020. The value of deformation modulus is dependent on the strain of each layer, which means the structural coefficient α depends on the layer thickness and the pressure. For a given soil type, it is therefore assumed that the ratio E/E_M depends solely on the deformation. The advantage of this approach is that it is 'intrinsic' and independent of the shape or stiffness of the foundation. It can therefore be extended to the study of any foundation (isolated or grouped footings, raft or slab).

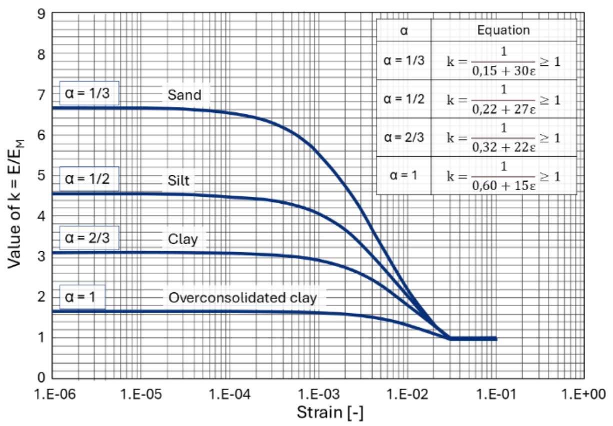

Figure 3. $E/E_M = f(\varepsilon)$ according to Hoang et al. 2018.

The corrected cone tip resistance q_t (CPT data) can also be used to estimate the deformation modules in soils according to the Eq. (2). α_M is an empirical factor, which varies from different authors.

$$E_{oed} = \alpha_M \cdot (q_t \cdot \sigma_{vO}) \tag{2}$$

Robertson (2009) suggested that α_M varies with normalized cone penetration resistance Q_t. Estimates of drained constrained modulus from undrained cone penetration is approximate. Estimates can be improved with additional soil information, such as plasticity index and natural water content.

The soil report gives also values for the oedometric modulus of the different layers based on rough estimations and values from literature.

The results from different test methods for deformation modulus under tank A are shown in the following Fig. 4 following the correlations mentioned above.

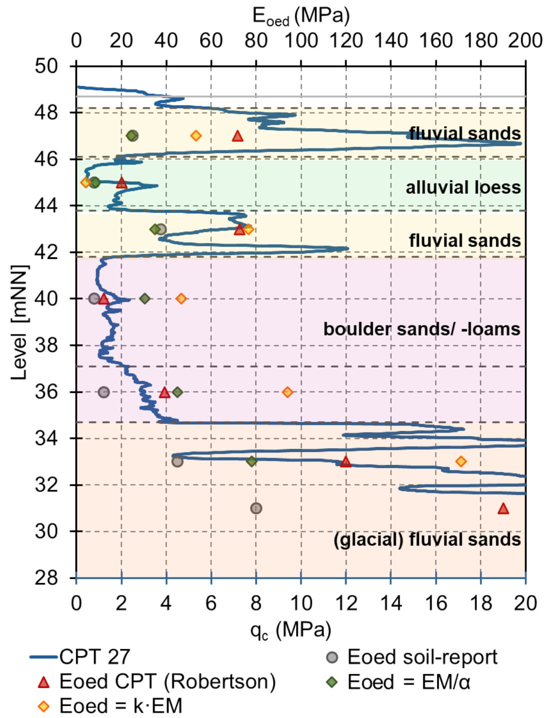

Figure 4. Comparison of the deformation modulus based on CPT 27 and PMT 27.

The profile shown is located at the edge of tank A, where a CPT 27 and PMT 27 were carried out very close to each other.

The different approaches show a wide range of the deformation moduli for the individual soil layers. However, it was possible to establish with certainty that the values stated in the soil report were too conservative and that any more precise investigations would result in a better design for the project. With the knowledge gained from the pressuremeter, it was possible to design and optimise the ground improvement solution.

3. Ground Improvement with CMC

3.1. CMC execution and Quality Control

In the early 1990s, Menard developed the innovative concept of Controlled Modulus Columns (CMC). This technology consists of a rigid inclusion network installed in the soil down to a load-bearing layer and disconnected from the above structure (often by the mean of a load transfer platform (LTP)): both inclusions and surrounding soil participate to the foundation system. Using CMC brings therefore similar effectiveness as piling, with the elegance of structural support on a shallow foundation.

In this project CMC elements with a diameter of 320 mm were arranged in a grid under the tanks. In total more than 55 km CMC were executed within 4 months with only one rig. As LTP, a recycled crushed concrete pad was used with a thickness of 60 cm.

During full production works, operation parameters were recorded which include (i) the depth of the columns; (ii) drilling energy; (iii) concrete consumption; (iv) torque during penetration and (v) execution time.

Figure 5. CMC execution for the tank foundation.

Under each tank several load tests on isolated single columns (SLT) were carried to verify the predicted inclusion behaviour, see section 3.2. To verify end bearing capacities, five additional dynamic load tests were done on site.

3.2. CMC Design

3.2.1. Semi-empirical mobilization laws of Frank and Zhao

To characterize the soil-structure interaction in a soil reinforced by columns, Frank and Zhao (1982) proposed two semi-empirical laws for mobilization of skin friction along the column and for end-bearing at the tip of the column. This model is based on pressuremeter data. The skin friction mobilization law is defined according to the relationship between the shear stress τ and the relative displacement s_s (between the rigid inclusion and the soil around the shaft of the column). This law depends directly on the limit value of skin friction $q_{s,ult}$ which could be correlated from the limit pressure p_L according to Eurocode 7-2 Appendix E or NF-P 94-262 (2012). The end-bearing mobilization law is defined according to the relationship between the stress at the column toe q_b and the vertical displacement at the inclusion tip s_b in the bearing layer. The end-bearing resistance $q_{b,ult}$ could be also correlated from the limit pressure p_L.

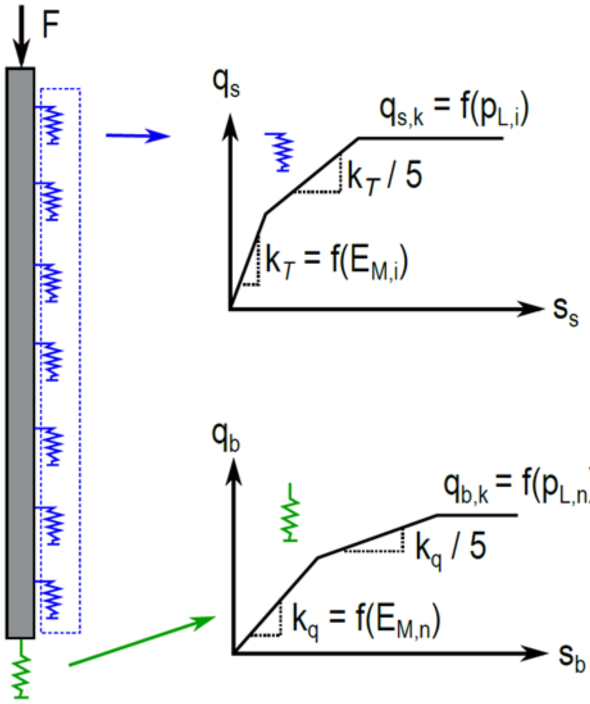

Figure 6. Mobilization laws for RI according to Frank and Zhao 1982.

The mobilization curves of skin friction and tip resistance are determined according to the model shown in Fig. 6 as a function of the pressuremeter modulus E_M.

The values for the coefficients m_q and m_τ depend on the soil type. For coarse-grained soils, $m_q = 4.8$ and $m_\tau = 0.8$. For fine-grained soils, $m_q = 11.0$ and $m_\tau = 2.0$ are used.

3.2.2. Unit Cell Calculation

The design of the tank slabs over RI with load transfer platform (LTP) based on the analytical model MV2 described in the recommendations ASIRI. It is a two-phase model which examines the interaction between the domain of inclusions and the surrounding soil in a unit cell. The calculation is iterative. The first iteration assumes a distribution of the load between the two domains, which enables to assess the settlements directly below the footing in each domain. The soil settlement based on pressuremeter modulus E_M. In the following iterations the load distribution is varied to reach the equality of the settlements directly below the slab. The two domains are linked by load transfer curves according to section 3.2.1. The results of the unit cell calculation are as follows and shown in Fig. 7 for tank A:

- distribution of settlements within the soil and within the RI,
- distribution of the axial load and the mobilized lateral friction between the soil and the RI

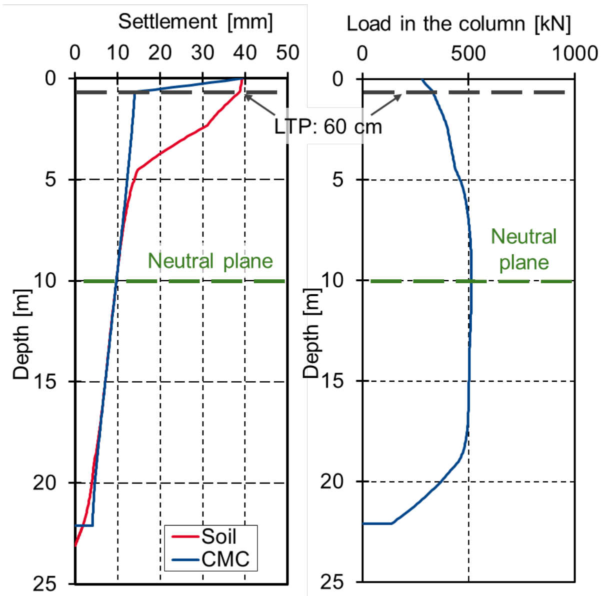

Figure 7. Results of unit cell calculation for Tank A.

For tank A the calculation shows that the column head punches about 25 mm into the LTP, what means more than 50 % of the total predicted settlement. The most part of soil deformations takes part in the alluvial loess layer. Up to 10 m below surface the deformation of the surrounding soil is predicted larger than the column settlement, which results in negative skin friction. Below the neutral plane positive skin fiction is activated and the load within the column decreases.

3.3. Design Parameter Verification with SLT

The prediction based on Semi-empirical mobilization laws by Frank and Zhao (1982) usually shows good agreement with settlement behaviour measured during load tests on CMC. This is shown in the studies of Racinais et al. (2017) and Tinat et al. (2019).

To validate the settlement behaviour of the single column, which means the dataset of q_s and q_b for the CMC under each tank, single load tests (SLT) up to 600 kN (60 tons) were applied to the column head. The results were compared with the predictions based on PMT results and the mobilizations laws.

The test loads were carried out using the drilling rig as an abutment. For this purpose, the carrier with its crossbeam was moved centrally over the test column. A hydraulic cylinder was positioned between the crossbeam of the drilling rig and the CMC.

The measuring equipment is like a classic Benkelmann beam. The load plate is placed on the CMC head. The load is gradually increased by the hydraulic cylinder and measured by means of an intermediate load cell. The displacement of the column on the column head is measured using a Benkelmann beam. The test load setup is shown in Fig. 8 below.

Figure 8. Single Load Test on CMC from Top of the LTP.

The comparison of one static load test and prediction for Tank A is shown in Fig. 9. The measured curve shows less settlement than predicted, what means a light stiffer column behaviour. However, both curves show settlements in the small strain area of the load settlement curve and the difference between measurement and prediction ranges only between 2 and 4 mm.

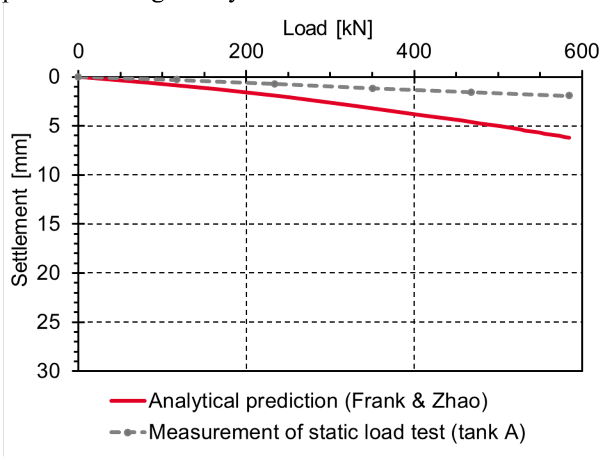

Figure 9. Comparison of Single Load Test (SLT) and prediction under tank A.

4. Settlement results during Hydrotest

Hydrostatic load tests were carried out about 4 months after completion of ground improvement works. The height of water at full load was maintained at 18.5 m for minimum 10 days depending on the final leak-proof and sealing works at the tank shell.

Calculations for settlement considering the CMC were carried out based on the location of the PMT tests. At each test location, settlement calculation was carried out using the procedure described earlier (Eq. (1) for deformation modulus and Frank and Zhao laws for rigid inclusion behaviour). The predicted settlement for tank A, tank B and tank C are summarized in Tab. 2 below. For tank A, the PMT in the centre of the tank is used for the settlement calculation.

Table 2. Predicted settlements based on PMT and Frank and Zhao model and measured settlements during hydrotest

	Tank A	**Tank B**	**Tank C**
From PMT predicted settlement	39	43	43
From hydrotest measured s_{edge} (mean values)	22	24	23
From hydrotest measured s_{center}	39	46*	45*

*) estimated from measurements on the slab after emptying the tank

Each tank was equipped with six settlement sensors at the edge of the tank. This made it possible to detect any potential tilting of the tanks. The final settlement around the tank shell during the hydrotest is shown in Fig. 10. The measured settlement at the tank shell (s_{edge}) for tank A, tank B and tank C ranged from 18–26 mm, 16-29 mm and 11–30 mm respectively.

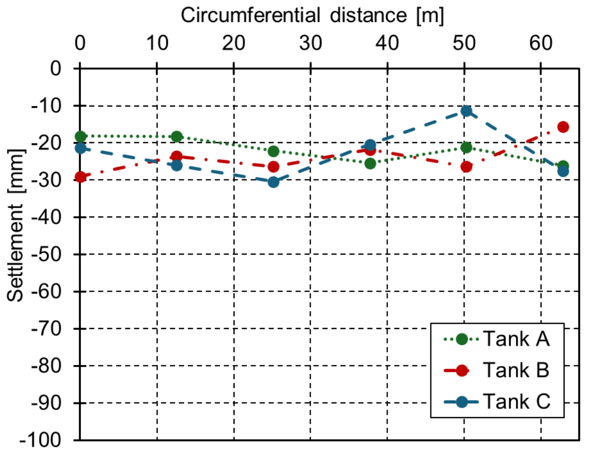

Figure 10. Settlements during hydrostatic loading measured at edge of tank A, B and C.

The mean settlement (s_{edge}) was 22 mm for tank A, 24 mm for tank B and 23 mm for tank C. The maximum differential settlement around the tank shell was found at tank C which was 16 mm between 2 adjacent measured points over a distance of 12.6 m. This is equivalent to 1/800 which satisfied the recommendations of 1/100.

Further measurement equipment was installed under tank A, why tank A is focussed to in the following.
The settlement of tank A as a function of the filling of the hydrotest and the time is shown in Fig. 11. The settlement occurs immediately after the load increase due to the water level. A small part of the settlement only occurs after a few days. The fluctuations during the day are due to temperature changes.

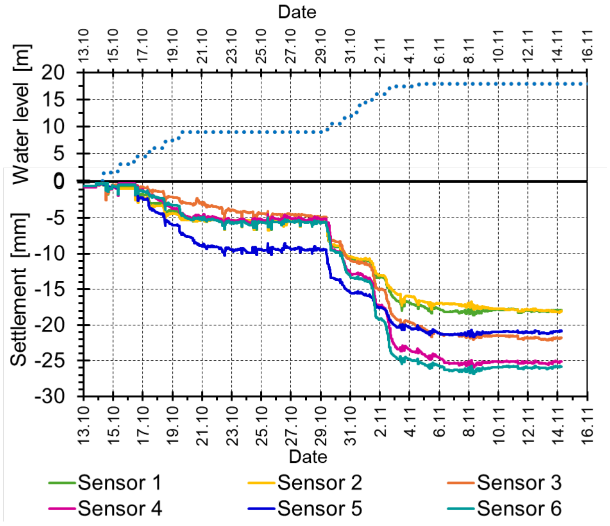

Figure 11. Edge settlement of tank A during hydro test.

An inclinometer was installed under tank A from the edge of the tank to the centre. This allows a settlement analysis of the cross-section. The sag of the slab can be visualised. This data was also recorded over the entire filling procedure, see Fig. 12.

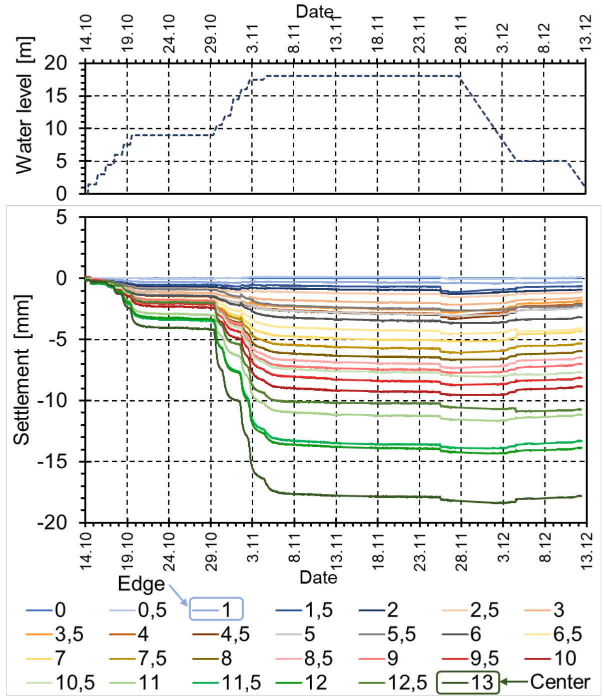

Figure 12. Settlement of inclinometer every 0.5m.

The different lines show the final settlement at different points (each meter) of the inclinometer. The initial value 0 is the fixed reference of the inclinometer one meter outside the tank. Line 1 is the measuring point directly under the edge of the tank and Line 13 is the measuring point in the centre of the tank.

The center settlements are larger than those at the edge of the tank. There is a final sag of about 19 mm from center to edge. The resulting settlement in the center after full loading is 39 mm, which means a typical settlement ratio of s_{center}/S_{edge} = 1.9 for that tank geometry.

The total settlement on the following stages is shown in Fig. 13 as a cross-section.

- 14.10.: one day before filling
- 18.10.: shortly after reaching 30 % fluid hight
- 20.10.: shortly after reaching 50 % fluid hight
- 29.10: after 10 days of 50 % fluid hight
- 05.11: shortly after reaching 100 % fluid hight
- 12.11: after 10 days of 100 % loading

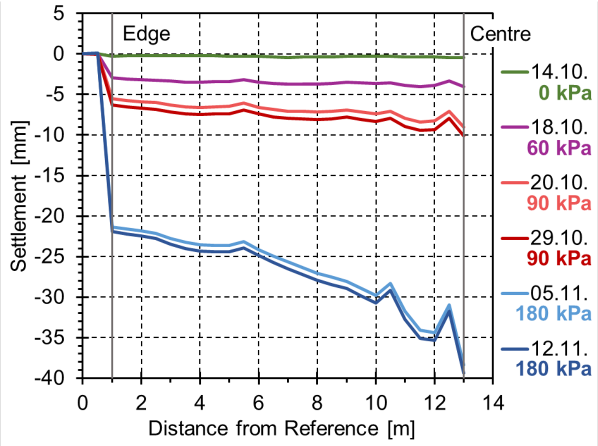

Figure 13. Cross-section settlement from edge to center of the slab.

Most part of the settlement occurred after just one day, because the CMC are anchored in coarse-grained soils. Two weeks after full filling, the settlement has only increased by approx. 1 mm, so the CMC eliminated a potential consolidation or creep behaviour.

The measured settlement can be compared with the calculated settlement using the methodology described above for different stages during filling process. Fig. 14 shows the calculated and measured settlement depending on the load induced by the fluid weight.

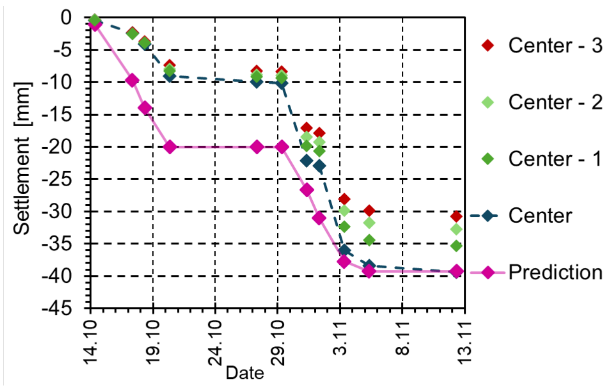

Figure 14. Measurement vs Settlement prediction based on pressuremeter of tank A.

The predicted settlement based on PMT and the measurement in the centre of tank A show good agreement. At a lower filling level, the calculation provides a higher settlement.

Further measurement equipment was installed under tank A to analyse the load distribution between the CMC and the soil between the columns. These measurement results will be presented in further publications.

5. Conclusions

Large steel tanks on marginal ground conditions can be founded on shallow foundations after ground improvement works. Due to accurate soil parameters from the PMT, it was possible to implement an efficient ground improvement solution using Controlled Modulus Columns (CMC). Menard pressuremeter tests were carried out to get more reliable soil parameters as well as the skin friction and the tip resistance for the rigid inclusion design.

Observations of the behaviour of tanks during hydrostatic load test provide a basis for the expected performance of tanks during service. The results of settlement monitoring carried out during hydrostatic load test shows a good agreement with the predictions based on PMT.

The tanks will be in service for the next 30 years. The present case study conclude that geotechnical input parameters are the most influencing factor for optimizing foundations in terms of costs and CO_2 Consumption for all kind of foundation works.

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